Chapter 1

Biosolids Management Practices and Regulatory Requirements

1.0 Introduction

In 1948, the U.S. Congress enacted the original Federal Water Pollution Control Act (FWPCA). Since its passage, the FWPCA has been amended many times. Two of the most important amendments were (1) the 1972 FWPCA Amendments and (2) the 1977 Clean Water Act Amendments [10]. These amendments define the basic national framework for water quality and water pollution control in the United States. Today, the comprehensive federal law is simply referred to as the U.S. Clean Water Act (CWA).

The primary objective of the CWA is to restore and maintain the chemical, physical, and biological integrity of the nation's waters. To prevent contamination and deterioration of water quality, wastewater from industrial, commercial, and residential activities is treated at wastewater treatment plants (WWTPs) before it is discharged to surface water or groundwater (Fig. 1.1).

At present, there are more than 15,000 municipal wastewater treatment plants or publicly owned treatment works (POTWs) in the United States that process over 34 billion gallons of domestic sewage and other wastewater each day [21]. Sewage sludge represents the largest source of residual solids generated during the treatment of municipal wastewater by POTWs as well as by privately and federally

Figure 1.1 Aerial view of typical municipal wastewater treatment plant (WWTP). (*Courtesy of Waterlink, Inc.*)

owned wastewater treatment works. The annual amount of sewage sludge (i.e., biosolids) generated during the treatment of domestic sewage is estimated at approximately 47 pounds for every individual in the United States. Figure 1.2 illustrates the collection and treatment of domestic and industrial wastewater resulting in the production, treatment, use, and disposal of sewage sludge.

In the United States, the use or disposal of sewage sludge has been regulated under various federal environmental statutes. Land disposal and reuse of sewage sludge were regulated initially under the solid waste disposal regulations of 40 Code of Federal Regulations (CFR) Part 257, which was jointly promulgated under the 1976 Resource Conservation and Recovery Act (RCRA) and Sections 405 and 307 of the 1977 CWA Amendments. RCRA (PL 94-580) required that solid wastes be used or disposed in a safe and environmentally acceptable manner. Sewage sludge was included by definition in the RCRA provisions relating to solid waste management. The 1977 CWA Amendments (PL 95-217) contained two major provisions affecting sewage sludge use and disposal. First, Section 405 of the 1977 CWA Amendments required that the U.S. Environmental Protection Agency (USEPA) issue guidelines and regulations for the disposal and reuse of sewage sludge. Second, Section 307 of the CWA Amendments required pretreatment of industrial wastes if such wastes, when discharged

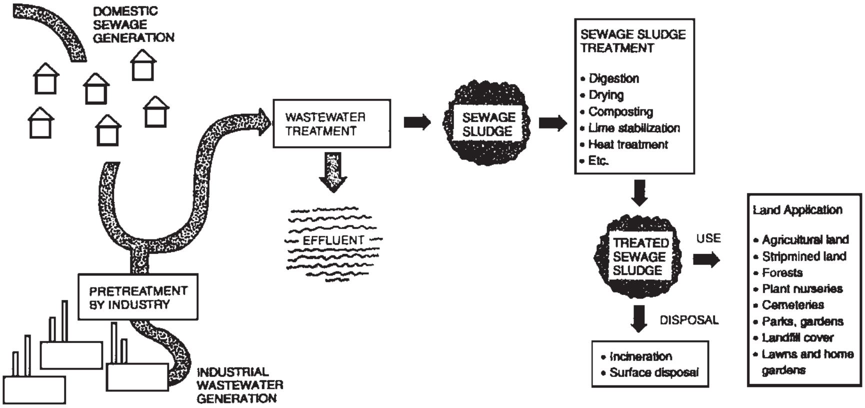


Figure 1.2 Schematic illustration of the generation, treatment, use, and disposal of sewage sludge.

into municipal sewage collection systems, inhibited wastewater treatment or the beneficial use of sewage sludge. In addition to RCRA and the CWA Amendments, the 1972 Marine Protection, Research and Sanctuaries Act (MPRSA) regulated the discharging of sewage sludge to oceans and estuaries until the Ocean Dumping Ban Act of 1988 prohibited this disposal practice [10].

In 1987, Section 405(d) of the CWA was amended to require the USEPA to establish sewage sludge pollutant standards that adequately protected public health and the environment from any reasonably anticipated adverse effects of pollutants in sewage sludge that is used or disposed [21]. These regulations were to include identification of the various beneficial uses for sludge while specifying factors to be taken into account in developing management practices for each type of reuse or disposal option. The 1987 CWA Amendments also required that any CWA Section 402 (National Pollutant Discharge Elimination System, NPDES) permit include sewage sludge use or disposal standards unless these requirements were included in another permit. The 1987 CWA Amendments expanded the regulated universe to include all treatment works treating domestic sewage (TWTDS), even those not requiring an NPDES permit. TWTDS include all sewage sludge or wastewater treatment systems used to store, treat, recycle, and reclaim municipal or domestic sewage.

In summary, to maintain regulatory compliance with the CWA requirements, POTWs must adopt and implement federally mandated procedures ensuring the proper treatment, use, and disposal of sewage sludge. Furthermore, as a result of Section 405 of the 1977 and 1987 CWA Amendments, increased use of sewage sludge recycling has become a clear objective of U.S. environmental policy.

1.0.1 Summary statistics for sewage sludge use and disposal in the United States

In 1988, the USEPA collected information on the use or disposal of sewage sludge through a two-part National Sewage Sludge Survey (NSSS). In Part I, a questionnaire survey was used to obtain both technical and financial information on the sewage sludge use or disposal practices employed by POTWs. In Part II, information on the quality of sewage sludge was obtained by analyzing sewage sludge from several POTWs for specific pollutants. Results from the NSSS were used as the basis for establishing several of the sewage sludge pollutant limits found in the 40 CFR Part 503 sludge rule (see Sec. 1.1). The number of POTWs and the magnitude of sewage sludge generated (dry-mass basis) as reported in the 1988 NSSS are sumarized in Table 1.1.

In 1988, POTWs with a design flow rate of over 100 million gallons per day (MGD) accounted for 30.1 percent of the sewage sludge used or disposed by POTWs. POTWs with a design flow rate of between 10 and 100 MGD used or disposed 38.4 percent of the total annual amount of sewage sludge generated in the United States, while POTWs with a flow rate of between 1 and 10 MGD used or disposed 24.0 percent of the sewage sludge. In contrast, while they account for more than half of all POTWs in the United States, POTWs with a flow rate of less than 1 MGD generated only 7.5 percent of the annual amount of sewage sludge used or disposed.

The 1988 NSSS identified four principal categories of practices employed by POTWs for the reuse and or disposal of sewage sludge. Table 1.2 illustrates that, in 1988, the most prevalent sludge reuse/disposal practice was land application (34.6 percent), followed by sewage sludge codisposal in municipal solid waste landfills. With respect to the total mass of sewage sludge generated, codisposal in municipal landfills was the preferred disposal practice in 1988, accounting for 33.7 percent of the total amount of sludge generated.

TABLE 1.1 Number of Publicly Owned Treatment Works (POTWs), Actual Flow, and Estimated Sewage Sludge Quantities in the United States*

POTW flow rate (MGD)†	No. of POTWs	Quantity of sewage sludge (dmt)‡	Percent
>100	35	2,120,512	30.1
10–100	459	2,709,604	38.4
1–10	2,666	1,692,086	24.0
<1	9,588	530,339	7.5
TOTAL	12,748	7,052,540	100.0

*Adapted from ref. [18].

†MGD, million gallons per day.

‡dmt, dry metric ton (1000 kg) = 0.9072 · U.S. ton. (kg = 2.2 lb.)

TABLE 1.2 Use and Disposal Practices of Sewage Sludge in the United States*

Use/disposal practice	Percentage of POTWs using a particular practice	Percentage of total sewage sludge generated
Land application	34.6	33.5
Codisposal landfill	22.2	33.7
Incineration	2.8	16.1
Surface disposal	10.1	10.4
Unknown transfers	30.3	6.3†

*Adapted from refs. [21,23].
†Ocean disposal—banned in 1988.

Due to the increased level to which municipal wastewater is now required to be treated, it is anticipated that the sewage sludge volumes have increased significantly since 1988. Some of the regulatory requirements that have mandated higher levels of wastewater treatment include (1) the reduction in permissible levels of nitrogen and phosphorus in wastewater discharges to surface waters and (2) the conversion of primary treatment-only facilities to full secondary treatment [21]. In addition to the increased stringency in federal and local water quality discharge standards, industrial pretreatment programs have had a significant impact on sewage sludge management. With the overall improvement in sewage sludge quality as a result of implementation of industrial pretreatment programs, a large volume of sewage sludge can now be directed toward beneficial use, such as land application and the production and sale of sewage sludge amendment products (e.g., compost, heat-dried pellets, alkaline-stabilized soil additives, and soil substitute products). To document the impact of changing water quality standards on sewage sludge quality and generation rates, the USEPA is currently developing the scope for a second national sewage sludge survey [2,3].

Although regulatory compliance issues have led to consideration of new approaches to sewage sludge recycling, in some cases, rising transportation and labor costs have stimulated changes in sewage sludge management. For example, wastewater treatment authorities recently have been faced with dramatic increases in sewage sludge disposal costs. In the 1970s, costs for sewage sludge disposal generally were less than $100 per dry ton, whereas recent short-term private contracts to implement land-based sewage sludge disposal alternatives have been reported to be as high as $800 per dry ton [23]. Such increases in disposal costs, along with the difficulties in siting sewage sludge disposal facilities, have led to situations where long-distance sewage sludge transport becomes necessary (e.g., New York City sewage sludge transported to Arizona for reuse/disposal). With such high sewage sludge management costs, more attention is being paid to the development and implementation of innovative approaches to sewage sludge recycling.

1.0.2 Institutional barriers and liability issues

Although the technological feasibility of innovative methods for recycling sewage sludge can be demonstrated repeatedly, in many cases, achieving public acceptance of new sludge management methods becomes insurmountable. The reluctance to accept the results of technological innovation directly influences the numerous political, regulatory, and financial policy barriers that wastewater treatment authorities must address in developing sludge management programs. The skepticism recorded about the proposed changes in current sludge disposal/recycling practices and policies included legitimate public concerns over protecting public health, the environment, and tax revenue. In recent years, potential liability associated with the beneficial use of sewage sludge has become a concern to both proponents and opponents of sewage sludge recycling [30].

Under federal law, anyone responsible for a hazardous substance release that is not federally permitted is liable for the costs of cleaning up the release under the 1980 Comprehensive Environmental Response, Compensation and Liability Act (CERCLA, or Superfund). Potential Superfund liability has created concerns over the potential for future liability associated with sewage sludge use/disposal practices for both sludge generators and landowners. In addition, groups such as the Farm Credit Bank and various food processing organizations also have raised concerns over potential liabilities [21]. The potential for litigation brought on behalf of food processors and/or the public has created significant psychological and financial barriers to farmers who would otherwise use sewage sludge as either a low-cost fertilizer or soil amendment.

To ease some of the liability concerns, the 40 CFR Part 503 sludge rule clarifies that Superfund liability does not apply to the beneficial use of sewage sludge [21]. Moreover, the Farm Credit Bank in conjunction with the USEPA has developed an indemnification statement that is currently being employed by several companies to clarify the legal responsibilities of the sludge preparer, land applier, and farmer when sewage sludge reuse/disposal projects are in compliance with applicable standards and management practices of the 40 CFR Part 503 rule [21].

Finally, overcoming the nontechnical issues such as public perception and legal liability fears may prove to be the greatest barrier facing sewage sludge management authorities in the future. Studies of public acceptance and institutional barriers to changes in sewage sludge management practices suggest that techniques such as (1) providing adequate public involvement in the decision-making process, (2) addressing public nuisance concerns early, (3) use of stakeholder advisory groups, and (4) aggressive education programs may minimize opposition to implementation of innovative sludge management practices.

1.1 Regulatory Aspects to Biosolids Management

In compliance with the requirements of Section 405(d) of the 1987 CWA Amendments, on February 19, 1993, the final version of 40 CFR Part 503, "Standards for the Use or Disposal of Biosolids," was published in the *Federal Register* [22]. In the 40 CFR Part 503 rule, the term *biosolids* was introduced as a replacement for the term *sewage sludge.* The new term was designed to reflect the beneficial characteristics of the residual solids generated from municipal wastewater treatment. The 40 CFR Part 503 rule defines *biosolids* as the final solid, semisolid, or liquid residue generated during the treatment of domestic sewage in a municipal wastewater treatment plant [19]. The 40 CFR Part 503 rule applies to biosolids generated from the treatment of domestic wastewater as well as domestic septage.

Biosolids permitting requirements apply to all TWTDS, i.e., facilities that generate, treat, or provide disposal of biosolids, including nondischarging and biosolids-only (i.e., sludge) facilities. A TWTDS facility must apply for a federal biosolids permit from the USEPA or an approved state biosolids program if it manages biosolids that are ultimately subject to the 40 CFR Part 503 rule. In other words, if the biosolids are applied to land, placed in a surface disposal site, incinerated, or sent to a municipal solid waste monofill, the TWTDS facility requires a permit under the 40 CFR Part 503 rule.

The 40 CFR Part 503 rule does not apply to materials such as grease trap residues or other nondomestic wastewater residues pumped from commercial facilities such as solids produced by industrial wastewater treatment facilities or grit and screenings from publicly owned treatment works (POTWs). Wastewater biosolids disposed in municipal solid waste landfills or used as landfill cover material are regulated by federal and local solid waste regulations [21].

The 40 CFR Part 503 rule was designed to protect public health and the environment from any reasonably anticipated adverse effects of pollutants that may be present in biosolids. A schematic diagram illustrating the various components of the 40 CFR Part 503 rule is provided in Fig. 1.3. The provisions of the 40 CFR Part 503 rule are consistent with USEPA's policy of promoting beneficial uses of biosolids. Most of the requirements contained in the 40 CFR Part 503 rule were generated based on results from extensive multimedia risk-assessment studies conducted by the USEPA [28].

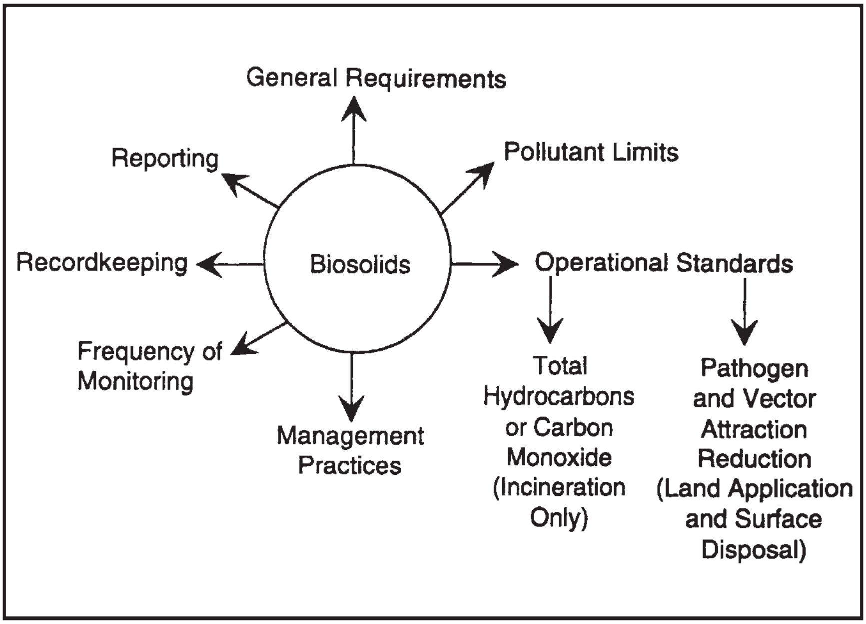


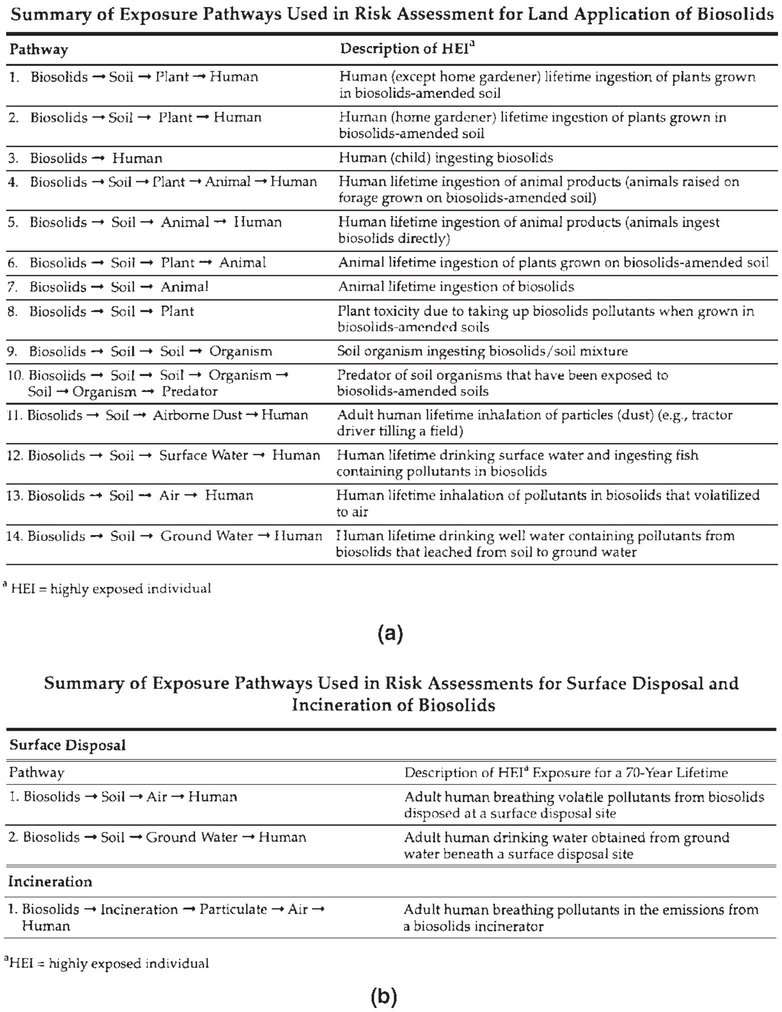
Figure 1.3 Various components of the 40 CFR Part 503 sludge rule.

1.1.1 Risk-assessment basis for the 40 CFR Part 503 rule

The Part 503 rule was developed with the realization that the use or disposal of biosolids may result in measurable changes in the environment. The biosolids risk-assessment process provided a scientific basis for determining acceptable environmental change when biosolids were used or disposed. *Acceptable environmental change* has been defined by the USEPA as any measurable change that still maintains adequate protection for public health and the environment. The risk-assessment procedures used in developing the sludge rule (40 CFR Part 503) were based on the methodology formulated by the National Academy of Science, which included the following four steps: (1) hazard identification, (2) exposure assessment, (3) dose-response evaluation, and (4) risk characterization [28].

To evaluate both the human health and environmental risks associated with biosolids reuse and disposal practices, the USEPA analyzed the health impacts on humans, animals, plants, and soil organisms resulting from exposure to pollutants found in biosolids. The USEPA evaluated 14 various exposure pathways in which human beings and the environment may be exposed to pollutants contained in land-applied biosolids. Similarly, the USEPA evaluated two exposure pathways through which human beings and the environment may be affected by the surface disposal of biosolids, while the impact of biosolids incineration on human health was modeled assuming one principal exposure pathway [25]. Figure 1.4 summarizes the various exposure pathways evaluated by the USEPA in the development of the 40 CFR Part 503 rule.

Employing human health and environmental risk assessments to establish permissible biosolids use and disposal standards represents a paradigm shift away from the policy-driven methodology employed by many European countries and Canadian provinces.



Summary of Exposure Pathways Used in Risk Assessment for Land Application of Biosolids

Pathway	Description of HEI[a]
1. Biosolids → Soil → Plant → Human	Human (except home gardener) lifetime ingestion of plants grown in biosolids-amended soil
2. Biosolids → Soil → Plant → Human	Human (home gardener) lifetime ingestion of plants grown in biosolids-amended soil
3. Biosolids → Human	Human (child) ingesting biosolids
4. Biosolids → Soil → Plant → Animal → Human	Human lifetime ingestion of animal products (animals raised on forage grown on biosolids-amended soil)
5. Biosolids → Soil → Animal → Human	Human lifetime ingestion of animal products (animals ingest biosolids directly)
6. Biosolids → Soil → Plant → Animal	Animal lifetime ingestion of plants grown on biosolids-amended soil
7. Biosolids → Soil → Animal	Animal lifetime ingestion of biosolids
8. Biosolids → Soil → Plant	Plant toxicity due to taking up biosolids pollutants when grown in biosolids-amended soils
9. Biosolids → Soil → Soil → Organism	Soil organism ingesting biosolids/soil mixture
10. Biosolids → Soil → Soil → Organism → Soil → Organism → Predator	Predator of soil organisms that have been exposed to biosolids-amended soils
11. Biosolids → Soil → Airborne Dust → Human	Adult human lifetime inhalation of particles (dust) (e.g., tractor driver tilling a field)
12. Biosolids → Soil → Surface Water → Human	Human lifetime drinking surface water and ingesting fish containing pollutants in biosolids
13. Biosolids → Soil → Air → Human	Human lifetime inhalation of pollutants in biosolids that volatilized to air
14. Biosolids → Soil → Ground Water → Human	Human lifetime drinking well water containing pollutants from biosolids that leached from soil to ground water

[a] HEI = highly exposed individual

(a)

Summary of Exposure Pathways Used in Risk Assessments for Surface Disposal and Incineration of Biosolids

Surface Disposal	
Pathway	Description of HEI[a] Exposure for a 70-Year Lifetime
1. Biosolids → Soil → Air → Human	Adult human breathing volatile pollutants from biosolids disposed at a surface disposal site
2. Biosolids → Soil → Ground Water → Human	Adult human drinking water obtained from ground water beneath a surface disposal site
Incineration	
1. Biosolids → Incineration → Particulate → Air → Human	Adult human breathing pollutants in the emissions from a biosolids incinerator

[a] HEI = highly exposed individual

(b)

Figure 1.4 Various exposure pathways evaluated by the USEPA in developing the 40 CFR Part 503 rule: (*a*) exposure pathways for land-applied biosolids; (*b*) exposure pathways for surface disposal and incineration of biosolids.

Policy-driven approaches used to establish biosolids quality standards allow only small, incremental increases of pollutants into the environment. A typical example of a policy-driven biosolids land-application standard might include a mandate requiring that the soil metal concentrations resulting from biosolids land application shall not be permitted to exceed the 95th percentile of background soil concentrations. Unfortunately, these policy-driven approaches to establishing environmental standards not only result in overly conservative pollutant limits but often have neither a scientific nor technical basis.

In most cases, the USEPA determined that risk-based pollutant limits could be calculated to achieve the goal of protecting public health and the environment. However, in three cases, risk-assessment methodologies were not sufficiently developed to provide a reasonable estimate of risk. These cases included establishing (1) pathogen reduction criteria for land-applied biosolids, (2) vector attraction reduction criteria for land-applied biosolids, and (3) total hydrocarbon (THC) limits in biosolids incineration emissions. In lieu of developing a risk-based pollutant limit for these cases, the USEPA adopted technology-based biosolids management requirements to ensure an adequate margin of protection for human health and the environment [19,25].

Once risk assessments were completed, the basic approach adopted by the USEPA to establish the permissible pollutant concentrations was to use the lower of either (1) the risk-derived concentration or (2) the 99th percentile concentration derived from the 1988 USEPA NSSS [18]. The NSSS summarized pollutant concentration data in biosolids generated from 186 statistically representative POTWs. In the case of the pollutants chromium and selenium, the 99th percentile concentration found in the 1988 NSSS was lower than the concentration derived from risk assessments. Therefore, the initial limiting concentration specified for both metals for land-applied biosolids was the 99th percentile concentration found in the 1988 NSSS.

The USEPA received many comments from both the regulated community and the public after the initial promulgation of the 40 CFR Part 503 rule [30]. In addition to public comments, several industry groups and POTWs initiated lawsuits against the USEPA contending that the land-application pollutant limits set for chromium and selenium in the rule were overly stringent [30]. In these particular lawsuits, the District of Columbia Circuit Court concluded that Section 405 of the CWA mandated that only risk-based pollutant concentrations could be promulgated in the 40 CFR Part 503 rule. Since the maximum chromium level reported in the 1988 NSSS and subsequently investigated in the USEPA risk assessments did not pose a significant risk to human health and the environment, the USEPA decided to delete all chromium limits for land-applied biosolids from the 40 CFR Part 503 rule. Moreover, the USEPA revised the selenium pollutant concentration limits, concluding that it could not legally adopt a more stringent concentration limit for selenium in land-applied biosolids than the risk-assessment-based concentration of 100 mg/kg (dry solids basis).

1.2 Land Application of Biosolids

Land application of biosolids includes all forms of applying bulk or bagged biosolids to land for beneficial use. Beneficial uses include biosolids application to (1) agricultural land for food production, (2) agricultural land for production of feed and fiber crops, (3) pasture and range land, (4) nonagricultural land (e.g., forests), (5) disturbed lands (e.g., highway embankments, mine reclamation, etc.), (6) construction sites and gravel pits, (7) public contact sites (e.g., parks and golf courses), and (8) home lawns and gardens. Figure 1.5 presents photographs depicting the land application of various types of biosolids for agricultural production.

The 40 CFR Part 503 rule requires that any person applying biosolids to land or any person who prepares biosolids for beneficial use must obtain a permit. The 40 CFR Part 503 rule defines a *person* as an individual, association, partnership, corporation, municipality, state or federal agency, or any individual working on behalf of one of these entities. The self-implementing nature of the 40 CFR Part 503 rule requires that biosolids land appliers comply with the rule even if they have not applied for and/or have not been issued a permit covering biosolids use. Similarly, USEPA (or an approved state regulatory agency) can take enforcement actions directly against persons who violate the 40 CFR Part 503 requirements regardless of whether or not they have been issued a biosolids permit [21,30].

Regardless of the land-application end use (i.e., agricultural or nonagricultural), seven types of requirements must be met to legally apply biosolids to land: (1) general requirements, (2) pollutant limits, (3) management practices, (4) operational standards covering pathogen and vector attraction reduction requirements, (5) frequency of monitoring requirements, (6) recordkeeping requirements, and (7) reporting requirements. Each of these requirements is discussed in further detail in the following sections.

1.2.1 General requirements for land-applied biosolids

Subpart B of the 40 CFR Part 503 rule specifies the legal requirements for land applying biosolids and/or any material derived from biosolids (e.g., land application of biosolids composted with yard wastes). General requirements mandate that the preparer of bulk biosolids provide any subsequent preparer and any land applier of biosolids with the appropriate "notice and information" certification necessary to

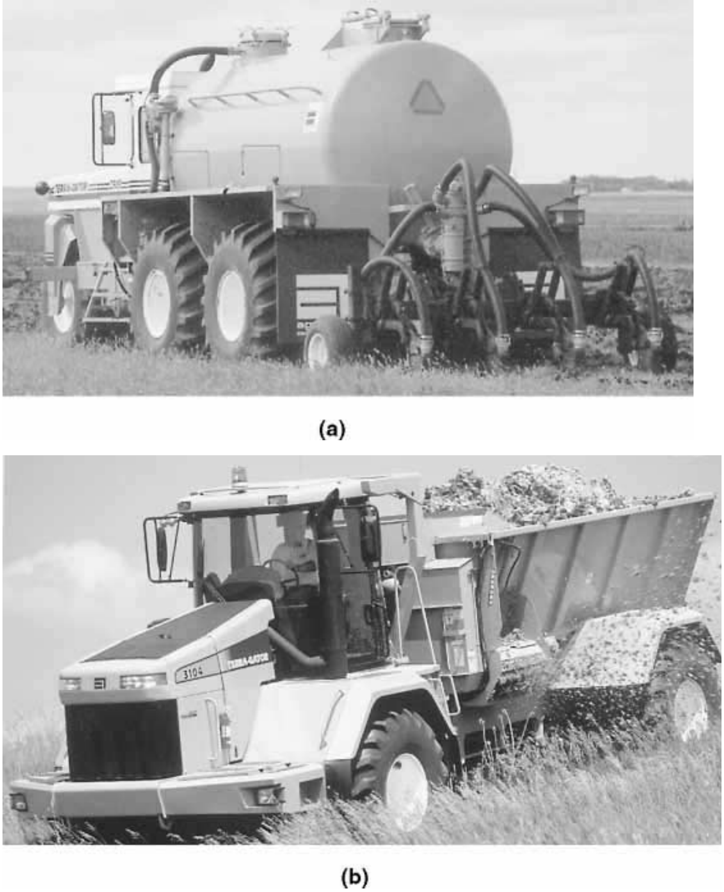


Figure 1.5 Biosolids land application for agricultural production: (*a*) subsurface injection of liquid biosolids; (*b*) surface application of dewatered biosolids. (*Courtesy of Ag-Chem Equipment Company, Inc.*)

comply with Subpart B. Subpart B requirements (i.e., pollutant limits, class of pathogen control, and vector attraction reduction) are designed to ensure that all preparers of biosolids that do not meet specific quality requirements have written agreement with any biosolids land applier before land application of biosolids commences.

In addition to having a written contract with the land applier, the preparers of land-applied biosolids must provide the state regulatory authority with information pertaining to the site location, time period of application, and the name, address, telephone numbers, and NPDES permit number of the biosolids applier. The regulation also requires all land appliers of bulk biosolids that are subject to the cumulative pollutant limits to provide written notification to the permitting authority for the state in which the bulk biosolids are applied.

If, for any reason, bulk biosolids subject to cumulative limits have been applied to the site but the cumulative amount of pollutants applied is unknown, no further amount of biosolids can be applied to the site unless it can meet the more stringent pollutant concentration limits (see Sec. 1.2.2).

In addition to the general requirements, the 40 CFR Part 503 rule requires that biosolids meet two levels of quality with respect to pollutant limits, i.e., pollutant ceiling concentrations and any one of the following: (1) pollutant concentration limits, (2) cumulative loading rate limits, or (3) annual pollutant loading rate limits. The 40 CFR Part 503 rule also has created two levels of biosolids quality with respect to pathogen concentrations, i.e., Class A and Class B biosolids. Finally, the 40 CFR Part 503 rule permits two types of approaches for meeting vector attraction reduction, namely, (1) biosolids processing or (2) use of physical barriers. The following sections describe each of these requirements and their impact on the suitability of biosolids to be applied to land.

1.2.2 Pollutant limits

A central feature of the biosolids land-application requirements is pollutant limits. It should be noted that at the time of this writing, the only regulated pollutants for land-applied biosolids were heavy metals. It should be noted that the USEPA recently promulgated a proposed limit of 300 parts per trillion (300 ppt) for dioxin in land-applied biosolids (dry-mass basis), but this standard has not yet been codified into law [33]. The heavy metal pollutant limits are divided into two types: (1) concentration limits (i.e., limits on the concentrations of pollutants in biosolids) and (2) loading rate limits (i.e., limits on the rate at which pollutants may be applied to land). Concentration limits are further divided into two types: (1) ceiling concentration limits, which govern whether a biosolids can be applied to land at all, and (2) pollutant concentration limits, which define biosolids that are exempted from meeting pollutant loading rate limits, certain recordkeeping requirements, etc.

All land-applied biosolids must meet the ceiling concentration limits for heavy metals. The ceiling concentrations are the maximum concentration limits for nine heavy metals typically found in biosolids (Table 1.3).

If the concentration limit for any one of the heavy metals exceeds the level given in Table 1.3, the biosolids cannot be applied to land. The ceiling concentration limits for heavy metals were included in 40 CFR Part 503 to encourage industrial pretreatment efforts and to prevent the introduction of heavily contaminated materials into the environment.

Depending on the effectiveness of industrial pretreatment programs and wastewater treatment operation, the heavy metal concentrations in biosolids may be reduced to the pollutant concentration limits (see Table 1.3). POTWs whose biosolids meet pollutant concentration limits are offered two important advantages with regard to biosolids land application, namely, (1) there are no limits on the lifetime quantity of pollutants that can be applied to a site, and (2) the biosolids application rate depends only on the agronomic rate (see Sec. 1.3.2.1).

Like concentration limits, loading rate limits are also divided into two types: (1) cumulative pollutant loading rates (CPLRs) and (2) annual pollutant loading rates (APLRs) (Table 1.4). Bulk biosolids that meet ceiling concentration limits but do not meet pollutant concentration limits must meet cumulative pollutant loading rates, which specify the total lifetime quantity of pollutants that can be applied to a site (see Table 1.4). Once the cumulative pollutant loading rate has been reached, no more biosolids of this quality may be applied to a site.

In contrast to biosolids that are applied in bulk, biosolids that are sold or given away in bags or other containers meeting ceiling limits but not meeting pollutant concentration limits must meet APLRs, which specify the total amount of pollutant that can be applied to a site in any one year. The following sections provide additional information specific to the land application of bulk and bagged biosolids.

1.2.2.1 Land application of bulk biosolids. The 40 CFR Part 503 rule mandates that bulk biosolids cannot be applied to agricultural land, forest land, or a public contact site at a rate greater than the agronomic rate. The *agronomic rate* is defined as the biosolids application rate that provides nitrogen (or phosphorus) at a rate that just satisfies the crop nutrient requirements. Figure 1.6 is a photograph depicting

TABLE 1.3 Concentration Limits for Biosolids Applied to Land*

Heavy metal	Ceiling concentration limits (mg/kg)†	Pollution concentration limits‡ (mg/kg)†
Arsenic	75	41
Cadmium	85	39
Copper	4300	1500
Lead	840	300
Mercury	57	17
Molybdenum	75	—
Nickel	420	420
Selenium	100	36
Zinc	7500	280

*Adapted from ref. [31].
†Dry-weight basis.
‡Monthly average concentration.

TABLE 1.4 Loading Rate Limits for Land-Applied Biosolids*

Pollutant	Cumulative pollutant loading rate limits (kg/ha)†	Annual pollutant loading rate limits (kg/ha)†
Arsenic	41	2.00
Cadmium	39	1.90
Copper	1500	75.00
Lead	300	15.00
Mercury	17	0.85
Nickel	420	21.00
Selenium	100	5.00
Zinc	2800	140.00

lb/acre = 0.8922 · kg/ha.
*Taken from refs. [24,31].
†To qualify as exceptional quality biosolids, none of the heavy metal concentration can exceed the pollutant concentration limits.

Figure 1.6 Photograph of biosolids being applied to land in bulk at a forest site.

the land application of biosolids in bulk at a forest site. Bulk biosolids can be applied to land at a reclamation site at a rate greater than the agronomic rate if authorized by the permitting agency [31].

In all cases, when bulk biosolids that do not meet pollutant concentration limits are applied to land, the application rate and site life must be determined as part of the overall land-application design. Preparers or appliers of bulk biosolids have the option of using the CPLR values to estimate either (1) a maximum site life based on a given biosolids application rate or (2) a maximum annual whole sludge application rate (AWSAR) in terms of dry metric tons (dmt) per hectare (or U.S. tons per acre) given a design site life. In most cases, POTWs will use their existing biosolids land-application rate (i.e., AWSAR) to estimate site life if their biosolids application rate is at or below the agronomic rate. However, the latter approach to biosolids land-application design is sometimes used in cases where it is necessary for the POTW to adjust the AWSAR downward to extend site life.

To estimate site life, an APLR must be estimated for each regulated pollutant given the existing biosolids land-application rate. The APLR is obtained by multiplying the concentration of each pollutant by the AWSAR, as illustrated by Eq. (1.1).

$$\text{APLR}\left(\frac{\text{kg}}{\text{ha}\cdot\text{yr}}\right) = \text{biosolids pollutant concentration}\left(\frac{\text{mg}}{\text{kg}}\right) \cdot \text{AWSAR}\left(\frac{10^3\ \text{kg}}{\text{ha}\cdot\text{yr}}\right)\cdot\frac{\text{kg}}{10^6\ \text{mg}} \qquad (1.1)$$

Once the APLR is estimated, the site life can be obtained by dividing the CPLR by the derived APLR [Eq. (1.2)].

$$\text{Site life (years)} = \frac{\text{CPLR}\left(\dfrac{\text{kg}}{\text{ha}}\right)}{\text{APLR}\left(\dfrac{\text{kg}}{\text{ha}\cdot\text{yr}}\right)} \qquad (1.2)$$

When site life is calculated for each regulated pollutant, the shortest time duration becomes the design site life for the biosolids land-application program. Example 1.0 illustrates the use of Eqs. (1.1) and (1.2) in estimating site life for a biosolids land-application system.

Example 1.0 The CPLR for arsenic is 41 kg/ha. If the concentration of arsenic in the biosolids is 10 mg/kg (dry weight), estimate the site life based on arsenic if the AWSAR is to be maintained at 15 dmt/ha ($15\cdot10^3$ kg/ha).

solution

Step 1. Estimate the APLR using Eq. (1.1).

$$\text{APLR}\left(\frac{\text{kg}}{\text{ha}\cdot\text{yr}}\right) = \text{biosolids pollutant concentration}\left(\frac{\text{mg}}{\text{kg}}\right) \cdot \text{AWSAR}\left(\frac{10^3\ \text{kg}}{\text{ha}\cdot\text{yr}}\right)\cdot\frac{\text{kg}}{10^6\ \text{mg}}$$

$$= \frac{10\ \text{mg}}{\text{kg}}\cdot\frac{15\cdot10^3\ \text{kg}}{\text{ha}\cdot\text{yr}}\cdot\frac{\text{kg}}{10^6\ \text{mg}}$$

$$= \frac{0.15\ \text{kg}}{\text{ha}\cdot\text{yr}}$$

Step 2. Estimate the site life using Eq. (1.2).

$$\text{Site life (years)} = \frac{\text{CPLR}\left(\dfrac{\text{kg}}{\text{ha}}\right)}{\text{APLR}\left(\dfrac{\text{kg}}{\text{ha} \cdot \text{yr}}\right)}$$

$$= \frac{\dfrac{41\ \text{kg}}{\text{ha}}}{\left(\dfrac{0.15\ \text{kg}}{\text{ha} \cdot \text{yr}}\right)} = 273\ \text{years}$$

NOTE: In the actual biosolids land-application design, similar calculations would be performed for each of the nine regulated heavy metals. The metal yielding the shortest site life would become the limiting pollutant.

1.2.2.2 Land application of bagged biosolids. When the biosolids preparer cannot control the number of biosolids applications made to a site directly (i.e., when biosolids in bags or other containers are given away or sold), APLRs must be met (see Table 1.4). In this case, as long as the annual limits are met, the total pollutant load to the site over time will not exceed levels identified through the USEPA risk assessments as protective of human health and the environment [28].

For the case of biosolids sold or given away in bags or other containers, only the AWSAR (in dry metric tons/hectare or dry U.S. tons/acre) needs to be determined. To estimate the AWSAR, Eq. (1.3) is used. It should be noted that Eq. (1.3) employs the APLR limits found in Table 1.4.

$$\text{AWSAR}\left(\frac{\text{dry metric tons}}{\text{hectare}}\right) =$$

$$\frac{\text{APLR}\left(\dfrac{\text{kg}}{\text{ha} \cdot \text{yr}}\right)}{\text{concentration of pollutant in biosolids}\left(\dfrac{\text{mg}}{\text{kg}}\right) \cdot \left(\dfrac{\text{kg}}{10^6\ \text{mg}} \cdot \dfrac{10^3\ \text{kg}}{\text{dmt}}\right)}$$

$$= \frac{\text{APLR}\left(\dfrac{\text{kg}}{\text{ha} \cdot \text{yr}}\right)}{\text{concentration of pollutant biosolids}\left(\dfrac{\text{mg}}{\text{kg}}\right) \cdot 0.001} \qquad (1.3)$$

When AWSARs for all nine regulated pollutants are calculated, the lowest AWSAR becomes the limiting application rate for those biosolids. Example 1.1 illustrates the use of Eq. (1.3) in estimating the biosolids application rate for bagged biosolids that ensures that the APLRs are not exceeded.

Example 1.1 The Little County Water Reclamation Facility is considering selling its biosolids to the general public in 100-pound sacks. What is the AWSAR in dry metric tons per hectare per year if the biosolids have the following average heavy metal concentrations?

Metal	Concentration in biosolids (mg/kg)
Arsenic	20.3
Cadmium	52.1
Copper	1133.1
Lead	723.1
Mercury	4.1
Nickel	321.7
Selenium	27.8
Zinc	2241.6

solution

Step 1. Calculate the AWSAR for each regulated pollutant using Eq. (1.3) and the APLRs from Table 1.2. For example, for arsenic, the APLR limit is 2.0 kg/ha·yr (see Table 1.4). Given this APLR, the AWSAR can be estimated as follows:

$$\text{AWSAR}\left(\frac{\text{dry metric tons}}{\text{ha}\cdot\text{yr}}\right) = \frac{\text{APLR}\left(\dfrac{\text{kg}}{\text{ha}\cdot\text{yr}}\right)}{\text{concentration of pollutant in biosolids}\left(\dfrac{\text{mg}}{\text{kg}}\right)\cdot\left(\dfrac{\text{kg}}{10^6\ \text{mg}}\cdot\dfrac{10^3\ \text{kg}}{\text{dmt}}\right)}$$

$$= \frac{\dfrac{2.0\ \text{kg}}{\text{ha}\cdot\text{yr}}}{\left(\dfrac{20.3\ \text{mg}}{\text{kg}}\right)\cdot\left(\dfrac{\text{kg}}{10^6\ \text{mg}}\cdot\dfrac{10^3\ \text{kg}}{\text{dmt}}\right)} = \frac{98.5\ \text{dmt}}{\text{ha}\cdot\text{yr}}$$

Step 2. The AWSAR can be calculated for each heavy metal using the same procedure. The results are given in the following table:

Metal	Concentration in biosolids (mg/kg)	APLR (kg/ha·yr)	AWSAR (metric tons/ha·yr)
Arsenic	20.3	2.0	98.5
Cadmium	52.1	1.9	36.5
Copper	1133.1	75.0	66.2
Lead	723.1	15.0	20.7
Mercury	4.1	0.9	207.3
Nickel	321.7	21.0	65.3
Selenium	27.8	5.0	179.9
Zinc	2241.6	140.0	62.5

Step 3. The limiting AWSAR is 20.7 metric tons/ha·yr, which was estimated for lead. Therefore, the maximum annual biosolids application rate for these biosolids is 20.7 metric tons/ha·yr.

Biosolids sold or given away in bags or other containers are required to have a label attached or a handout sheet provided. The information required on the label or handout sheet includes (1) the name and address of the preparer, (2) a statement prohibiting application except in accordance with the instructions on the label, and (3) the calculated AWSAR that does not cause the APLR to be exceeded (see Sec. 1.2.3.5). Finally, when metal concentrations limit the biosolids loading rate, the nutrient levels must be monitored to determine if supplemental fertilization is required. Example 1.2 illustrates the approach for estimating the level of supplemental fertilization required as a result of biosolids land application.

Example 1.2 The Little County Water Reclamation Facility (Example 1.1) has negotiated with a local nursery to deliver several hundred sacks of biosolids over the course of the growing season to supply nutrients to ornamental shrubbery. If the local nursery estimates that the crop nitrogen requirement is 150 pounds of nitrogen per acre-year, what would be the amount of nitrogen provided by the biosolids relative to the crop nutrient requirements during the first year? Assume that the nitrogen content of the biosolids is 1.5 percent and that 30 percent of the nitrogen (dry-mass basis) is available during the first year of application.

solution

Step 1. From Example 1.1, the AWSAR was estimated to be 20.7 dmt/ha·yr. Since the crop nutrient needs are given in pounds per acre-year, the AWSAR in metric tons per hectare-year must be converted to U.S. units.

$$\frac{9.22 \text{ tons biosolids}}{\text{acre} \cdot \text{yr}} = \frac{20.7 \text{ metric tons}}{\text{ha} \cdot \text{year}} \cdot \left(\frac{1000 \text{ kg}}{\text{metric tons}}\right) \cdot \left(\frac{2.2 \text{ lb}}{\text{kg}}\right) \cdot \left(\frac{\text{ton}}{2000 \text{ lb}}\right) \cdot \left(\frac{\text{ha}}{2.47 \text{ acre}}\right)$$

Step 2. Since nitrogen is only 1.5 percent of the total biosolids added and, of this, only 30 percent is available in the first year, the available nitrogen from biosolids (pounds of nitrogen per acre-year) is calculated as follows:

$$\frac{83.0 \text{ lb nitrogen}}{\text{acre} \cdot \text{yr}} = \frac{9.22 \text{ tons biosolids}}{\text{acre} \cdot \text{yr}} \cdot \left(\frac{2000 \text{ lb}}{\text{ton}}\right) \cdot \frac{0.015 \text{ lb N applied}}{\text{lb biosolids}} \cdot \frac{0.3 \text{ lb N available}}{\text{lb N applied}}$$

Step 3. Since the biosolids can only supply 83.0 pounds of the required nitrogen when applied at a AWSAR of 20.7 metric tons per hectare per year, an additional 67 pounds of nitrogen per acre must be added through supplemental fertilization during each growing season.

NOTE: This is only an approximate method for estimating the supplemental nutrient requirements. In most cases, there is both native nitrogen and nitrogen from previous biosolids application available for meeting crop nutrient requirements. To account for these other nitrogen sources in determining supplemental nitrogen requirements, see Chap. 7.

1.2.3 Management practices

In addition to heavy metal concentrations and loading limits, the 40 CFR Part 503 rule requires that certain management practices be met when biosolids are being applied to land. The only instance where a land applier is exempt from management practices is when exceptional-quality (EQ) biosolids are being applied (see Sec. 1.4).

Management practices were included in the 40 CFR Part 503 rule to (1) constrain risks when actual risks were not evaluated, (2) support risk-modeling assumptions, or (3) ensure proper handling of biosolids. A summary of the management practices for land application of biosolids is given in Table 1.5. Details on each of the land application management practices are provided in the following sections.

1.2.3.1 Endangered species. The 40 CFR Part 503 rule prohibits the land application of biosolids if they could have a negative impact on endangered or threatened species or their designated critical habitat. *Critical habitat* is defined as any environment where an endangered or threatened species lives and grows during its life cycle [24]. It is the responsibility of the land applier to determine if land application of biosolids will adversely affect the endangered species or their critical habitat. In addition to seeking advice from the permitting authority, land appliers can contact the U.S. Department of Interior's Fish and Wildlife Service (FWS), which publishes an annual list of endangered and threatened species [24,31].

1.2.3.2 Application to flooded, frozen, or snow-covered land. Application of biosolids to flooded, frozen, or snow-covered land is not prohibited by the 40 CFR Part 503 rule. However, biosolids applied to such land must not enter surface waters or wetlands unless specifically authorized by a permit issued under Sections 402 or 404 of the CWA. Some common runoff controls at biosolids land-application sites include slope restrictions, buffer zones/filter strips, berms, dikes, silt fences, diversions, siltation basins, and terraces [24,31].

TABLE 1.5 40 CFR Part 503 Management Practices for Land-Applied Biosolids*

Management practice	Reason included in the 40 CFR Part 503 rule
Protection of threatened or endangered species	Consistency with federal regulation (50 CFR Parts 17.11 and 17.12)
Restriction of biosolids land application on flooded, frozen, or snow-covered ground	Prevents biosolids from entering surface waters and wetlands
Ten-meter (33-ft) buffer from U.S. waters	Protects waters of the United States: helps ensure risk is no greater than that calculated in the biosolids risk assessment, which assumed a 10-m buffer zone from surface waters
Agronomic application rate limit	Protects groundwater from nitrate contamination
Labeling requirements for bagged, containerized biosolids	Helps ensure that appliers use proper application rates, which ensure that pollutant limits are met

*Adapted from refs. [24,31].

1.2.3.3 Buffer zone—distance to surface waters. Bulk biosolids may not be applied within 10 m (i.e., 33 ft) of any surface waters (e.g., intermittently flowing streams, creeks, rivers, wetlands, or lakes) of the United States unless otherwise specified by the permitting authority [24,31]. Permitting authorities can allow exceptions to this requirement if the application of biosolids is expected to enhance the local environment. For example, biosolids application may be used to revegetate a stream bank or otherwise assist in minimizing bank erosion.

1.2.3.4 Agronomic rates. The *agronomic rate* is a biosolids land application rate that will result in the application of nitrogen that just meets crop or vegetative requirements, thus minimizing the amount of nitrogen that will pass below the root zone of the crop or vegetation to the groundwater. The 40 CFR Part 503 rule requires that the rate of application of bulk biosolids be equal to or less than the agronomic rate except in the case of a reclamation site, where a different rate of application may be allowed by the permitting authority. Although the biosolids preparer is required to supply the biosolids land applier with information on the nitrogen content of the biosolids, the land applier is responsible for determining that the biosolids are applied at a rate that does not exceed the agronomic rate for that site. Procedures for the design of the agronomic rate differ depending on such factors as the total and available nitrogen content, nitrogen losses, nitrogen from sources other than biosolids, and the nutrient requirements for the expected crop yield. Moreover, in some cases, phosphorus rather than nitrogen is used for determination of the agronomic rate. For details on the procedures for estimating the agronomic rate, see Chap. 7.

1.2.3.5 Labeling requirements for bagged or containerized biosolids. Bagged or containerized biosolids sold or given away must be applied at a rate equal to or less than the APLR limit (see Table 1.2). To ensure that biosolids are applied at a rate that does not exceed the APLR limit, a label or information sheet must be affixed to the bag or container. At a minimum, the label or information sheet must contain the following information:

- The name and address of the person who prepared the biosolids
- A statement that prohibits application of the biosolids to the land except in accordance with the instructions on the label or information sheet
- A maximum AWSAR that does not cause the APLR to be exceeded
- Biosolids nitrogen content

It should be noted that it is the responsibility of the preparer of biosolids to calculate the AWSAR for biosolids (see Example 1.1).

1.2.4 Pathogen levels in biosolids

In addition to meeting pollutant limits and management practices, land-applied biosolids must meet either the Class A or Class B pathogen-reduction criteria. The pathogen-reduction criteria for both classes of biosolids are given in Table 1.6. These criteria use a combination of technological and microbiological approaches to ensure adequate protection of human health and the environment from

TABLE 1.6 Maximum Concentrations of Pathogens Permitted in Biosolids*

Class A Biosolids	
Salmonella sp.	Less than 3 MPN† per 4 g total solids (or less than $1 \cdot 10^3$ MPN† fecal coliforms per gram total solids)
Enteric viruses	Less than 1 MPN† per 4 g total solids
Viable helminth ova	Less than 1 MPN† per 4 g total solids
Class B Biosolids	
Fecal coliforms	Less than $2 \cdot 10^6$ colony-forming units (CFUs) per gram total solids

NOTE: These requirements must be met when the biosolids are used or disposed.
*Adapted from ref. [34].
†MPN, most probable number.

pathogens. The objective of the Class A criteria is to reduce the pathogens in biosolids to below detectable levels, whereas Class B criteria ensure that pathogen concentrations have been reduced to levels that are unlikely to pose a threat to public health and the environment. Both Class A and Class B pathogen-reduction criteria are discussed in greater detail in the following sections.

1.2.4.1 Class A biosolids. All biosolids applied to lawns or home gardens and all biosolids sold or given away in bags or other containers must meet Class A pathogen-reduction criteria (see Table 1.6). Biosolids that meet this more stringent level of pathogen control are not subject to any harvesting or public access restrictions.

To meet Class A pathogen-reduction criteria (Table 1.6), a POTW can choose one of six alternatives. These alternatives include (1) use of a time/temperature-based process employed to treat the biosolids while meeting the pathogen limit in biosolids based on an indicator organism (fecal coliforms) or *Salmonella* sp. (see Table 1.6), (2) use of an alkali/air-drying stabilization process while also meeting the pathogen-based limit (see Table 1.6), (3) demonstration that the performance of a process for reducing enteric viruses and helminth ova meets the bacteria-based pathogen limit (see Table 1.6), (4) testing for pathogens (i.e., fecal coliform bacteria, enteric viruses, and helminth ova) at the time biosolids are used or disposed, (5) biosolids treatment in a process to further reduce pathogens (PFRP), or (6) use a process deemed equivalent to PFRP by the permitting authority. Each of these alternatives is summarized in Table 1.7.

TABLE 1.7 Alternatives Used to Meet Class A Biosolids Pathogen-Reduction Criteria

Alternative 1:	Thermally treated biosolids
Alternative 2:	Biosolids treated in high pH–high-temperature process
Alternative 3:	Biosolids treated in other processes*
Alternative 4:	Biosolids treated in unknown processes†
Alternative 5:	Use of processes to further reduce pathogens (PFRPs)‡
Alternative 6:	Use of processes equivalent to PFRP§

*This requirement relies on comprehensive monitoring of bacteria, enteric viruses, and viable helminth ova as well as sludge treatment operating conditions. It is assumed that the treatment process is meeting Class A criteria as long as it is operating under the same conditions that successfully reduced the pathogen densities.

†This requirement is similar to alternative 3, except that there is no option to substitute monitoring of effective operating parameters for microbiological monitoring.

‡PFRPs include (1) composting, (2) heat drying, (3) heat treatment, (4) thermophilic aerobic digestion, (5) beta-ray irradiation, (6) gamma-ray irradiation, and (7) pasteurization.

§Any process that can be demonstrated, through microbiological monitoring, to reduce *Salmonella* sp., enteric viruses, and viable helminth ova to below detectable levels may be used for PFRP equivalency. This is normally conducted on a site-specific basis by the regulatory authority.

If a POTW chooses the first alternative, the generated biosolids must meet pathogen-reduction criteria (see Table 1.6) using either the fecal coliforms or *Salmonella* sp. as indicator organisms. The biosolids must be shown to contain fewer than $1 \cdot 10^3$ most probable number (MPN) fecal coliforms per gram of total dry solids or less than 3 MPN *Salmonella* sp. per 4 g of total dry solids at the time the biosolids are used, disposed, or prepared for use or disposal. Additionally, the temperature must be maintained at a specified level and for a period of time based on the type of heating process employed.

For each of the four types of heating regimes described in the 40 CFR Part 503 rule, empirical equations are used to determine the minimum length of time that biosolids must be subjected to a given temperature to achieve Class A pathogen reduction (Table 1.8). The empirical equations take into consideration (1) the solid-liquid characteristics of the biosolids, (2) particle size, and (3) the mechanism by which particles are brought into contact with the heat. In addition, the time-temperature equations account for the fact that the internal structure of the mixture can inhibit mixing. For example, since less information is available about the operational parameters that could influence pathogen destruction for heating regime C, a safety factor is incorporated in the time-temperature equation used for treating biosolids under these conditions. Use of the equations in Table 1.8 is illustrated in Examples 1.3 and 1.4.

Example 1.3 The Poole County Water Reclamation Facility has installed a biosolids drier that will be operated at 65°C (149°F). If centrifuged biosolids having a solids content of 12 percent are to be discharged to the drier, estimate the minimum processing time necessary to achieve class A pathogen-reduction if heating regime A is followed.

solution Use the time-temperature equation that describes regime A (see Table 1.8) to estimate the minimum processing time.

$$D = \frac{131{,}700{,}000}{10^{0.14 \cdot T}} = \frac{131{,}700{,}000}{10^{0.14 \cdot (65)}}$$

$$= 0.105 \text{ day, or } 2.51 \text{ hours } (151 \text{ minutes})$$

Example 1.4 The Poole County Water Reclamation Facility (see Example 1.3) has determined that in order to minimize expenditures on biosolids storage facilities, it needs to have a maximum biosolids processing time in the drier of 15 minutes (0.0104 day). To ensure that Class A biosolids can still be generated by the system, what is the minimum temperature to which biosolids must be subjected for the 15-minute period if heating regime A is followed?

TABLE 1.8 Time-Temperature Requirements for Meeting Class A Biosolids Pathogen-Reduction Requirements—Alternative 1*

Regime	Applies to	Requirement	Time-temperature equation§
A	Biosolids with 7% solids or greater	Temperature of biosolids must be 50°C (122°F) or higher for 20 min or longer	$D = \frac{131{,}700{,}000}{10^{0.14 \cdot T}}$
B	Biosolids with 7% solids or greater in the form of small particles and heated by contact with either warmed gases or an immiscible liquid	Temperature of biosolids must be 50°C (122°F) or higher for 50 s or longer	$D = \frac{131{,}700{,}000}{10^{0.14 \cdot T}}$
C	Biosolids with less than 7% solids	Biosolids must be heated for at least 15 s but less than 30 min	$D = \frac{131{,}700{,}000}{10^{0.14 \cdot T}}$
D	Biosolids with less than 7% solids	Temperature of biosolids is 50°C (122°F) or higher with at least 30 min or longer contact time	$D = \frac{50{,}070{,}000}{10^{0.14 \cdot T}}$

*Adapted from ref. [34].
†D = time in days; T = temperature in degrees Celsius.

solution Use the time-temperature equation that describes regime A (see Table 1.8) to estimate the minimum processing temperature.

$$D = \frac{131{,}700{,}000}{10^{0.14 \cdot T}}$$

$$0.0104 = \frac{131{,}700{,}000}{10^{0.14 \cdot T}}$$

$$T = 72.2°\mathrm{C}$$

When the second alternative is chosen to achieve Class A biosolids, the generated biosolids must meet the same pathogen concentration limits as alternative 1 (see Table 1.6). In addition, the POTW must raise the pH of the biosolids to 12 for 72 hours while the temperature remains above 52°C (125.6°F) for at least 12 hours. At the end of the 72-hour period, the biosolids must be air-dried to 50 percent solids [24].

If alternative 3 is chosen to achieve Class A biosolids, the generated biosolids must meet the same pathogen-reduction criteria as alternative 1 (see Table 1.6). In addition to meeting the pathogen-reduction criteria, the effectiveness of a Class A process must be demonstrated.

Demonstration of a Class A process requires that the biosolids be analyzed for enteric viruses and viable helminth ova before pathogen-reduction treatment. If the concentration of enteric viruses in untreated biosolids does not exceed 1 plaque-forming unit (PFU) per 4 g of total dry solids, and when viable helminth ova do not exceed one per 4 g of total dry solids after pathogen-reduction treatment, the biosolids are considered Class A until the next monitoring episode. When one or the other limit is exceeded before biosolids processing, but the final pathogen criteria are met after processing, then the process parameters and their operational conditions used to achieve sufficient pathogen reduction must be documented. Future biosolids exiting the treatment process are then considered to be Class A if the documented operational conditions are met during biosolids processing.

When the POTW chooses alternative 4 (i.e., treatment in unknown processes) to achieve Class A biosolids, the biosolids must be tested at every monitoring episode (i.e., when a batch of biosolids is recycled or disposed or when the batch is being prepared for sale/give away) for the concentration of either fecal coliform or *Salmonella* sp. as well as enteric viruses and viable helminth ova. If the fecal coliform concentration is less than $1 \cdot 10^3$ MPN per gram of total solids or *Salmonella* sp. are less than 3 MPN per 4 g of total dry solids and enteric viruses and viable helminth ova are present at less than 1 PFU and one viable helminth ovum per 4 g of total dry solids, respectively, then the biosolids meet Class A pathogen-reduction criteria.

Alternatives 5 and 6 pertain to the use of processes to further reduce pathogens (PFRPs). PFRPs are processes operated at specific conditions deemed by the USEPA to effectively achieve pathogen levels that satisfy the Class A biosolids pathogen-reduction criteria (Table 1.9). In addition to PFRPs, any process that can be demonstrated, through microbiological monitoring, to reduce *Salmonella* sp., enteric viruses, and viable helminth ova to below detectable levels may be employed as a PFRP-equivalent process. Since employing a PFRP (or PFRP equivalent) eliminates the requirement for microbial monitoring, most POTWs will choose either of these two approaches to meet Class A pathogen-reduction criteria.

PFRP equivalency determinations can be made both on a site-specific and a national basis. A site-specific PFRP equivalency determination only pertains to one particular operation at one location under specific conditions. Under these circumstances, PFRP equivalency cannot be assumed to apply to the same process performed at a different location or for any modification of the process. A biosolids treatment process that is able to consistently produce the required pathogen reduction at different locations across the country may qualify for national PFRP equivalency. While the biosolids permitting authority normally conducts the evaluation of site-specific PFRP

TABLE 1.9 Processes to Further Reduce Pathogens (PFRPs)*

Process	Description
Composting	Using either the mechanical in-vessel or static aerated pile composting methods, the temperature of the biosolids is maintained at 55°C (131°F) or higher for 3 days. Using the windrow composting method, the temperature of the biosolids is maintained at 55°C (131°F) or higher for 15 days or longer. During the period when the compost is maintained at 55°C (131°F) or higher, there shall be a minimum of five turnings of the windrow.
Heat drying	Biosolids are dried by direct or indirect contact with hot gases to reduce the moisture content of the biosolids to 10% or lower. Either the temperature of the biosolids particles exceeds 80°C or the wet bulb temperature of the gas in contact with the biosolids as they leave the dryer exceeds 80°C (176°F).
Heat treatment	Liquid biosolids are heated to a temperature of 180°C (356°F) or higher for 30 minutes.
Thermophilic aerobic digestion	Liquid biosolids are agitated with air or oxygen to maintain aerobic conditions, and the mean cell residence time of the biosolids is 10 days at 55 to 60°C (131 to 140°F).
Beta-ray irradiation	Biosolids are irradiated with beta rays from an accelerator at dosages of at least 1.0 Mrad† at room temperature (ca. 20°C, 68°F).
Gamma-ray irradiation	Biosolids are irradiated with gamma rays from certain isotopes, such as cobalt-60 and cesium-137, at room temperature (ca. 20°C, 68°F).
Pasteurization	The temperature of the biosolids is maintained at 70°C (158°F) or higher for 30 minutes or longer.

*Adapted from refs. [24,34].

†Mrad = megarad (10^8 ergs per gram).

equivalency, the USEPA's Pathogen Equivalency Committee is consulted for national equivalency recommendations.

1.2.4.2 Class B biosolids. The objective of meeting Class B biosolids pathogen-reduction criteria is to ensure that the concentration of pathogenic microorganisms in biosolids is reduced sufficiently to protect human health and the environment. At a minimum, Class B pathogen-reduction criteria must be met for all land-applied biosolids. To be categorized as Class B biosolids, the fecal coliform concentrations in biosolids may not be greater than $2 \cdot 10^6$ MPN or $2 \cdot 10^6$ colony-forming units (CFUs) per gram (dry weight) of total biosolids. To achieve Class B pathogen-reduction criteria, a POTW may choose one of three alternatives. The first alternative consists of determining the concentration of fecal coliforms during each monitoring period using a minimum of seven samples taken over a 2-week period prior to biosolids being applied to land or disposed [34]. The samples must demonstrate that the geometric mean concentration of fecal coliforms is less than either $2 \cdot 10^6$ MPN per gram of total solids (dry weight) or $2 \cdot 10^6$ CFUs per gram of total solids. For the second alternative, a POTW may use a process to significantly reduce pathogens (PSRP). PSRPs are processes operated at specific conditions deemed by the USEPA to effectively achieve pathogen levels that satisfy the Class B biosolids criteria (Table 1.10).

The third alternative allows the POTW to employ a process determined by the permitting authority to be equivalent to a PSRP. The approaches available to POTWs to meet Class B biosolids pathogen-reduction criteria are summarized in Table 1.11.

TABLE 1.10 Processes to Significantly Reduce Pathogens (PSRPs)*

Process	Description
Aerobic digestion	Biosolids are agitated with air or oxygen to maintain aerobic conditions for a specific mean cell residence time at a specific temperature. Values for the mean cell residence time and temperature shall be between 40 days at 20°C (68°F) and 60 days at 15°C (59°F).
Air drying	Biosolids are dried on sand beds or paved or unpaved basins. The biosolids dry for a minimum of 3 months. During 2 of the 3 months, the ambient average daily temperature is above 0°C (32°F).
Anaerobic digestion	Biosolids are treated in the absence of air for a specific mean cell residence time at a specific temperature. Values for mean cell residence time and temperature shall be between 15 days at 35°C (95°F) to 55°C (131°F) and 60 days at 20°C (68°F).
Composting	Using either the mechanical in-vessel, static aerated pile or windrow composting methods, the temperature of the biosolids is raised to 40°C or higher and remains at 40°C (104°F) or higher for 5 days. For 4 hours during the 5 days, the temperature in the compost pile exceeds 55°C (131°F).
Lime stabilization	A sufficient amount of lime is added to the biosolids to raise their pH to 12 after 2 hours of contact.

*Adapted from refs. [24,34].

TABLE 1.11 Alternatives Available to Meet Class B Biosolids Pathogen-Reduction Criteria

Alternative 1: Monitoring of fecal coliforms*
Alternative 2: Use one of the processes to significantly reduce pathogens (PSRPs)†
Alternative 3: Use one of the processes equivalent to PSRP‡

*This alternative requires that the geometric mean of seven samples of treated biosolids be less than 2 million CFUs or MPN per gram of biosolids prior to land application or disposal.

†PSRPs include (1) aerobic digestion, (2) air drying, (3) anaerobic digestion, (4) composting, and (5) lime stabilization.

‡Equivalency is granted by the permitting agency on a site-specific basis.

TABLE 1.12 Site Restrictions for Land Application of Class B Biosolids*

1. Food crops with harvested parts that touch the biosolids-soil mixture and are totally above the land surface shall not be harvested for at least 14 months after application of biosolids.
2. Food crops with harvested parts below the land surface shall not be harvested for at least 20 months after application of biosolids when the biosolids remain on the land surface for 4 months or longer prior to incorporation into the soil.
3. Food crops with harvested parts below the land surface shall not be harvested for at least 38 months after application of biosolids when the biosolids remain on the land surface for less than 4 months prior to incorporation into the soil.
4. Food crops, feed crops, and fiber crops shall not be harvested for at least 30 days after application of biosolids.
5. Animals shall not be allowed to graze on the land for 30 days after application of biosolids.
6. Turf grown on land where biosolids have been applied shall not be harvested for at least 1 year after application when the harvested turf is placed on either land with a high potential for public exposure or a lawn, unless otherwise specified by the permitting authority.
7. Public access to land with a high potential for public exposure shall be restricted for 1 year after application of the biosolids. Public access to land with a low potential for public exposure shall be restricted for 30 days after application of biosolids.

*Adapted from ref. [31].

It is important to note that viable helminth ova are not necessarily reduced in Class B biosolids. Since Class B biosolids may contain significant levels of pathogens, site restrictions that limit crop harvesting, animal grazing, and public access must be enforced when a Class B biosolid is applied to land. The site restrictions associated with the land application of Class B biosolids are described in Table 1.12.

1.2.5 Vector attraction reduction requirements

The presence of pathogens in land-applied biosolids does not necessarily result in producing adverse effects to human health and the environment. Pathogens pose a disease risk to humans and animals only if there exist transport routes by which they may come into contact with a target species. A principal transport route for pathogens is vector transmission. Important vectors that facilitate transmission of pathogens from biosolids include insects, rodents, and birds [31]. Because of the importance of vectors in disease transmission, vector attraction reduction is a critical component of the biosolids land-application regulation. The 40 CFR Part 503 rule contains 11 options available for POTWs to meet the vector attraction reduction requirements for land-applied biosolids (Table 1.13).

The technical options described in Table 1.13 reduce vector attraction to biosolids by either decreasing the attractiveness of biosolids to vectors or by preventing the vectors from coming into contact with the biosolids (i.e., physical barriers). It should be noted that the first eight vector attraction reduction options are considered adequate for meeting the high-quality vector attraction reduction requirement that exempts Class A biosolids from the general requirements and management practices (i.e., exception-quality biosolids; see Sec. 1.2.9.1). Class A biosolids that meet the vector attraction reduction requirements using one of the last three options remain subject to the general requirements and management practices. Figure 1.7 depicts the use of biosolids injection (option 9) to meet the vector attraction reduction requirements.

1.2.6 Monitoring frequency

Biosolids applied to agricultural land, a forest, a public contact site, or a reclamation site must be monitored for pollutant concentrations, pathogens, and vector attraction reduction. The minimum monitoring frequency for bulk as well as bagged biosolids can be 1, 4, 6, or 12 times per year depending on the annual amount of biosolids that are applied.

POTWs or others applying 0 to less than 290 dry metric tons (dmt) of biosolids (i.e., 320 dry U.S. tons) annually must test for heavy metals, pathogens, and vector attraction reduction efficiency once per year. Those facilities applying 290 to less than 1500 dmt of biosolids annually (320 to 1650 dry U.S. tons) must test for heavy metals, pathogens, and vector attraction reduction efficiency once per quarter. Those facilities applying 1500 to less than 15,000 dmt of biosolids annually (1650 to 16,500 dry U.S. tons) must test once every 60 days (six times per year), and those facilities land applying or receiving 15,000 dmt (16,500 dry U.S. tons) or more must test once a month unless otherwise specified by the permitting authority [24,34]. The biosolids monitoring frequencies are summarized in Table 1.14.

1.2.7 Recordkeeping

The 40 CFR Part 503 rule requires that both the preparer and land applier of biosolids keep certain records. Recordkeeping require-

TABLE 1.13 Options for Meeting Vector Attraction Reduction for Land Application of Biosolids*

1. *Reduction of volatile solids content.* Reduction of vector attraction is achieved if the mass of volatile solids in the biosolids is reduced by at least 38% during treatment. The volatile solids reduction can include any additional volatile solids reduction that occurs before the biosolids leave the treatment works, which is anticipated when the material is processed in drying beds or is composted.
2. *Additional digestion of anaerobically digested biosolids.* Biosolids are considered to have achieved satisfactory vector attraction reduction if they lose less than 17% additional volatile solids when they are anaerobically digested in a bench scale reactor at 30 to 37°C (86 to 98.6°F) for an additional 40 days.
3. *Additional digestion of aerobically digested biosolids.* Biosolids with a 2% or less solids content are considered to have achieved satisfactory vector attraction reduction if they lose less than 15% additional volatile solids when they are aerobically digested in the laboratory in a bench scale unit at 20°C (68°F) or higher for an additional 30 days.
4. *Specific oxygen uptake rate for aerobically digested biosolids.* Biosolids are considered to have achieved sufficient reduction in vector attraction if it can be demonstrated that the specific oxygen uptake rate is equal to or less than 1.5 mg oxygen per hour per gram of biosolids at 20°C (68°F).
5. *Aerobic process at greater than 40°C (104°F).* Biosolids that are aerobically treated for 14 days or longer during which time the average temperature is higher than 45°C (113°F) are considered to meet the vector attraction reduction criteria.
6. *Addition of alkali.* Biosolids are considered to be adequately reduced in vector attraction if sufficient alkali is added to (1) raise the pH to at least 12, (2) maintain a pH of at least 12 without addition of more alkali for 2 hours, (3) maintain a pH of at least 11.5 without addition of more alkali for an additional 22 hours.
7. *Moisture reduction of biosolids containing no unstabilized solids.* Vector attraction is considered to be sufficiently reduced if the biosolids do not contain unstabilized solids generated during primary wastewater treatment and if the solids content of the biosolids is at least 75%.
8. *Moisture reduction of biosolids containing unstabilized solids.* Vector attraction of biosolids is considered to be adequately reduced if the solids content is increased to 90% or greater.
9. *Injection.* Adequate vector attraction reduction can be achieved by injecting the biosolids below the ground. No significant amount of biosolids may be present on the soil surface within 1 hour. If Class A biosolids were injected to achieve vector attraction reduction, the injection must occur within 8 hours after the biosolids are discharged from the pathogen-reduction treatment process.
10. *Incorporation of biosolids into the soil.* Biosolids applied to the land surface or placed on a surface disposal site must be incorporated into the soil within 6 hours after application to or placement on the land. If Class A biosolids are incorporated, they must be added to the soil within 8 hours after the biosolids are discharged from the pathogen reduction process.
11. *Covering biosolids.* Biosolids placed on a surface disposal site must be covered with soil or other material at the end of each operating day. Daily covering reduces vector attraction by creating a physical barrier.

*Adapted from refs. [24,34].

Figure 1.7 Soil injection of biosolids used to comply with the 40 CFR Part 503 vector attraction reduction requirements.

TABLE 1.14 Frequency of Monitoring for Heavy Metals, Pathogens, and Vector Attraction Reduction Efficiency for Land-Applied Biosolids*

Amount of biosolids (dry metric tons per year)†	Amount of biosolids (dry U.S. tons per year)‡	Monitoring frequency
0–290	0–320	Once per year
290–1500	320–1650	Once per quarter
1500–15,000	1650–16,500	Once per 60 days
>15,000	>16,500	Once per month

*Adapted from ref. [24].
†Dry metric ton = 1000 kg (1kg = 2.2 lb).
‡U.S. ton = 2000 lb.

ments vary depending on whether biosolids are applied in bulk or are dispensed from a bag or other container that was sold or given away. Recordkeeping requirements also vary depending on whether pollutant concentration limits are met, whether Class A or Class B pathogen requirements are met, and the type of vector attraction reduction procedures used. The person responsible for keeping certain records also varies. For example, those who prepare biosolids for land application must keep pollutant concentration data, whereas the biosolids land applier (if different from the preparer) must keep information pertaining to management practices. General recordkeeping responsibilities are summarized in Table 1.15.

Except as noted, records must be kept for at least 5 years. Records are to be submitted annually to the permitting authority for all facilities that have an approved wastewater pretreatment program under 40 CFR Part 403 (i.e., Class I solid waste management facilities), together with those facilities with a design flow rate of at least 1 million gallons per day [24,30,31]. In addition, annual reports are required when any cumulative metal loading reaches 90 percent of the allowed cumulative pollutant loading rate.

The most intensive recordkeeping is required for bulk biosolids that meet ceiling limits but not pollutant concentration limits. These biosolids are subject to general and management requirements, and both the preparer and supplier (if different) are required to keep records for at least 5 years and, in a few cases, indefinitely. Specific recordkeeping requirements for Class A and Class B biosolids are described in the following sections.

1.2.7.1 Recordkeeping for Class A biosolids. For Class A biosolids that do not meet pollutant concentration limits, the preparer must keep pollutant concentration data, management practice certifications (and descriptions), and pathogen and vector attraction reduction certifications (and descriptions). An example of a management practice certification is provided in Fig. 1.8.

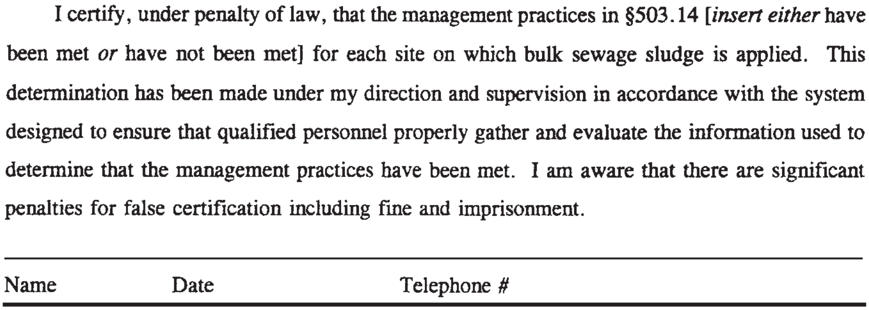
The biosolids land applier is required to maintain information on the land-application site location, number of hectares (or acres) in the site, data and time of application, cumulative amount of pollutant applied, total amount of biosolids applied in the application, and certifications and descriptions of information obtained from the biosolids preparer. At a minimum, this information must include (1) appropriate certifications and biosolids quality information from the preparer, (2) the nitrogen content of the biosolids (obtained from the preparer), and (3) information collected from the permitting authority on past applications of biosolids subject to cumulative limits. Information collected by the land applier is the only information that the 40 CFR Part 503 rule requires to be kept indefinitely.

For biosolids or materials derived from biosolids that meet pollutant concentration limits and Class A pathogen and vector attraction reduction requirements, the preparer must only maintain information on pollutant concentrations and pathogen and vector attraction reduction certifications and descriptions, unless the biosolids are injected or incorporated to meet vector attraction reduction requirements. If Class A biosolids are injected or incorporated into soil, then the land applier must certify and describe the vector attraction reduction and also must certify and describe management practice requirements (this type of

TABLE 1.15 Recordkeeping Responsibilities by Type of Biosolids*

Record	Type of biosolid					
	Bulk biosolids subject to cumulative limits		Biosolids meeting pollutant concentration limits		Material derived from bulk sewage sludge (bulk or bags)	Bagged sewage sludge not meeting pollutant concentration limits
	Class A	Class B	Class A	Class B		
Pollutant concentration	Preparer	Preparer	Preparer	Preparer	Preparer	Preparer
Management practice certification	Applier	Applier	None if not injected or incorporated	Applier	None	Preparer
Site-restriction certification and description	None	Applier	None	Applier	None	None
Vector attraction reduction certification and description	Preparer or applier	Preparer or applier	Preparer or applier	Preparer or applier	Preparer	Preparer
Pathogen-reduction certification and description	Preparer	Preparer	Preparer	Preparer	Preparer	Preparer

*Adapted from refs. [24,30,31].



I certify, under penalty of law, that the management practices in §503.14 [*insert either* have been met *or* have not been met] for each site on which bulk sewage sludge is applied. This determination has been made under my direction and supervision in accordance with the system designed to ensure that qualified personnel properly gather and evaluate the information used to determine that the management practices have been met. I am aware that there are significant penalties for false certification including fine and imprisonment.

Name	Date	Telephone #

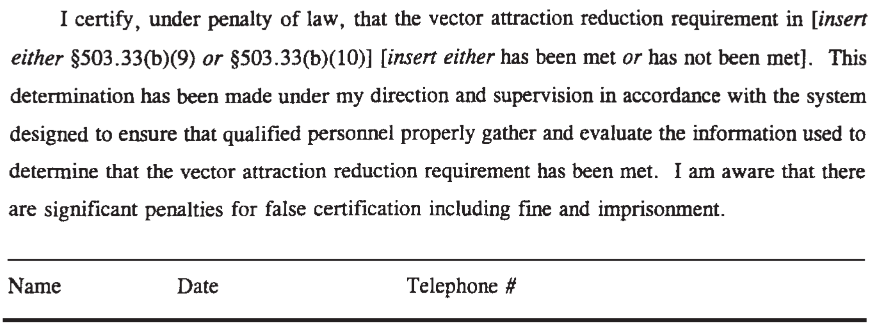
Figure 1.8 Example of certification for compliance with the management practice provisions of the 40 CFR Part 503 rule.

biosolids vector attraction reduction practice is not exempt from general requirements and management practices because this vector reduction procedure is not one of those that meets the high-quality requirement for vector attraction reduction). An illustration of the vector attraction reduction certification is provided in Fig. 1.9.

It should be noted that materials derived from biosolids (e.g., commercial fertilizers) that meet pollutant concentration limits and Class A pathogen and vector attraction reduction requirements are not subject to any Subpart B requirements (i.e., no recordkeeping or reporting requirements). POTWs that prepare biosolids in bags or other containers but whose biosolids do not meet pollutant concentration limits must include in their files (1) the AWSAR that does not cause the APLR to be exceeded, (2) pollutant concentrations and vector attraction reduction certifications (and descriptions), and (3) a certification that the labeling management practice has been met.

1.2.7.2 Recordkeeping for Class B biosolids. If biosolids meet only Class B pathogen and vector attraction reduction requirements, then the land applier must keep the management practice and site-restriction certifications and descriptions for 5 years. Examples of site-restriction certifications are provided in Fig. 1.10. If injection or incorporation of biosolids into soil is used as a vector attraction reduction method, the applier also must retain information certifying and describing the method.

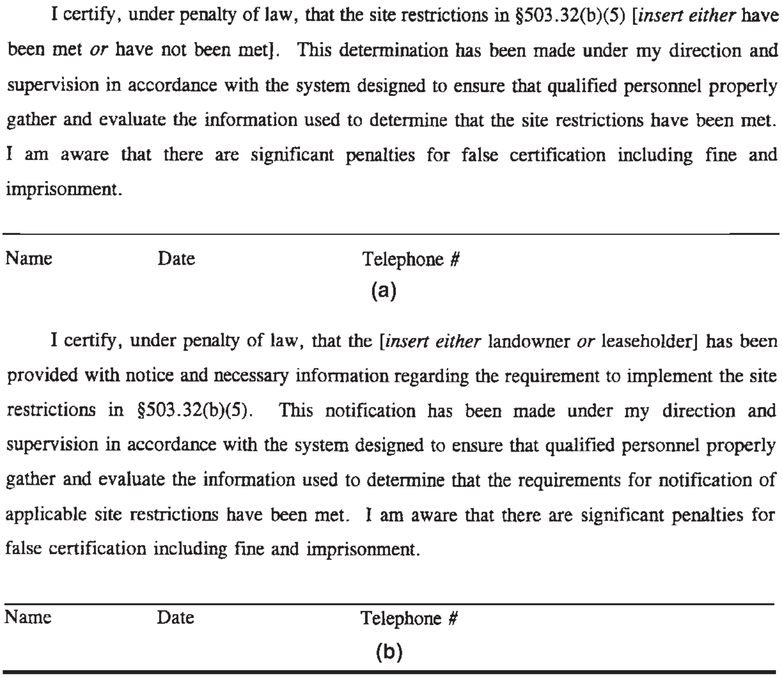
If the bulk biosolids meet Class B pathogen and vector requirements and pollutant concentration limits, the applier does not have to keep site location and application information. If the Class B biosolids were not injected or incorporated to meet vector attraction reduction, the preparer must only keep records of pollutant concentrations, Class B pathogen certification (and descriptions), and vector attraction reduction certifications (and descriptions). If injection or incorporation were used to meet the vector attraction reduction requirement, the applier



I certify, under penalty of law, that the vector attraction reduction requirement in [*insert either* §503.33(b)(9) *or* §503.33(b)(10)] [*insert either* has been met *or* has not been met]. This determination has been made under my direction and supervision in accordance with the system designed to ensure that qualified personnel properly gather and evaluate the information used to determine that the vector attraction reduction requirement has been met. I am aware that there are significant penalties for false certification including fine and imprisonment.

Name Date Telephone #

Figure 1.9 Example of certification statement verifying compliance with the vector attraction reduction provisions of the 40 CFR Part 503 rule.



I certify, under penalty of law, that the site restrictions in §503.32(b)(5) [*insert either* have been met *or* have not been met]. This determination has been made under my direction and supervision in accordance with the system designed to ensure that qualified personnel properly gather and evaluate the information used to determine that the site restrictions have been met. I am aware that there are significant penalties for false certification including fine and imprisonment.

Name Date Telephone #

(a)

I certify, under penalty of law, that the [*insert either* landowner *or* leaseholder] has been provided with notice and necessary information regarding the requirement to implement the site restrictions in §503.32(b)(5). This notification has been made under my direction and supervision in accordance with the system designed to ensure that qualified personnel properly gather and evaluate the information used to determine that the requirements for notification of applicable site restrictions have been met. I am aware that there are significant penalties for false certification including fine and imprisonment.

Name Date Telephone #

(b)

Figure 1.10 Example of certification for compliance with the site-restriction provisions of the 40 CFR Part 503 rule: (*a*) typical site-restriction certification used when the biosolids land applier is different from the biosolids generator; (*b*) typical certification employed when the biosolids land applier and generator are the same.

must certify and describe management practices, site restrictions, and vector attraction reduction.

1.2.8 Reporting requirements

Once per year, certain POTWs must report information contained in their records to the permitting agency. Any information required to be maintained only by land appliers is not required to be reported, except when biosolids subject to cumulative limits are applied on a site that is within 90 percent of the cumulative pollutant loading rate limit. In this case, all the site-specific information [e.g., site location, number of hectares (or acres), pollutant loadings, and quantity of biosolids applied] must be obtained from the biosolids land applier and submitted to the permitting agency.

POTWs that must submit reports to the permitting agency include Class I facilities and POTWs with an influent wastewater flow rate equal to or greater than 1 million gallons per day (MGD) or that serve a population of 10,000 or more. It should be noted that POTWs whose industrial dischargers are required to pretreat (typically those processing 5 MGD or more of wastewater) are considered Class I facilities. Regardless of the POTW's flow rate, any POTW may be designated as Class I by the permitting authority based on the potential of its biosolids use or disposal practice to adversely affect public health or the environment [30].

1.2.9 Summary of options for complying with biosolids land-application criteria

The three principal characteristics that determine the suitability of biosolids to be applied to land, i.e., pollutant limits, pathogen levels, and vector attraction reduction, have been combined to yield four biosolids land-application options that meet the compliance requirements of the 40 CFR Part 503 rule. The four options include

1. Exceptional-quality option (EQ)
2. Pollutant concentration option (PC)
3. Cumulative pollutant loading rate option (CPLR)
4. Annual pollutant loading rate option (APLR)

Each of the biosolids land-application compliance options is discussed in the following sections. It is important to recognize that each option is assumed to be equally protective of public health and the environment.

1.2.9.1 Exceptional-quality (EQ) biosolids. For biosolids to qualify under the exceptional-quality (i.e., EQ) option, the following requirements must be met:

- The ceiling concentration (mg/kg) for pollutants may not be exceeded.
- The pollutant concentration limits (mg/kg) may not be exceeded.
- One of the Class A pathogen-reduction alternatives must be met (see Table 1.5).
- One of the first eight vector attraction reduction options must be employed (see Table 1.13).

Once biosolids meet EQ requirements, they are not subject to the land-application general requirements or management practices described in the 40 CFR Part 503 rule. However, the USEPA regional administrator or the state biosolids director may determine that such requirements are necessary to protect public health and the environment. Under most circumstances, EQ biosolids may be applied as freely as any other fertilizer or soil amendment to any type of land. EQ biosolids may be sold or given away in bulk form, bags, or other containers.

1.2.9.2 Pollutant concentration (PC) biosolids. For biosolids to qualify for land application under the pollutant concentration (PC) compliance option, they must meet the following requirements:

- Ceiling concentration for pollutants may not be exceeded.
- Pollutant concentration limits may not be exceeded.
- One of the Class B pathogen-reduction alternatives must be employed together with the appropriate site restrictions and management practices (see Table 1.12).
- One of the 11 vector attraction reduction options must be employed (see Table 1.13).
- Monitoring, recordkeeping, and reporting requirements must be followed.

It should be noted that PC biosolids can be applied to all types of land except public-access sites (e.g., lawn and home gardens). Moreover, PC biosolids may only be sold or given away in bulk quantities (i.e., they may not be distributed in bags or other containers).

1.2.9.3 Cumulative pollutant loading rate (CPLR) biosolids. The CPLR is the maximum amount of regulated heavy metals that can be applied to a site. For biosolids to qualify for land application as CPLR biosolids, the following requirements must be met:

- Ceiling concentrations (mg/kg) for pollutants may not be exceeded.
- Cumulative pollutant loading rates (kg/ha or lb/acre) may not be exceeded.
- Either the Class A or Class B pathogen-reduction criteria must be met.
- One of the 11 vector attraction reduction options must be employed (see Table 1.13).
- Monitoring, recordkeeping, and reporting requirements must be met.
- Applicable site restrictions and management practices must be followed.

When the CPLR for any regulated pollutant is reached at a site, no additional biosolids may be applied. CPLR biosolids may only be sold or given away in bulk form and not in bags or other containers.

1.2.9.4 Annual pollutant loading rate (APLR) biosolids. An APLR is the maximum amount of regulated heavy metals that can be applied to a site in any 1 year through land application of biosolids. The APLR compliance option is only available to biosolids that are sold or given away in bags or other containers for land application. For biosolids to qualify for land application under the APLR option, the following requirements apply:

- Ceiling concentrations (mg/kg) for pollutants may not be exceeded.
- Annual pollutant loading rates (kg/ha or lb/acre) may not be exceeded.
- One of the Class A pathogen-reduction alternatives must be employed (see Table 1.7).
- One of the first eight vector attraction reduction options must be employed (see Table 1.13).
- Monitoring, recordkeeping, and reporting requirements must be met.
- Applicable site restrictions and management practices must be followed.

A summary of the four land-application options that comply with the legal requirements of the 40 CFR Part 503 rule is given in Table 1.16. The recordkeeping and reporting requirements for each option are summarized in Table 1.17.

1.2.10 Domestic septage

Domestic septage is defined in the 40 CFR Part 503 rule as the liquid or solid material removed from a septic tank, portable toilet, type III

TABLE 1.16 Summary of Options That Comply with the Biosolids Land-Application Regulations in 40 CFR Part 503*

Option†	Pollutant limits	Pathogen requirements	Vector attraction reduction requirements
EQ (exceptional-quality) biosolids	Bulk or bagged biosolids meet pollution concentration limits	Any one of the Class A requirements	Any one of the vector attraction reduction requirements
Pollutant concentration biosolids	Bulk biosolids meet pollutant concentration limits	Any one of the Class A or Class B requirements	For Class A biosolids, vector attraction reduction requirements 9 or 10. For Class B biosolids, any one of the 11 vector attraction reduction requirements
Cumulative pollutant loading rate	Bulk biosolids subject to cumulative pollutant loading rate limits	Any one of the Class A or Class B requirements	Any one of the 11 vector attraction reduction requirements
Annual pollutant loading rate	Bagged biosolids subject to annual pollutant loading rate limits	Any one of the Class A requirements	Any one of the first 8 vector attraction reduction requirements

*Adapted from ref. [24].

†Each of these options also requires that the biosolids meet the ceiling concentration limits for heavy metal pollutants and that the frequency of monitoring, recordkeeping, and reporting requirements be met.

marine sanitation device, or a similar system that receives only domestic septage (i.e., household, noncommercial, nonindustrial sewage). While a septic tank is normally used by households, a type III marine sanitation device is the name given to a holding tank for receiving wastes from a boat or other water-going vessel.

The term *septage* has been used to refer to many materials pumped out of various types of waste-receiving tanks. Although septage contains many different substances depending on the type of waste being treated in the septic system, domestic septage contains mostly water, sewage, grit, organic fecal matter, and small amounts of polluting substances. The most common nutrients contained in domestic septage are nitrogen and phosphorus. These nutrients, together with certain trace elements and organic matter, make domestic septage valuable for use as a fertilizer and/or soil conditioner on agricultural land, forests, and reclamation sites. Typical physical and chemical properties of domestic septage relative to sewage sludge are provided in Table 1.18.

TABLE 1.17 Recordkeeping and Reporting Requirements for Land-Applied Biosolids*

Type of biosolid	Records that must be kept	Person responsible for recordkeeping		Records that must be reported
		Preparer	Applier	
EQ† biosolids	Pollutant concentrations	X		X
	Pathogens-reduction certification and description	X		X
	Vector attraction reduction certification and description	X		X
PC† biosolids	Pollutant concentrations	X		X
	Management practice certification and description		X	
	Site restriction certification and description (Class B requirements)		X	
	Pathogen-reduction certification and description	X		X
	Vector attraction reduction certification and description	X	X‡	X
CPLR† biosolids	Pollutant concentrations	X		X
	Management practice certification and description		X	
	Site restriction certification and description (Class B biosolids)		X	
	Pathogen-reduction certification and description	X		X
	Vector attraction reduction certification and description	X	X‡	X
APLR† biosolids	Pollutant concentrations	X		X
	Management practice certification and description	X		X
	Pathogen-reduction certification and description	X		X
	Vector attraction reduction and description	X		X
	AWSAR§ for biosolids	X		X

*Adapted from ref. [24].

†EQ, exceptional-quality; PC, pollutant concentration; CPLR, cumulative pollutant loading rate; APLR, annual pollutant loading rate.

‡The preparer certifies and describes vector attraction reduction methods other than injection and incorporation of biosolids into the soil. These methods must be certified by the land applier.

§Annual whole sludge application rate (dmt/hectare-year or U.S. ton/acre-year).

TABLE 1.18 Physical and Chemical Characteristics of Domestic Septage Relative to Sewage Sludge*

	Concentration (mg/kg, dry-weight basis)	
Parameter	Domestic septage	Sewage sludge
Arsenic	4	10
Cadmium	3	7
Copper	140	740
Lead	35	130
Mercury	0.15	5
Nickel	15	43
Selenium	2	5
Zinc	290	1200
Nitrogen as N	2%	2–7%
Phosphorus as P	<1%	1–3%
pH	6–7	5–8
Grease	6–12%	5–10%
Biochemical oxygen demand (BOD_5)†	6480 mg/liter	>2000 mg/liter
Total solids	3.4%	3–35%

*Adapted from ref. [15].
†BOD_5 is highly variable in sewage sludge.

The specific definition of domestic septage in the 40 CFR Part 503 rule does not include many of the other materials that are often called septage by industry. For example, commercial and industrial septage are not considered domestic septage. The factor that differentiates commercial and industrial septage from domestic septage is not the type of establishment generating the waste; rather, it is the type of waste being produced. For example, even though restaurant grease trap wastes are not considered domestic septage, the sanitation waste residues and residues from food and normal dish cleaning from a restaurant are considered domestic septage. Similarly, sanitation waste residues from a gasoline station are domestic septage, whereas wastes containing petroleum are classified as nondomestic septage. Finally, it is critical to note that any mixing of domestic and nondomestic septage (e.g., the collection of both domestic and nondomestic septage in a pumper truck or holding tank) causes the entire batch of septage to be considered nondomestic septage. Nondomestic septage is not regulated under the 40 CFR Part 503 rule [20,26].

1.2.10.1 Use and disposal of domestic septage. In addition to being suitable for discharge in municipal wastewater or septage-only treatment facilities, domestic septage may be applied to nonpublic contact sites, applied to public contact sites, or disposed in a biosolids-only monofill or municipal landfill. If domestic septage is discharged into a treatment facility that receives only domestic septage, the treated septage can be applied to either public or nonpublic contact sites. *Public* contact sites are defined as lands with a high potential for contact by the public, such as public parks, ball fields, cemeteries, plant nurseries, turf farms, and golf courses. If treated septage is to be applied to a public contact site, the septage is considered a sewage sludge and must meet the more extensive requirements of the 40 CFR Part 503 rule, including (1) general requirements, (2) pollutant limits, (3) management practices, (4) operational standards covering pathogen-reduction criteria and vector attraction reduction requirements, (5) monitoring requirements, (6) recordkeeping, and (7) reporting requirements. If the treated septage is to be applied to a nonpublic contact site (e.g., agricultural land, forest, or reclamation site), the requirements governing land application are less burdensome (see Sec. 1.2.10.2).

If domestic septage is discharged into a sanitary sewer or directly into a POTW that also receives municipal wastewater, the person discharging the domestic septage must follow the rules of that POTW. The residual solids from the treatment of the sewage sludge and domestic septage would be covered by the specific provisions of 40 CFR Part 503 rule that apply to the biosolids use or disposal practice being followed. Alternatively, if domestic septage is placed in a sewage sludge-only landfill (called *surface disposal* in the 40 CFR Part 503 rule) or incinerated in a sewage sludge-only incinerator, its disposal is covered by the requirements in the 40 CFR Part 503 rule for those disposal practices.

Finally, if domestic septage is placed in a municipal solid waste landfill, its disposal is covered by the rules of the disposal facility, which, in turn, must comply with the requirements of 40 CFR Part 258 that specifies the criteria for the disposal of nonhazardous wastes [10,11].

1.2.10.2 Land application of domestic septage. To meet the federal requirements for land application of domestic septage to nonpublic contact sites, the land applier must ensure that he or she has only domestic septage. Only when domestic septage is applied to nonpublic contact sites are the less burdensome land-application requirements specified within the 40 CFR Part 503 rule applicable. Persons who apply domestic septage to nonpublic contact sites (agricultural land, forests, and reclamation sites) must comply with (1) limits on volumetric application rates based on the nitrogen demand of the crop and restrictions on crop harvesting, animal grazing, and site access and (2) provisions for control of pathogens and vector attraction reduction [20,26].

Domestic septage applied to nonpublic contact sites is not required to meet the specific ceiling, cumulative, or pollution concentration limits that are applicable to land-applied sewage sludge. When a domestic septage land applier chooses not to meet these limits, the annual domestic septage application rate (gallons per acre-year) depends on the amount of nitrogen required by the planned crop and the yield. The maximum annual domestic septage application rate may be estimated by Eq. (1.4). The factor 0.0026 in Eq. (1.3) was obtained by assuming that (1) the nitrogen content in septage is completely mineralized over a 3-year period, (2) domestic septage is approximately 2.5 percent solids, and (3) the nitrogen content in septage is approximately 350 mg/kg (dry-weight basis) [15,20,26].

$$\text{Annual application rate}\left(\frac{\text{gallons}}{\text{acre}\cdot\text{yr}}\right) = \frac{\text{lb of nitrogen required by the crop}\left(\dfrac{\text{lb N}}{\text{acre}\cdot\text{yr}}\right)}{0.0026} \qquad (1.4)$$

Although Eq. (1.4) was included in the 40 CFR Part 503 rule to facilitate the land application of a "typical" domestic septage, in some cases, the assumptions that are inherent in Eq. (1.4) do not apply. For example, in cases where the domestic septage has been dewatered prior to land application (e.g., which typically occurs during storage of domestic septage), the solids content will be significantly greater than the 2.5 percent assumed in developing Eq. (1.4). Under these circumstances, it is recommended that the dewatered septage be treated by the land applier as a POTW sewage sludge [15,20,26]. This approach would necessitate the measurement of the actual nitrogen content of the septage prior to land application. In other cases, the domestic septage may have a nitrogen content far in excess of the assumed valued of 350 mg/kg (e.g., septage discharged from a type III marine sanitation device). Although the 40 CFR Part 503 rules does not require it, good practice would dictate that the volume of domestic septage that is applied to land should be reduced from the value calculated using Eq. (1.4) if its nitrogen content is significantly greater than 350 mg/kg. In any event, under no circumstances can domestic septage be applied to land at rates in excess of those calculated by Eq. (1.4). Examples of domestic septage application rates for various crops using Eq. (1.4) are provided in Table 1.19.

1.2.10.3 Domestic septage pathogen-reduction requirements. Domestic septage that is to be applied to land on a nonpublic contact site must

TABLE 1.19 Typical Domestic Septage Rates*

Crop	Yield (bushel/acre-year)	Nitrogen requirement (lb N/acre-year)	Application rate† (gal/acre-year)
Corn	100	100	38,500
Oats	90	60	23,000
Barley	70	60	23,000
Grass/hay	4 tons/acre	200	77,000
Wheat	70	105	40,400
Soybeans	40	30	11,500
Cotton	1.5 bale/acre	90	35,000

*Adapted from refs. [15,20,26].
†Calculated using Eq. (1.4).

be managed so that pathogens are reduced. The 40 CFR Part 503 rule offers domestic septage land appliers two alternatives to meet this requirement. The first alternative uses crop, grazing, and site restrictions exclusively with no chemical treatment, while the second alternative requires raising the pH of the septage to 12 for 30 minutes through chemical addition.

For septage land appliers who choose the second alternative to meet pathogen-reduction criteria, the chemicals most commonly used to raise the pH of domestic septage are hydrated lime ($Ca(OH)_2$) and quicklime (CaO). Regardless of the chemical chosen, at a minimum, two separate representative samples of the chemically treated domestic septage must be taken 30 minutes apart to verify that the pH remains at a value of 12 or higher for that time period.

The lime can be added to the septage in dry form or as a slurry. In most cases, 20 to 40 lb of lime must be added per 1000 gallons of domestic septage. The exact amount will vary depending on the septage characteristics. If, after adding lime, the septage does not maintain a pH of at least 12, more lime must be added so that the septage will remain at a pH of 12 or higher for 30 minutes. The pH of the domestic septage sample can be evaluated using either a pH meter or pH-sensitive colored paper.

Although there is the added expense of chemical addition, the second alternative for meeting the pathogen-reduction requirement has an advantage in that domestic septage subject to alkaline treatment need only meet crop restriction requirements (i.e., there are neither grazing nor site restrictions). The requirements for both pathogen-reduction alternatives are summarized in Table 1.20.

It should be noted that some of the pathogen-reduction requirements outlined in Table 1.20 describe approaches that are unique for certain types of crops. The principal crop characteristic that affects the choice of pathogen-reduction approach is whether the edible part of the crop will touch the soil-septage mixture. Table 1.21 provides a list

TABLE 1.20 Land Application of Domestic Septage: Pathogen-Reduction Requirements*

Alternative I: Septage That Is Land Applied without Chemical Treatment

Crop restrictions

- Food crops with harvested parts that touch the septage-soil mixture and are totally above ground shall not be harvested for 14 months after application of domestic septage.
- Food crops with harvested parts below the soil surface of the land shall not be harvested for 38 months after application of domestic septage.
- Animal feed, fiber, and those food crops that do not touch the soil surface shall not be harvested for 30 days after application of the domestic septage.
- Turf grown on land where domestic septage is applied shall not be harvested for 1 year after application of the domestic septage when the harvested turf is placed on either a lawn or land with a high potential for public exposure, unless otherwise specified by the permitting authority.

Grazing restrictions

- Animals shall not be allowed to graze on the land for at least 30 days after application of domestic septage.

Site restrictions

- Public access to land with a low potential for public exposure shall be restricted for at least 30 days after application of domestic septage. Examples of restricted access include remoteness of site, posting with no trespassing signs, and/or fencing.

Alternative II: Chemically Treated Septage That Is Land Applied†

Crop restrictions

- Food crops with harvested parts that touch the septage-soil mixture and are totally above ground shall not be harvested for 14 months after application of domestic septage.
- Food crops with harvested parts below the soil surface of the land shall not be harvested for 20 months after application of domestic septage when the domestic septage remains on the land surface for 4 months or longer prior to incorporation into the soil.
- Food crops with harvested parts below the surface of the land shall not be harvested for 38 months after application of domestic septage when the domestic septage remains on the land surface for less than 4 months prior to incorporation into the soil.
- Animal feed, fiber, and those food crops that do not touch the soil surface shall not be harvested for at least 30 days after application of the domestic septage.
- Turf grown on land where domestic septage is applied shall not be harvested for 1 year after application of the domestic septage when the harvested turf is placed on either a lawn or land with a high potential for public exposure, unless otherwise specified by the permitting authority.

Grazing restrictions

- None

Site restrictions

- None

---

*Adapted from refs. [20,26,31].

†Domestic septage has had its pH raised to 12 or higher by addition of alkaline material and, without adding more alkaline materials, the domestic septage remains at a pH of 12 or higher for at least 30 minutes prior to land application.

TABLE 1.21 Crops Affected by Domestic Septage Pathogen Requirements*

Crops that do not touch the soil	Crops that do touch the soil	Crops that are below ground
Peaches	Melons	Potatoes
Apples	Eggplant	Yams
Corn	Squash	Sweet potatoes
Wheat	Tomatoes	Rutabaga
Oats	Cucumbers	Peanuts
Barley	Celery	Onions
Oranges	Strawberries	Leaks
Grapefruit	Cabbage	Radishes
Cotton	Lettuce	Turnips
Soybeans	Hay	Beets

*Adapted from ref. [15].

of various crops and distinguishes whether or not the edible parts are likely to touch the soil-septage mixture.

1.2.10.4 Vector attraction reduction approaches. If the septage pathogen-reduction requirements are met using alternative I, one of two approaches must be employed to meet vector attraction reduction. The first approach is to use subsurface injection of the septage, whereas the other approach requires incorporation of the septage into the soil (i.e., plowing or disking) within 6 hours of land application.

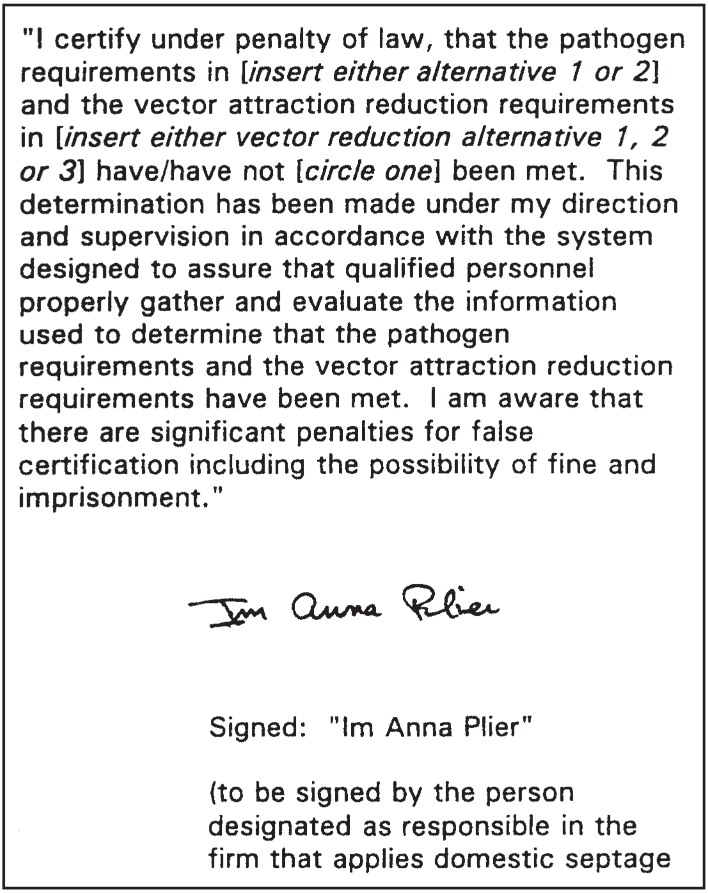
If the septage pathogen-reduction requirements are met using alternative II (i.e., alkaline treatment of septage), vector attraction reduction requirements are assumed to be met. A summary of the vector attraction reduction alternatives for land application of domestic septage is provided in Table 1.22.

If the pathogen and vector attraction reduction requirement for domestic septage is met using alkali addition, each container, i.e., truck

TABLE 1.22 Domestic Septage Vector Attraction Reduction Approaches*

Approach I	Domestic septage shall be injected below the surface of the land, *and* no significant amount of the domestic septage shall be present on the land surface within 1 hour after the domestic septage has been injected.
Approach II	Domestic septage applied to the land surface shall be incorporated into the soil surface plow layer within 6 hours after application.
Approach III	The pH of domestic septage shall be raised to 12 or higher by addition of alkaline material and, without the addition of more alkaline material, shall remain at 12 or higher for 30 minutes.

*Adapted from refs. [26,31].



"I certify under penalty of law, that the pathogen requirements in [*insert either alternative 1 or 2*] and the vector attraction reduction requirements in [*insert either vector reduction alternative 1, 2 or 3*] have/have not [*circle one*] been met. This determination has been made under my direction and supervision in accordance with the system designed to assure that qualified personnel properly gather and evaluate the information used to determine that the pathogen requirements and the vector attraction reduction requirements have been met. I am aware that there are significant penalties for false certification including the possibility of fine and imprisonment."

Im Anna Plier

Signed: "Im Anna Plier"

(to be signed by the person
designated as responsible in the
firm that applies domestic septage

Figure 1.11 Typical pathogen-reduction certification for land appliers of domestic septage.

load, of domestic septage must be tested for pH before it is applied to agricultural land, forest, or a reclamation site. Domestic septage that is not treated by pH adjustment does not need to be monitored.

The land applier of domestic septage must sign a certification form that states that the pathogen and vector attraction reduction requirements of the 40 CFR Part 503 rule have been met. The septage land applier must retain this certification for at least 5 years. Figure 1.11 illustrates a typical pathogen and vector attraction reduction certification form for land-applied domestic septage.

1.2.10.5 Management practices. There are no specific federal management practice requirements for land appliers of domestic septage to nonpublic contact sites in the 40 CFR Part 503 rule. On the other hand, many states do require that specific management practices be followed. State or local management requirements may include the specification of minimum distances between domestic septage land-application sites and drinking water wells and/or surface water. Good practice also would suggest a caution against applying domestic septage to flooded, frozen, or snow-covered land that could result in contaminated runoff.

1.2.10.6 Recordkeeping. For land application of domestic septage to nonpublic contact sites, the records that must be kept for at least 5 years are summarized in Table 1.23.

1.2.11 Liability issues and enforcement oversight

The 40 CFR Part 503 rule is self-implementing, which means that its provisions must be followed regardless of whether or not a permit has been issued. The 40 CFR Part 503 rule includes enforcement measures regarding the proper testing and application of biosolids [24]. Landowners and leaseholders who use biosolids beneficially as a fertilizer substitute or soil conditioner in accordance with the 40 CFR Part 503 rule specifications are protected from liability under CERCLA (i.e., Superfund) as well as any enforcement action from the USEPA. Where the federal requirements are not followed, appliers of biosolids are vulnerable to USEPA enforcement actions or citizen-initiated law suits and can be required to remediate any problems for which they are found liable.

The USEPA oversight of biosolid land-application practices includes a program for administering permits and for monitoring, reporting, and inspecting [30]. Preparers and land appliers of biosolids are

TABLE 1.23 Records That Must Be Kept by Septage Haulers/Land Appliers*

1. The location of the site on which domestic septage is applied.
2. The number of acres in the application site.
3. The date and time domestic septage is applied.
4. The nitrogen requirement of the crop or vegetation grown on the site in a 365-day period.
5. The rate in gallons per 365-day period at which domestic septage is applied.
6. Certification that pathogen and vector attraction reduction requirements have been met.
7. Descriptions of how pathogen and vector attraction reduction requirements have been met.

*Adapted from refs. [20,26,31].

required to maintain appropriate records, and Class I biosolids management facilities must self-report on their activities during the preceding calendar year by February 19. Annual compliance reports must include information on biosolids quality and, in the case of CPLR biosolids, a field-by-field analysis of the site activity including information on management practices and the cumulative application of regulated pollutants.

To ensure compliance with the 40 CFR Part 503 rule, the USEPA conducts routine sampling and inspection of regulated facilities. If discrepancies are identified, enforcement actions can be taken, including fines, injunctive relief, and, in cases of deliberate violation of the federal requirements, criminal imprisonment [24].

1.3 Surface Disposal

In the 40 CFR Part 503 rule (Subpart C), *surface disposal* is defined as an activity in which biosolids are placed on land for final disposal. However, if the disposal activity involves the discharge of biosolids that meet pollutant concentrations (e.g., cumulative pollutant loading rates or annual pollutant loading rates) as well as ceiling limits, and the biosolids are land applied at agronomic rates, it is considered land application, not surface disposal. Although some surface-disposal sites may be used for beneficial purposes (e.g., nutrient management), the placement of biosolids on land for treatment or storage is not considered surface disposal. Finally, codisposal of biosolids in a municipal solid waste landfill is not considered surface disposal under the 40 CFR Part 503 rule. The practice of discharging biosolids to a municipal landfill is regulated by 40 CFR Part 258 [11,19,30,32]. The biosolids surface-disposal facilities regulated under the 40 CFR Part 503 rule include the following: (1) monofills, (2) surface impoundments and lagoons, (3) waste piles, (4) dedicated disposal sites, and (5) dedicated beneficial-use sites. Each of these facilities is described briefly in the following sections. Readers interested in obtaining further detail on the design of surface-disposal facilities are directed to the following references [14,32].

Monofills are defined as landfills in which only biosolids are disposed. The two basic methods of monofilling biosolids include (1) trenching and (2) area filling. In the trenching method, biosolids are disposed entirely below the ground surface. Since subsurface excavation is required, trenching is only feasible in situations where the groundwater and/or bedrock is located at a sufficient depth to permit biosolids land disposal while still maintaining an adequate soil buffer between the bottom of the biosolids deposits and the groundwater surface or bedrock [14].

Since the trench sidewalls provide adequate structural support, soil is not required as a biosolids bulking agent during surface disposal of biosolids in trenches. However, biosolids that are normally discharged directly into the trench from haul vehicles are covered daily by excavated soil. Odor control is achieved in trenching operations by the daily application of soil cover material. Because of their effective management of odors, trenches are ideal facilities for the disposal of unstabilized biosolids [14,32].

At present, the two types of trench designs suitable for biosolids or sludge disposal are the (1) narrow trench and (2) wide trench designs. *Narrow trenches* are defined as those having widths of less than 10 ft (3.0 m), while *wide trenches* are those constructed with

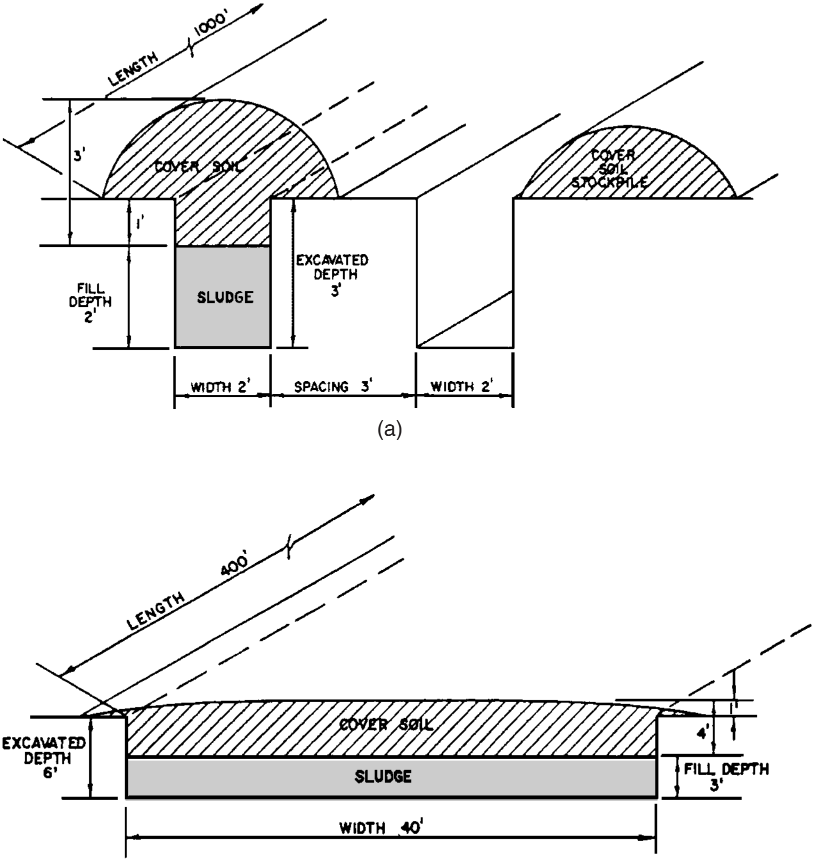


Figure 1.12 Schematic diagram of trenching operations: (*a*) narrow trench; (*b*) wide trench.

widths greater than 10 ft. Schematic diagrams of both a narrow and a wide trench design are depicted in Fig. 1.12.

Although the depth and length of both narrow and wide trenches are variable, the specific limitations on the values of these design parameters usually depend on such factors as (1) depth to groundwater and bedrock, (2) sidewall stability, and (3) equipment limitations [14,32]. Summary design criteria for both narrow and wide trenches are provided in Table 1.24.

The principal advantage of the narrow trench design is its ability to receive biosolids (or sludge) with a relatively low solids content. For biosolids or sludge with a solids content of between 15 and 20 percent, a narrow trench having a width of 2 to 3 ft is normally sufficient for proper surface-disposal operation [14,32]. Under most circumstances, soil applied as cover material on biosolids with such a low solids content would sink. However, because of the narrowness of the trench, the applied soil receives structural support from the solid ground on either

TABLE 1.24 Design Criteria for Narrow and Wide Trenches*

	Narrow Trench
Biosolids solids content	15–20% for widths of 2 to 3 ft (0.6–0.9 m), 20–28% for widths of 3 to 10 ft (0.9–3.0 m)
Biosolids characteristics	Unstabilized or stabilized
Hydrogeology	Deep groundwater and bedrock
Ground slopes	<20%
Bulking agent required	No
Cover soil required	Yes
Cover soil thickness	3 to 4 ft (0.9–1.2 m)
Imported soil required	No
Biosolids application rate	1200–5600 yd^3/acre (2300–10,600 m^3/ha)
	Wide Trench
Biosolids solids content	20–28% for land-based disposal equipment,† 28% for biosolids-based disposal equipment‡
Biosolids characteristics	Unstabilized or stabilized
Hydrogeology	Deep groundwater and bedrock
Ground slopes	<10%
Trench width	>10 ft (3.0 m)
Bulking agent required	No
Cover soil required	Yes
Cover soil thickness	3 to 4 ft (0.9–1.2 m) for land-based disposal equipment,† 4 to 5 ft (1.2–1.5 m) for biosolids-based disposal equipment‡
Imported soil required	No
Biosolids application rate	3200–14,500 yd^3/acre (6000–27,400 m^3/ha)

*Adapted from refs. [14,32].

†Land-based disposal equipment refers to equipment that discharges biosolids/sludge from the sides of the trench.

‡Biosolids-based disposal equipment refers to equipment that is driven into the trench before discharging biosolids/sludge.

side of the trench, allowing a bridge to form over the discharged biosolids (Fig. 1.13).

The principal disadvantage of the narrow trench design is that it is characterized by relatively inefficient land use. Typical biosolids application rates range from 1200 to 5600 cubic yards per acre (2300–10,600 m³/ha). Another disadvantage of the narrow trench design is that installation of liners for groundwater protection is impractical.

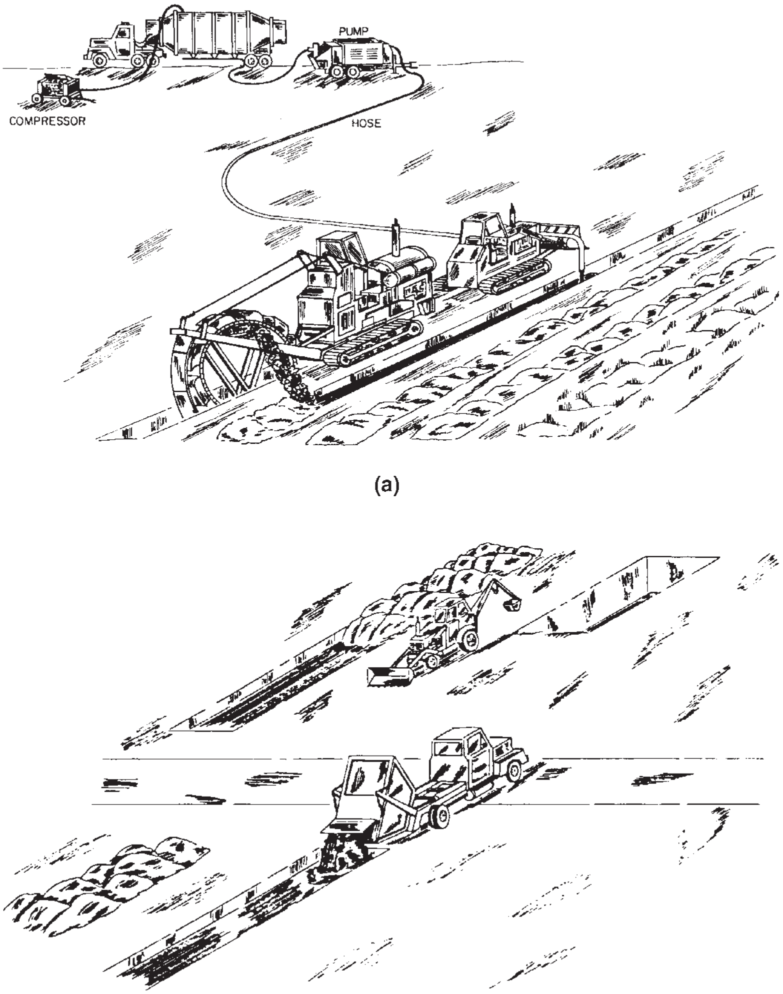


Figure 1.13 Illustration of narrow trench operation: (*a*) surface disposal of liquid biosolids; (*b*) surface disposal of dewatered biosolids.

The principal advantage of the wide trench design is that it is less land-intensive than the narrow trench design. Typical biosolids application rates in wide trench operations will range from 3200 to 14,000 cubic yards per acre (6000–27,400 m^3/ha). Another advantage of a wide trench design is that liners can be installed easily to protect groundwater from contamination. Finally, with the use of a liner, wide trench excavation can proceed closer to bedrock and/or groundwater than it could in narrow trenches constructed without such protection.

A principal disadvantage of the wide trench design is that to ensure the stability of the soil cover, only biosolids with a high solids content (e.g., 25 percent solids content or higher) should be discharged to the unit. The high solids content requirement, however, is limited by the fact that biosolids with a solids content of greater than 32 percent will not spread out evenly in a trench when discharged from the trench sidewall [14]. Therefore, if wide trenches are used for placement of biosolids with solids contents greater than 32 percent, biosolids haul vehicles must enter the trench and discharge the biosolids directly onto the trench floor. Another disadvantage of the wide trench design is its need for flatter terrain than that used for narrow trenches. To ensure the even spread of biosolids, the trench floor should be nearly level, which can be constructed more easily when located in low-relief areas [14].

In contrast to trenching applications, in area-filling operations, biosolids are placed above the original ground surface. Since excavation is not required, area-fill surface disposal is particularly amenable to areas characterized by shallow groundwater and/or bedrock. Although there are no restrictions regarding the solids content of the received biosolids, biosolids stability and bearing capacity must be relatively good to prevent slumping. To increase biosolids structural stability, imported soil is usually mixed with biosolids as a bulking agent prior to discharge to the disposal area. Finally, since the application of a daily soil cover is not an integral part of area-fill surface-disposal operations, stabilized biosolids are more suitable for disposal through this disposal practice.

In general, the principal advantage of an area-fill surface-disposal unit is that liners can be installed more readily than at trench operations. With the likely proximity of groundwater or bedrock to the ground surface at such sites, liner installation often is mandatory. The main disadvantage of area filling is that, with or without liners, surface runoff will be significant, and, therefore, installation of appropriate surface drainage control facilities is necessary.

Three approaches currently available for area filling of biosolids (or sludge) are (1) area mounding, (2) area-fill layer, and (3) diked containment. Each of these approaches is described in the following sections.

In area-fill mounding operations, the solids content of discharged biosolids should be at least 20 percent [14,32]. Biosolids received at the site typically are mixed with soil to produce a mixture that is more stable and has a greater bearing capacity than biosolids alone. Appropriate bulking agent ratios may vary between 0.5 to 2.0 parts soil for each part biosolid (mass basis) depending on the initial biosolids moisture content. At the mounding area, the biosolids-soil mixture is stacked into mounds approximately 6 ft (1.8 m) high. Soil cover material is then applied to the mounds at a depth of at least 3 ft (Fig. 1.14). Other relevant biosolids characteristics and site conditions appropriate for area-fill mounding operations are provided in Table 1.25.

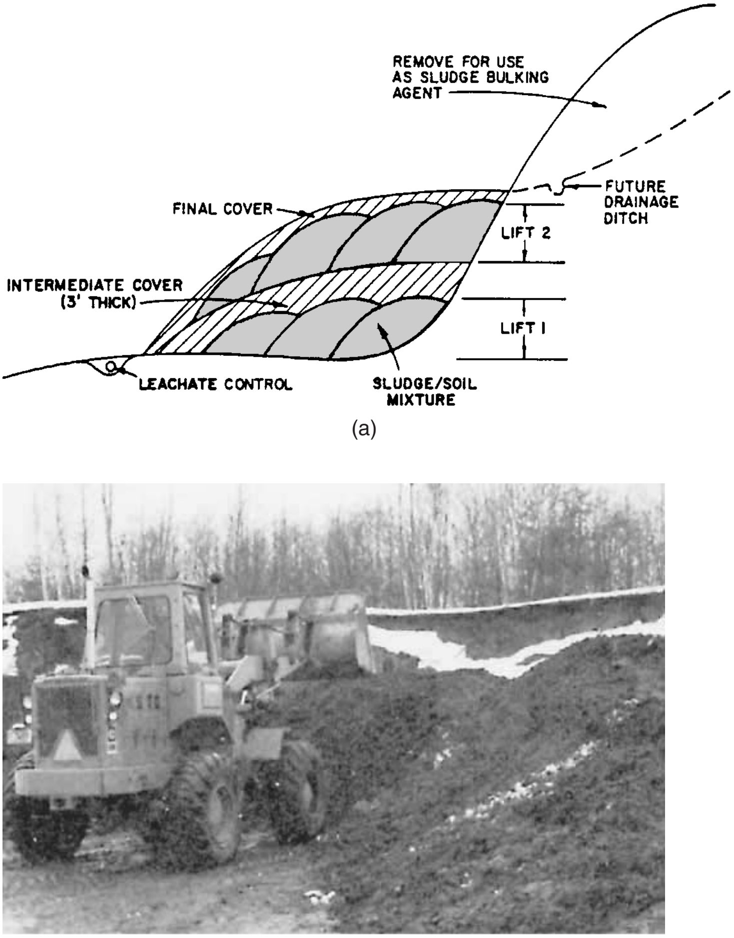
An advantage of the area-fill mounding operation is its optimal land utilization. Biosolids application rates typically range from 3000 to 14,000 cubic yards per acre (5700–26,400 m^3/ha). A principal disadvantage of area-fill mounding is the constant need to push and stack slumping mounds. For this reason, area-fill mounds often have high personnel and equipment requirements [14,32].

In area-fill layer operations, biosolids received at the surface-disposal site may have a solids content as low as 15 percent (Fig. 1.15). Biosolids (or sludge) received at the site are initially mixed with a soil bulking agent to produce a mixture that is more stable and has a greater bearing capacity than biosolids alone. The typical soil bulking agent to biosolids ratio will range from 0.25 to 1.0 parts soil to one part biosolids (mass basis). The actual ratio used in day-to-day operations will depend on the moisture content of the received biosolids.

After mixing the biosolids with soil, the mixture is spread evenly in layers ranging from 0.5 to 3.0 ft (0.15–0.91 m) in thickness. Interim cover between consecutive layers is sometimes applied in layers ranging from 0.5 to 12.0 in (1.3–31 cm) in thickness. The final cover on the area-fill-layer surface-disposal unit should be from 2 to 4 ft (0.6–1.2 m) in thickness. Other relevant design criteria for area-fill-layer surface-disposal facilities are provided in Table 1.26.

An advantage of an area-fill-layer operation is that completed fill areas are relatively stable. As a result, the maintenance demand (i.e., personnel and equipment requirements) is not as extensive as for area-fill mounds. The principal disadvantage of an area-fill-layer operation is poor land utilization. Biosolids application rates for such facilities range from 2000 to 9000 cubic yards per acre (3780–17,000 m^3/ha).

In diked containment operations, biosolids (or sludge) are placed entirely above the original ground surface. In some situations, dikes are constructed on level ground and all four sides of a containment area, whereas in others, the containment area may be placed at the toe of a hill so that the steep slope can be used (Fig. 1.16).



(a)

(b)

Figure 1.14 (*a*) Schematic diagram of area-fill mounding surface-disposal site; (*b*) photograph of area-fill mounding operations.

Access to diked containment surface-disposal units is provided at the top of the dikes so that haul vehicles can discharge biosolids (or sludge) directly into the containment area. In addition to the final soil cover, interim cover may be applied to the biosolids containment area during the filling process. Relevant site conditions as well as design criteria for diked containment surface disposal systems are provided in Table 1.27.

An advantage of diked containment surface-disposal systems is that since the facility is entirely above ground, it can be relatively large with typical dimensions of 50 to 100 ft (15–30 m) wide, 100 to 200 ft

TABLE 1.25 Area-Fill Mounding Design Criteria*

Biosolids solids content	>20%
Biosolids characteristics	Stabilized
Hydrogeology	Shallow groundwater and bedrock
Ground slopes	Suitable for steep terrain
Bulking agent required	Yes
Bulking agent	Soil
Bulking agent ratio	0.5 to 2.0 soil to 1 biosolids
Cover soil required	Yes
Cover soil thickness	3 ft (0.9 m) for final, 1 ft (0.3 m) for interim
Imported soil required	Yes
Biosolids application rate	3000–14,000 yd^3/acre (5700–34,600 m^3/ha)

*Adapted from ref. [29].

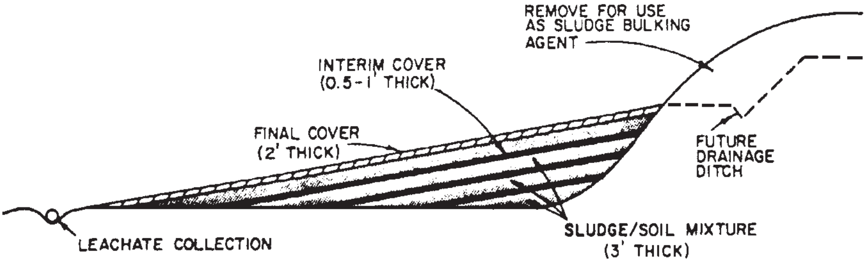


Figure 1.15 Schematic diagram of area-fill-layer surface disposal.

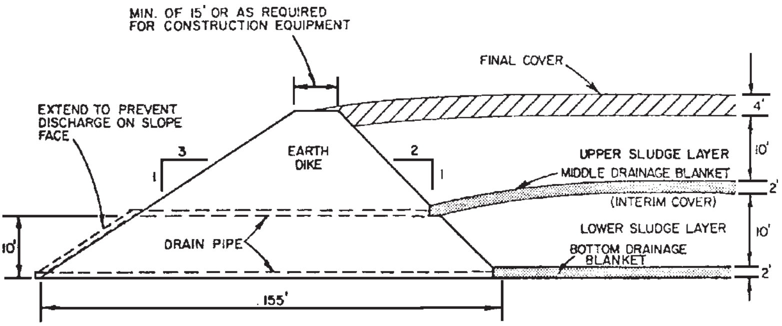
TABLE 1.26 Design Criteria for Area-Fill-Layer Surface-Disposal Facility*

Biosolids solids content	15%
Biosolids characteristics	Stabilized
Hydrogeology	Shallow groundwater and bedrock
Ground slopes	Suitable for medium slopes but level ground preferred
Bulking agent required	Yes
Bulking agent	Soil
Bulking agent ratio	0.25 to 1.0 soil to 1 biosolids
Cover soil required	Yes
Cover soil thickness	2 to 4 ft (0.6–1.2 m) for final, 0.5 to 1.0 ft (0.15–0.3 m) for interim
Imported soil required	Yes
Biosolids application rate	2000–9000 yd^3/acre (3800–17,600 m^3/ha)

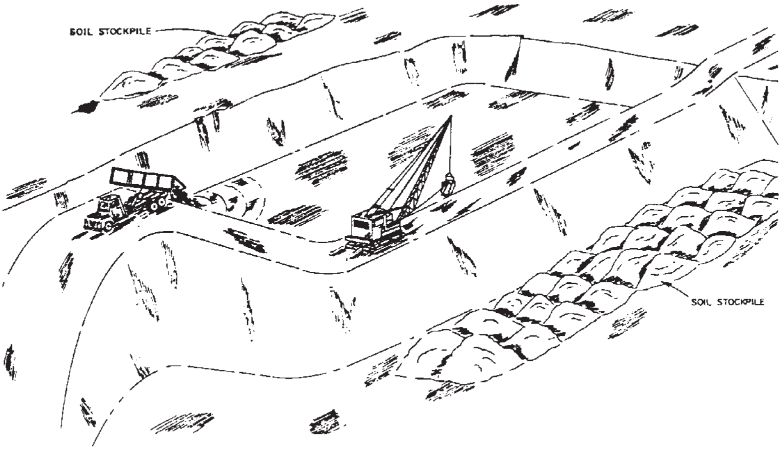
*Adapted from ref. [32].

(30–60 m) long, and 10 to 30 ft (3–9 m) deep. The biosolids loading rates to diked containment systems typically range from 4800 to 15,000 cubic yards per acre (9100–28,400 m^3/ha).

The principal disadvantage of diked containment systems is that due to the depth of the fill together with the weight of the interim and final cover, the biosolids may be subjected to significant pressures. As a result of the overburden pressure, moisture in the containment area is driven into the surrounding dikes and into the containment floor



(a)



(b)

Figure 1.16 (*a*) Schematic diagram of diked containment surface-disposal operation; (*b*) illustration of diked containment operation.

[14,32]. To prevent the migration of pollutants and to protect groundwater, liners and other leachate controls are essential in the design of diked containment surface-disposal systems.

Surface impoundments are above-ground or below-ground installations where liquid biosolids or sludge (e.g., 2 to 5 percent solids content) are placed for final disposal. While below-ground installations (referred to as *lagoons*) require soil excavation, dikes are used to contain the biosolids or sludge in above-ground systems (e.g., *surface impoundments*). Since the application of a daily soil cover is not part of normal operations for either lagoons or surface impoundments, stabilized biosolids are more suitable to be disposed in these systems.

The liquid level in both lagoons and surface impoundments is maintained at a constant height by an outflow decant pipe. Moisture is

TABLE 1.27 Design Criteria for Diked Containment Surface-Disposal Systems*

Biosolids solids content	20–28% for land-based disposal equipment,† 28% for biosolids-based disposal equipment‡
Biosolids characteristics	Unstabilized or stabilized
Hydrogeology	Shallow groundwater and bedrock
Ground slopes	Suitable for steep terrain as long as a level area is prepared inside dikes
Bulking agent required	No (but sometimes used)
Bulking agent	Soil
Bulking agent ratio	0.25 to 1.0 soil to 1 biosolids
Cover soil required	Yes
Cover soil thickness	1 to 2 ft (0.3–0.6 m) for interim (land-based disposal equipment†), 2 to 3 ft (0.6–0.9 m) for interim (biosolids-based disposal equipment‡), 3 to 4 ft (0.9–1.2 m) for final cover (land-based disposal equipment†), 4 to 5 ft (1.2–1.5 m) for final cover (biosolids-based disposal equipment‡)
Imported soil required	Yes
Biosolids application rate	4800–15,000 yd^3/acre (9100–28,400 m^3/ha)

*Adapted from refs. [14,32].
†Land-based disposal equipment refers to equipment that discharges biosolids/sludge from the sides of the trench.
‡Biosolids-based disposal equipment refers to equipment that is driven into the trench before discharging biosolids/sludge.

removed from lagoons and surface impoundments by both evaporation and through the outflow decant pipe (see Chap. 3). The liquid that is decanted may be conveyed to a sewer collection system or a POTW headworks, or it can be treated prior to discharge into the environment [29,32]. Seepage through the base of the containment area is controlled either by a liner and leachate collection system or, in some cases, by natural geologic conditions [1,29,35].

Biosolids disposal rates for lagoons or surface impoundments are similar to those for area-fill diked surface-disposal systems and typically will range from approximately 4000 to 15,000 cubic yards per acre (9100–28,400 m^3/ha). Therefore, a principal advantage of lagoons or surface impoundments is their efficient land use relative to trenches or area-fill surface-disposal systems [14,32].

It should be noted that, in many cases, lagoons and surface impoundments are used to treat and/or dewater biosolids. When lagoons and/or surface impoundments are used exclusively for biosolids treatment or dewatering, they are not considered surface-disposal sites. However, in some treatment or dewatering lagoons (or surface impoundments), the settled biosolids may remain in the unit indefinitely as more biosolids are added. When biosolids fill the lagoons or surface impoundments to capacity, the facility can be either dredged (with the solids taken to a landfill or incinerator) or covered with soil and closed. Only lagoons and surface impoundments that are intended for closure after filling to capacity with solids are regulated under the 40 CFR Part 503 rule.

Biosolids waste piles or mounds are surface-disposal systems constructed at or above the ground surface without any auxiliary containment structures (e.g., dikes). Mounds differ from biosolids area fills in that daily cover is not applied to mounds, and the addition of a bulking agent is optional. Since biosolids are not covered daily, only stabilized material that meets the pathogen (i.e., Class A or Class B) and vector attraction reduction requirements may be disposed in these units [14,32].

Dedicated surface-disposal (DSD) sites are surface-disposal units in which biosolids are injected into soil or incorporated (i.e., plowed) into soil after being sprayed or spread on the land surface [14,32]. Since biosolids are discharged to land in DSD systems at higher rates than are allowed when biosolids are used as a soil amendment or fertilizer (40 CFR Part 503, Subpart B), dedicated surface disposal does not qualify as land application [32]. DSD sites range in size from less than 10 acres (4 ha) to greater than 10,000 acres (4000 ha). A DSD site may have several active biosolids units. Individual units are typically 10 to 100 acres (4–40 ha) in area [14].

DSD sites are characterized by repetitive applications of biosolids at a site for several years. Depending on the biosolids moisture content, ground slope, and soil conditions, there are several methods of applying biosolids to DSD sites. These application methods include (1) spraying (using fixed or portable irrigation equipment), (2) ridge and furrow, (3) direct surface spreading by tank truck, tractors, and farm tank wagons, and (4) subsurface injection [14,32]. Although the site-specific biosolids disposal rates are determined by considering the biosolids moisture content, climate, and soil characteristics, in general, disposal rates typically range from 50 to 2000 tons (dry weight) per acre-year in these systems. To minimize the formation of leachate, the biosolids disposal rate for a particular site should never exceed the net soil evaporation rate (i.e., evaporation minus precipitation).

Some DSD sites are used to grow feed and/or fiber crops or vegetative cover. These facilities are known as *dedicated beneficial-use sites*. Since vegetation is grown on beneficial dedicated surface-disposal sites, biosolids disposal rates are lower at these sites (e.g., 31–83 dry metric tons/ha·yr) than a DSD site, on which no crops are grown. For dedicated beneficial-use surface disposal operations, the 40 CFR Part 503 rule requires that the permitting authority issue the owners/operations of the site a permit that specifies appropriate management practices that ensure the protection of public health and the environment.

Many POTWs employ DSD or dedicated beneficial-use surface disposal on their own property because these systems (1) are suitable for liquid biosolids disposal, (2) have minimum transportation costs (if adequate land area is available), and (3) have relatively low capital and operating costs. Finally, it should be noted that a POTW or other DSD site owner may choose to establish a beneficial dedicated surface-disposal site if soil erosion or soil acidity is a concern at the facility or if the POTW is committed to a beneficial-use policy. Although the vegetation can assist in controlling soil erosion and acidity, the primary purpose of the site is final disposal of biosolids.

1.3.0 Site life and size

Site life is determined by the size of the site, the quantity and quality of the biosolids, and the surface-disposal method. In estimating the required size, it is important to recognize that not all the site can be filled. Therefore, a site should be viewed in terms of (1) gross area (i.e., total area within the property boundary) and (2) usable fill area (excludes areas for buffers, access roads, and soil stockpiles—typically 50 to 70 percent of the gross area).

Although, in practice, a municipality will not define the site life initially, a minimum acceptable site life should be established because startup costs become less significant over an extended period. Example 1.5 illustrates a typical calculation of site life given the landfilling method and biosolids generation rate, whereas Example 1.6 provides an example of how site life may be estimated given the usable area, biosolids generation rate, and land-filling method.

Example 1.5 The Turner County Water Reclamation Facility desires to use a wide trench for disposal of its biosolids. If the solids content of the biosolids is 30 percent, estimate the total trench volume and usable average and minimum gross acreage if the biosolids generation rate is 70 yd^3/day. Other pertinent data include the following:

Trench life: 15 years

Trench dimensions: 45 ft wide by 10 ft deep by 200 ft long

Trench spacing: 10 ft between trenches

Buffer: 200 ft minimum from usable filling area to property line

Other assumptions: 25 percent additional area for access roads, dumping pad, and miscellaneous uses

solution

Step 1. Estimate trench volume needed.

$$\text{Trench volume (yd}^3) = \frac{70\ \text{yd}^3}{\text{day}} \cdot \frac{365\ \text{days}}{\text{year}} \cdot 15\ \text{years}$$

$$= 383{,}250\ \text{yd}^3$$

Step 2. Estimate the number of trenches required.

$$\text{Number of trenches required} = \frac{383{,}250 \text{ yd}^3 \cdot \dfrac{27 \text{ ft}^3}{\text{yd}^3}}{45 \text{ ft} \cdot 10 \text{ ft} \cdot 200 \text{ ft}}$$

$$= 115 \text{ trenches}$$

Step 3. Find usable area required. Note that since there is 10 ft in between trenches, the actual area used by each trench is 55 ft (width) · 210 ft (length).

$$\text{Usable area (acres)} = 115 \text{ trenches} \cdot 55 \text{ ft} \cdot 210 \text{ ft}$$

$$= 1{,}328{,}250 \text{ ft}^2 \cdot \frac{\text{acre}}{43{,}560 \text{ ft}^2}$$

$$= 30.5 \text{ acres}$$

Step 4. Estimate minimum gross area assuming that the site is approximately of equal length on all four sides.

$$\text{Usable area} = 1{,}328{,}250 \text{ ft}^2 = 1153 \text{ ft} \cdot 1153 \text{ ft}$$

Add a 200-ft buffer to each dimension plus 25 percent for roads, dumping pad, etc.

$$\text{Gross area (acres)} = (1353 \text{ ft} \cdot 1353 \text{ ft}) + 0.25 \cdot (1353 \text{ ft} \cdot 1353 \text{ ft})$$

$$= 42.03 \text{ acres} + 0.25 \cdot (42.03 \text{ acres})$$

$$\cong 53 \text{ acres}$$

Example 1.6 The Turner County Water Reclamation Facility (Example 1.5) has only 20 acres of usable fill area. Estimate the site life if the biosolids generation rate remains at 70 yd^3/day. Assume that all trench dimensional data from Example 1.5 apply.

solution

Step 1. Estimate the total number of trenches that can be accommodated within the 20-acre usable limit. Note that in estimating the number of trenches, the 10-ft buffer must be included.

$$\text{Number of trenches} = \frac{20 \text{ acres} \cdot \dfrac{43{,}560 \text{ ft}^2}{\text{acre}}}{55 \text{ ft (width)} \cdot 210 \text{ ft (length)}}$$

$$= 75.4 \text{ trenches}$$

Step 2. Estimate the trench volume available for biosolids disposal.

Trench volume available (yd^3)

$$= 75.4 \text{ trenches} \cdot \frac{45 \text{ ft (width)} \cdot 200 \text{ ft (length)} \cdot 10 \text{ ft (depth)}}{\text{trench}} \cdot \frac{yd^3}{27 \text{ ft}^3}$$

$$= 251{,}429 \text{ yd}^3$$

Step 3. Estimate site life (years).

$$\text{Site life (years)} = \frac{251{,}429 \text{ yd}^3}{\dfrac{70 \text{ yd}^3}{\text{day}} \cdot \dfrac{365 \text{ days}}{\text{year}}}$$

$$= 9.84 \text{ years}$$

1.3.1 Surface storage of biosolids

Under the 40 CFR Part 503 rule, discharging biosolids to a land surface is considered storage if biosolids are placed and remain on land for 2 years or less. If biosolids remain on land for longer than 2 years, the regulations require that the site be designated as an active biosolids surface-disposal unit, and it becomes subject to all the surface-disposal requirements mandated in the 40 CFR Part 503 rule. In limited circumstances, biosolids can remain on a site for more than 2 years and not be designated as an active disposal site. To legally store biosolids for more than 2 years on land, the biosolids preparer must demonstrate to the permitting authority that the storage unit is not an active surface-disposal site. At a minimum, the biosolid preparer must forward the following information to the permitting authority: (1) name and address of the person who prepares the biosolids, (2) name and address of the person who either owns or leases the land, (3) the location, by either street address or latitude and longitude, of the land, (4) an explanation of why biosolids need to remain on the land for more than 2 years prior to final use or disposal, and (5) the approximate time when biosolids will be transferred from storage to their final use or disposal destination. If the permitting authority allows the facility to store biosolids for more than 2 years, a copy of the forwarded information must be retained by the biosolids preparer for the entire period that the biosolids remain in storage.

1.3.2 Regulatory requirements for surface disposal

The seven regulatory requirements that must be met for legal operation of a biosolids surface-disposal site include (1) general requirements, (2) pollutant limits, (3) management practices, (4) operational standards for pathogen and vector attraction reduction, (5) monitoring requirements, (6) recordkeeping requirements, and (7) reporting requirements. Each of these requirements is described in the following sections.

1.3.2.1 General requirements. The 40 CFR Part 503 rule states that no person shall place biosolids on an active biosolid surface-disposal unit unless the requirements of Subpart C of the 40 CFR Part 503 rule are met. In addition to operating a biosolids surface-disposal facility in accordance with the provisions specified in the 40 CFR Part 503 rule, the regulation requires the immediate closure of all biosolids surface-disposal units that are located (1) within 60 m (200 ft) of a geologic fault with displacement in Holocene time (i.e., within the past 11,000 years), (2) in an unstable area, or (3) in a wetland. Although a biosolids surface-disposal unit whose location fits any of these criteria should have been closed by March 22, 1994, there are two conditions under which this requirement may be waived. First, if the permitting authority has indicated that the location of a specific surface-disposal unit within the 60 m of a fault with displacement in Holocene time is acceptable, the facility does not have to close, or second, if a permit were issued to the facility under Section 402 of the Clean Water Act (CWA) that allowed construction of the biosolids surface-disposal unit in a wetland, the facility may remain active.

When a biosolids surface-disposal unit is scheduled for closure, the permitting authority must be notified. The 40 CFR Part 503 rule requires that the owner/operator of the surface-disposal unit provide the permitting authority with a written closure plan at least 180 days prior to site closing [21,30,32]. The plan must provide detailed descriptions of both the closure and postclosure activities, including (1) the operation and maintenance of the leachate collection system for at least 3 years after closure (if the unit has such a system), (2) the system used to monitor the air for methane gas for at least 3 years after closure (if the surface disposal unit is covered), and (3) measures to restrict public access for at least 3 years after closure. In certain circumstances, the permitting authority may require that the closure plan include provisions for monitoring the air for methane gas or leachate collection for more than 3 years. For example, the permitting authority may determine that the monitoring of methane gas for a period of longer than 3 years is warranted when the biosolids discharged to the surface-disposal site had not been stabilized previously. Similarly, in areas of high rainfall, the permitting authority may deem it necessary to collect leachate for a period longer than 3 years to ensure that the integrity of the liner is maintained.

Finally, the general requirements require notification of subsequent property owners of the existence of the biosolids surface-disposal operation. In other words, when ownership of a biosolids surface-disposal site changes, the former owner must provide the subsequent owner with written notification that biosolids were placed on the land. The actual contents of the notification that is transmitted to the subsequent owner of a biosolids surface-disposal site will vary depending on when the land was sold and the provisions of the closure plan [21,24,30]. For example, if a biosolids surface-disposal site were covered, had a liner/leachate collection system, and was sold 1 year after closure, the notification must inform the subsequent owner that the property was used to dispose of biosolids and that the new owner must operate the leachate collection system, monitor the air for methane gas, and restrict public access to the site for at least an additional 2 years.

1.3.3 Pollutant limits

For surface disposal of biosolids, a *pollutant limit* is defined as the permissible concentration of pollutant in biosolids (dry-mass basis) [32]. Subpart C of 40 CFR Part 503 sets pollutant limits for arsenic, chromium, and nickel in biosolids that are applied to surface-disposal units without liners or leachate collection systems [28]. The USEPA determined that the human health risks associated with biosolids discharged to surface-disposal sites equipped with liners and leachate collection systems were negligible, and therefore, no pollutant limits were established for biosolids disposed in these facilities.

For regulatory purposes, the USEPA has defined a liner as a layer of relatively impervious soil, such as clay, or a layer of synthetic material that covers the bottom of an active biosolids surface-disposal unit and has a hydraulic conductivity of $1 \cdot 10^{-7}$ cm/s or less (Fig. 1.17). The liner reduces the vertical seepage of liquid (and pollutant migration) from the biosolids surface-disposal site into groundwater. Similarly, the USEPA has defined a leachate collection system as a system or device installed immediately above a liner that collects and removes residual liquids (and migrating contaminants) as they seep through the surface-disposal site [29,35].

There are basically two sets of pollutant limits established for arsenic, chromium, and nickel in biosolids discharged to surface-disposal units that are not equipped with liners and leachate collection systems. The first set of pollutant limits is applicable to situations in which the actual boundary of the surface disposal unit is 150 m (500 ft) or more from the property boundary. When this condition exists, the biosolids pollutant limits are those listed in Table 1.28.

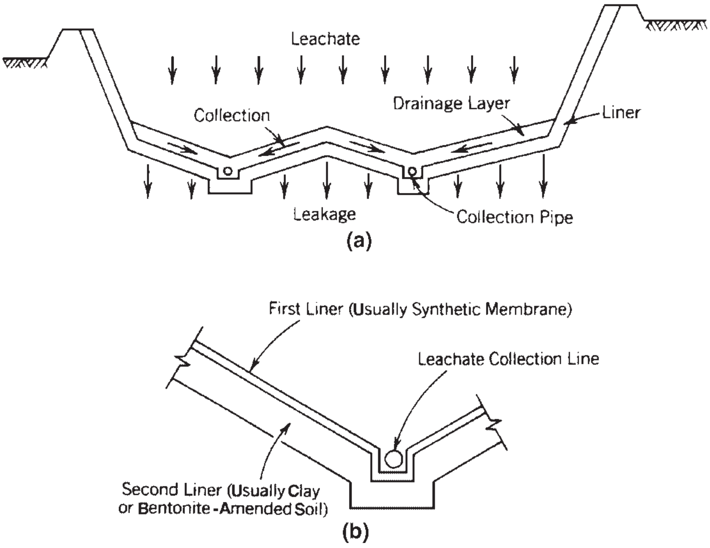


Figure 1.17 (*a*) Leachate collection system in surface-disposal system (refs. [1,11,29,35]). (*b*) Double-liner system typically used in biosolids surface-disposal sites (refs. [1,11,29,35]). (*Adapted by permission from A. Bagchi, Design, Construction, and Monitoring of Sanitary Landfill, Wiley, 1989.*)

However, when the distance from the boundary of the surface-disposal unit to the facility's property line is less than 150 m, the pollutant limits are more stringent. Table 1.29 summarizes the pollutant limits for arsenic, chromium, and nickel in biosolids discharged to a surface-disposal unit whose boundary is less than 150 m from the facility's property line. It should be noted that as the distance between the surface-disposal unit's boundary and the property line diminishes, the pollutant limits also decrease.

As an alternative to meeting the pollutant limits provided in either Table 1.28 or Table 1.29, the owner/operator of an unlined biosolids surface-disposal site can choose to meet site-specific pollutant limits set by the permitting authority. To use this option, the owner/operator of the unlined surface-disposal unit must provide site-specific information to the permitting authority. The permitting authority will use the site-specific information to develop site-specific pollutant limits using an exposure pathway risk-assessment evaluation [29]. In all cases, the site-specific pollutant limits must be either equal to or greater than the pollutant limits given in Table 1.28 or equal to the existing pollutant concentrations in the biosolids, whichever is more stringent [21,24,32].

TABLE 1.28 Pollutant Limits for Biosolids Disposed in Surface-Disposal Units without Liner and Leachate Collection Systems: Distance of Unit Boundary from Property Line Is at Least 150 m*

Pollutant	Pollutant limit (mg/kg)†
Arsenic	73
Chromium	600
Nickel	420

*Adapted from ref. [32].
†Dry-weight basis.

TABLE 1.29 Pollutant Concentration Limits: Unit Boundary to Property Line Distance Is Less than 150 m*

	Pollutant concentration (mg/kg)†		
Unit boundary to property line distance (m)	Arsenic	Chromium	Nickel
0 to less than 25	30	200	210
25 to less than 50	34	220	240
50 to less than 75	39	260	270
75 to less than 100	46	300	320
100 to less than 125	53	360	390
125 to less than 150	62	450	420

*Adapted from ref. [32].
†Dry-weight basis.

Site-specific limits may be a justifiable approach for the owner/operator of the biosolids surface-disposal unit if the site conditions vary significantly from those assumed in the risk assessment used to determine the 40 CFR Part 503 pollutant limits provided in Tables 1.28 and 1.29. In general, if the depth to groundwater is considerable or a natural clay layer underlies the site, the permittee should consider requesting that site-specific pollutant limits be established as the compliance standards.

1.3.4 Management practices

The 40 CFR Part 503 rule includes management practices that must be followed when biosolids are placed on a surface-disposal site. Although most of these management practices apply to all surface-disposal sites, a few apply only to sites with liners and leachate collection systems or to sites with covers. A summary of management practices for surface-disposal sites is provided in Table 1.30. Each management practice is described in further detail in the following sections.

TABLE 1.30 40 CFR Part 503 Management Practices for Surface Disposal*

Protection of threatened and endangered species	Consistency with federal regulation (50 CFR Parts 17.11 and 17.12)
Prohibition against restriction of base flood flow	Protects area's flooding capacity; also protects surface water public health from the release of pollutants in biosolids if a base flood occurs
Geologic stability requirements	Protects the structural integrity of the surface-disposal site and prevents the release of leachate (which may contain pollutants) from the site
Protection of wetlands	Protects wetlands from possible contamination when biosolids are placed in a surface-disposal site
Collection of runoff	Prevents runoff from a surface-disposal site (which may contain pollutants) from being released into the environment
Collection of leachate	Prevents leachate from a surface-disposal site from being released into the environment
Methane gas limit	Ensures that explosive conditions do not exist at the site
Restriction on crop production	If no crop production, prevents pollutants in biosolids at surface-disposal sites from being consumed by humans/animals; if crop production allowed, helps ensure that levels of pollutant taken up by crops do not negatively affect the food chain
Restriction on grazing	If no grazing, prevents animals from ingesting pollutants in biosolids at surface-disposal sites; if grazing allowed, helps ensure that levels of pollutants taken up by crops do not negatively affect the food chain
Restriction of public access	Minimizes public contact with pollutants that may be present in biosolids at surface-disposal sites
Protection of groundwater	Protects groundwater from nitrate contamination

*Adapted from refs. [24,32].

1.3.4.1 Endangered species. Under the 40 CFR Part 503 rule, biosolids cannot be discharged to an active biosolids surface-disposal unit where such disposal is likely to have an adverse effect on a threatened or endangered animal or plant species or their "critical habitats." *Critical habitat* is defined as any place where a threatened or endangered specifies lives and grows during any stage of its life cycle [24]. Any direct or indirect action (or the result of any direct or indirect action) in a critical habitat that diminishes the likelihood of survival and recovery of a listed species is considered destructive of a critical habitat [24,32]. A list of endangered and threatened species is published annually by the U.S. Fish and Wildlife Service as mandated by the Endangered Species Act (PL 99-625). Specific regulations pertaining to the protection of the endangered and threatened plants and animals may be found in 50 CFR Part 17 [10].

1.3.4.2 Restriction on base flood flow. To ensure adequate protection against the migration of biosolids pollutants during storm events, an active biosolid surface-disposal unit must not restrict the flow of a base flood. A *base flood* is defined as a flood that has a 1 percent chance of occurring in any given year (or a rain event that is likely to occur once in 100 years). To determine whether a biosolids surface-disposal site is in a 100-year flood plain, the owner/operator of the facility should consult flood insurance maps and/or flood boundary and floodway maps published by the Federal Emergency Management Agency (FEMA). States, counties, and towns typically have maps delineating flood plains as well.

If the owners/operators of a biosolids surface-disposal site report that their site is within a 100-year flood plain, the permitting authority has the responsibility for determining whether the active biosolids surface-disposal unit will restrict the flow of a base flood. This determination considers the flood plain storage capacity and the floodwater velocities that would exist with and without the presence of the biosolids surface-disposal unit [30,32]. If the permitting authority determines that the presence of the biosolids surface-disposal unit would cause the base flood level to rise 1 additional foot, then the unit is designated as restrictive to the flow of a base flood.

For surface-disposal units that restrict the flow of a base flood, the permitting authority has the option of closing the site or requiring the owner/operator of the site to implement a remedial action to avoid restricting the flow of the base flood. Remedial actions may include requiring the construction of embankments or implementing an alternative surface-disposal unit design intended to prevent the facility from being damaged by flood waters [24,30,32].

1.3.4.3 Geologic stability. In addition to minimizing the restriction of base flood flow, Subpart C of the 40 CFR Part 503 rule specifies the minimum distance between an active biosolids surface-disposal unit and certain types of geologic formations. The three geologic formations of concern are (1) fault areas with displacement in Holocene time, (2) unstable areas, and (3) seismic impact zones.

A *fault* is a crack in the earth along which the ground on either side of the crack may shift. Such ground movement is called *displacement*. The 40 CFR Part 503 rule requires that an active biosolids surface-disposal unit be located at least 60 m (200 ft) away from a fault that has displacement measured in Holocene time (i.e., recent geologic time of approximately the last 11,000 years). Requiring that the location of a biosolids surface-disposal site be a minimum distance from a fault area helps to ensure that the biosolids surface-disposal unit will not be damaged structurally if ground movement occurs and that leachate will not migrate.

An *unstable area* is land where natural and/or human activities may occur that would damage the structures of an active biosolids surface-disposal unit and allow the release of pollutants into the environment. Unstable areas include land where there is a high probability of landslides and/or where the land surface may suddenly collapse when underlying limestone or other materials dissolve [32]. Because of the potential release of biosolids into the environment, the 40 CFR Part 503 rule precludes the construction of biosolids surface-disposal units in an unstable area. Owner/operators of surface-disposal sites may be required by the permitting authority to conduct local geologic studies to demonstrate that unstable conditions do not exist at their site [24,32].

A *seismic impact zone* is an area that has a 10 percent probability that the horizontal ground level acceleration of the rock in the area exceeds 0.1 g once in 250 years [32]. When a surface-disposal site is located in a seismic impact zone, each active biosolids surface-disposal unit must be designed to withstand the maximum recorded horizontal ground level acceleration. This management practice ensures that the containment structures, such as the liner and leachate collection system, will not fail because of ground movement and that leachate will not be released due to seismic activity. In the United States, the U.S. Geological Survey (USGS) maintains records of the location of seismic impact zones.

Various design approaches have been developed for the construction of biosolids surface-disposal units sited in seismic impact zones. Appropriate design approaches for construction of surface-disposal units in seismic impact zones may include shallower unit side slopes and a more conservative design for dikes and runoff controls [32]. Moreover, contingencies for the leachate collection and conveyance systems should be incorporated into the design in case the primary system fails [29,32,35].

1.3.4.4 Protection of wetlands. *Wetlands* are defined as areas in which the soils are saturated with water during part of the year and that support vegetation typically found in saturated soils [12]. Wetlands perform important ecologic functions such as (1) holding floodwaters, (2) serving as wildlife habitat, (3) providing sources of food for numerous species, including 60 percent of the endangered species, (4) reducing soil erosion, and (5) minimizing pollutant migration. Examples of wetlands include swamps, marshes, and bogs [12,23,32].

Under the 40 CFR Part 503 rule, an active biosolids surface-disposal unit may not be located in a wetland unless the owner/operator has received from the regulatory authority a construction permit issued under Section 402 (NPDES permit) or Section 404 (Dredge and Fill permit) of the Clean Water Act (CWA). Imposing siting restrictions on the construction of biosolids surface-disposal units protects wetlands from potential biosolids contamination.

If the owners/operators of a surface-disposal site suspect that all or some portion of their active biosolids surface-disposal unit lies within a wetland, they should contact the U.S. Army Corps of Engineers district office to request a wetland delineation. A wetland delineation, which is the assessment used to determine whether wetlands are present, must be conducted by a qualified team of technical experts [10].

1.3.4.5 Collection of runoff. *Runoff* is defined as the precipitation or liquid that drains over the land surface. Since runoff from an active biosolids surface-disposal unit may contain biosolids pollutants, it must be collected and disposed of according to applicable environmental requirements (e.g., NPDES). The 40 CFR Part 503 rule requires that the runoff collection system for a biosolids surface-disposal unit have the capacity to capture and convey the runoff from a 25-year, 24-hour storm (i.e., a storm that is likely to occur once in 25 years for a 24-hour period) [10,27,29]. To minimize the production of runoff, all upgradient drainage should be collected and directed around the surface-disposal site (i.e., run-on controls). The drainage channels may be constructed of earth of corrugated metal pipe (CMP). Examples of run-on controls are shown in Fig. 1.18.

For proper design of runoff controls, the peak flow and the total runoff volume during the 25-year, 24-hour storm must be calculated to ensure that the stormwater capture and conveyance system is adequate. Approaches for estimating the minimum sizes of conveyance systems necessary for handling storm events are described in the following references [7,8,9].

On the surface-disposal site, all active and completed site working areas should be graded properly. The surface grade should be greater

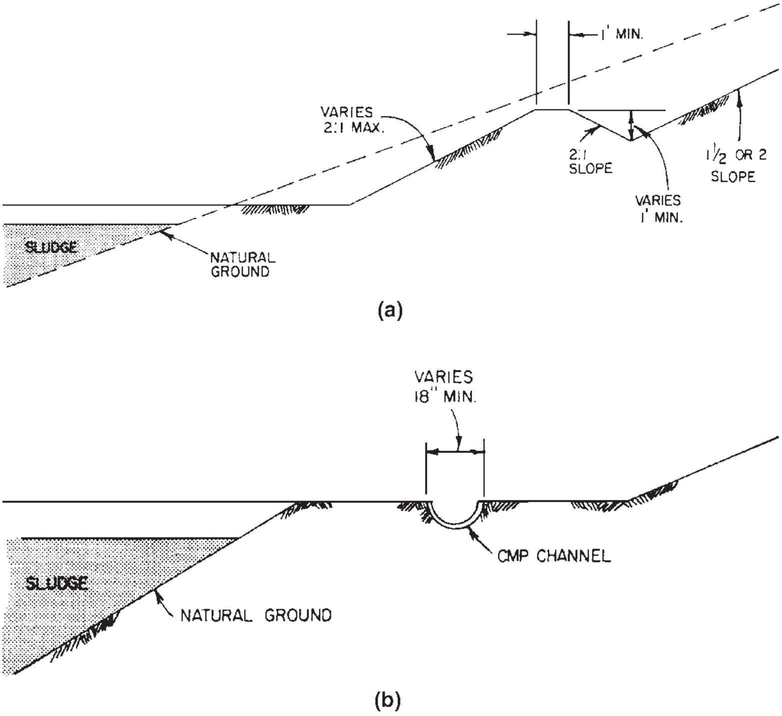


Figure 1.18 (*a*) Earthen run-on control system; (*b*) run-on control system constructed from corrugated metal pipe (CMP).

than 2 percent to promote runoff of precipitation and to inhibit ponding but less than 5 percent to reduce flow velocities and to minimize soil erosion. Siltation ponds may be constructed to settle solids contained in site runoff [8,9,13]. Straw bales, berms, and vegetation may supplement ponds or be used in conjunction with them to control runoff and siltation on the site.

1.3.4.6 Collection of leachate. *Leachate* is defined as the fluid generated from excess moisture contained in biosolids or from precipitation that percolates through the active biosolids surface-disposal unit (Fig. 1.19). If the active biosolids surface-disposal unit does not have a liner and leachate collection system, its operations are regulated through the pollutant limits (see Tables 1.28 and 1.29) as well as the previously described management practices. However, if the active biosolids surface-disposal unit has a liner and a leachate collection system, the owners/operators of the facility must follow two additional management practices.

First, for surface-disposal systems equipped with leachate collection systems, the leachate collection system must be operated and main-

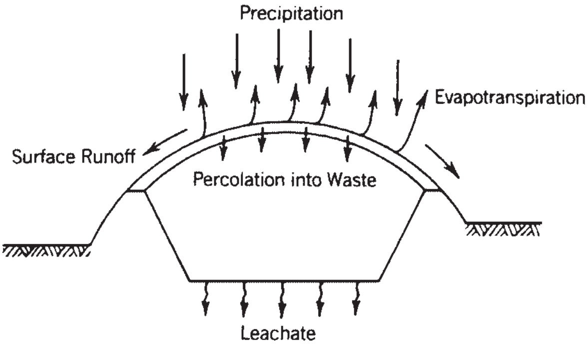


Figure 1.19 Schematic diagram illustrating the formation of leachate at a surface-disposal site (refs. [1,11]). (*Adapted by permission from A. Bagchi, Design, Construction, and Monitoring of Sanitary Landfill, Wiley, 1989.*)

tained according to design requirements and engineering recommendations [32,35]. The owners/operators of the surface-disposal site are responsible for ensuring that the leachate collection system is always operating according to design specifications and is maintained properly and routinely (e.g., system is periodically inspected to detect clogs and flushed to removed deposited solids). A schematic diagram of a leachate collection drain is given in Fig. 1.20. The second management practice requires that leachate be collected and disposed in accordance with applicable environmental requirements. For example, if leachate were to be discharged to surface water as a point source, then an NPDES permit is required. Alternatively, leachate may be used to irrigate adjacent land, or it may be discharged to a POTW collection system or treatment plant headworks. Both leachate collection management practices must be followed while the biosolids surface-disposal unit is active and then for 3 years after the unit is closed or for a longer period if required by the permitting authority [32].

1.3.4.7 Limitations on methane concentrations. For surface-disposal units that receive either daily or a final cover at closure, the 40 CFR Part 503 rule limits the concentration of methane in air in any structure within the property boundary as well as at the property line. Methane, which is an odorless and highly combustible gas, is generated under the anaerobic conditions that develop when biosolids are covered with soil or other material (e.g., geomembrane). To protect site personnel and the public from the risks of explosions, air must be monitored for methane gas continuously in all structures on the site and at the property line of the surface-disposal site.

For a surface-disposal site to be in compliance with this management practice, the methane gas concentration in any structure within the property line must be less than 25 percent of the *lower explosive limit*

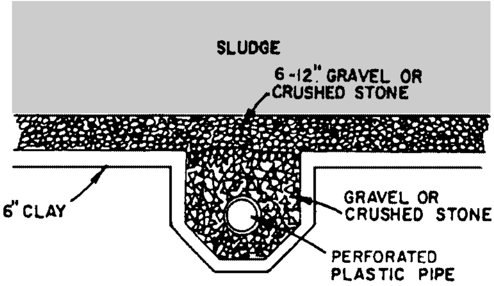


Figure 1.20 Schematic diagram of typical leachate collection drain.

(LEL), while the methane concentration at the property line cannot exceed the LEL. The LEL is defined as the lowest percentage (by volume) of methane gas in air that supports a flame at 25°C (77°F) and atmospheric pressure. Since the LEL for methane at these conditions is approximately 5 percent by volume (or 50,000 parts per million, ppmv), the air in any structure within the property line must not exceed 12,500 ppmv methane (i.e., $0.25 \cdot 50{,}000$ ppmv), while the methane concentration in air at the property line may not exceed 50,000 ppmv. If the continuous monitoring system indicates that methane gas concentrations exceed the regulatory limit in any structure or at the property line, the facility must employ engineering controls (e.g., venting systems, positive or negative air pressure systems, etc.) to reduce the methane concentration to permissible levels (Fig. 1.21).

To verify compliance with the methane levels at the property line, the permitting authority may determine that a methane-monitoring device at one downwind location on the property line is adequate because the wind patterns are consistent. However, where wind conditions at the site are highly variable, more than one device may be necessary to provide adequate protection.

Methane gas concentrations must be monitored continuously when the biosolids surface-disposal units are active and for 3 years after the last active biosolids surface-disposal unit on the site is closed. If unstabilized biosolids are disposed at a site, the permitting authority may require methane gas to be monitored for a period of longer than 3 years after closure because of the higher methane generation potential of unstabilized biosolids [32].

1.3.4.8 Restrictions on crop production. Food, feed, or fiber crops may not be grown on an active biosolid surface-disposal unit unless the owner/operator of the site can demonstrate to the permitting authority that through specific management practices, public health and the environment are protected from any reasonably anticipated adverse effect from biosolids pollutants. These specific management practices

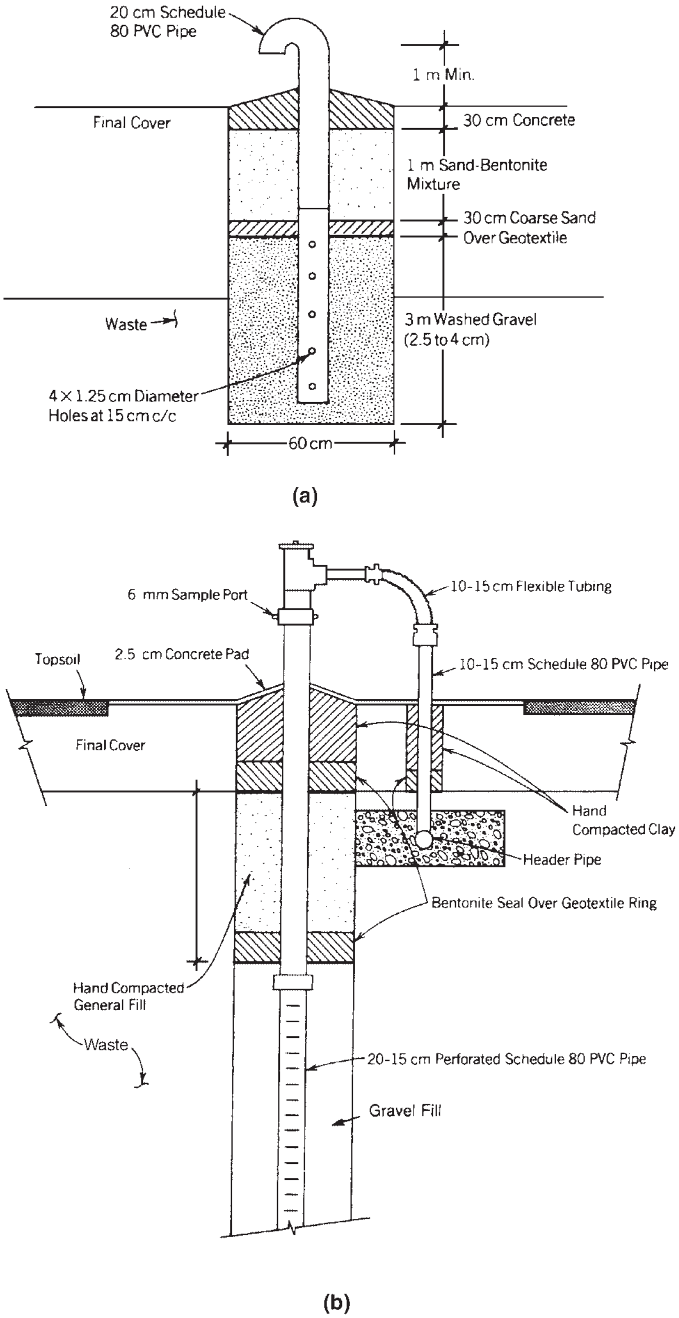


Figure 1.21 Schematic diagrams of (*a*) passive methane venting system and (*b*) active methane venting system (refs. [1,7,11]). (*Adapted by permission from A. Bagchi, Design, Construction, and Monitoring of Sanitary Landfill, Wiley, 1989.*)

may include testing crops and animal tissue for the presence of pollutants if animal feed is produced or the establishment of a monitoring schedule for the crops and/or any animal feed products derived from crops grown on the site [24,32].

1.3.4.9 Restrictions on grazing. Under the 40 CFR Part 503 rule, animals may not be grazed on an active biosolids surface-disposal unit unless the owner/operator of the site can demonstrate to the permitting authority that public health and the environment are protected from reasonably anticipated adverse effects [24,32]. Permits allowing animal grazing on biosolids surface-disposal sites typically include specific management practices such as the monitoring of biosolids pollutants in all animal products (e.g., dairy or meat). This grazing management practice ensures that unsafe levels of pollutants do not find their way into the human food chain. A biosolids surface-disposal site where a special permit allows for the production of crops and/or grazing of animals is designated as a *dedicated beneficial-use surface-disposal site.*

1.3.4.10 Restrictions on public access. The 40 CFR Part 503 rule restricts public access to a biosolids surface-disposal site during its active life as well as for 3 years after the last active biosolids surface-disposal unit has been closed. Restrictions on public access minimize public contact with any pollutants, including pathogens, that may be present in biosolids discharged to an active biosolids surface-disposal unit.

Fencing off an area and installing gates that lock are approaches that may be used to restrict public access to surface-disposal sites in densely populated areas, whereas natural barriers such as hedges, trees, embankments, and ditches, along with warning signs, may be adequate in less populated areas. In remote areas, it may be sufficient to post warning signs (e.g., do not enter, no trespassing, etc.) to restrict public access to biosolids surface-disposal sites [27,29].

1.3.4.11 Protection of groundwater. The 40 CFR Part 503 rule requires that biosolids discharged to an active biosolids surface-disposal unit must not contaminate an aquifer. An *aquifer* is defined by the USEPA as an area below the ground that can yield groundwater in sufficient quantities to supply wells or springs [24,29]. Moreover, according to the 40 CFR Part 503 rule, the contamination of an aquifer only refers to the introduction of any substance that can cause the level of nitrate-nitrogen in groundwater to increase above the maximum contaminant level (MCL) of 10 mg/liter or that can cause an increase to an existing exceedance of the groundwater MCL for nitrate-nitrogen.

To comply with this management practice, the owner/operator of the biosolids surface-disposal site must verify that the groundwater has not become contaminated due to the discharge of biosolids to the facility. Verification may be accomplished by either establishing a groundwater monitoring program developed by a qualified groundwater scientist or by obtaining a certification by a groundwater scientist that the groundwater will not be contaminated by the disposal of biosolids at the site.

Certification that biosolids surface disposal will not cause groundwater contamination is an option only if the site has a liner and a leachate collection system. In most cases where biosolids are discharged to a surface-disposal site without a liner and leachate collection system, establishing a groundwater monitoring program is required. A schematic diagram of a groundwater monitoring well typically used at surface-disposal sites is given in Fig. 1.22. In some circumstances (e.g., when the depth to groundwater is considerable and there is a natural clay layer under the soil or the rate at which biosolids are placed on the site is small, i.e., the agronomic rate), it may be possible to obtain certification that the biosolids surface-disposal site will not cause groundwater contamination despite the absence of a liner and leachate collection system [24,32,35].

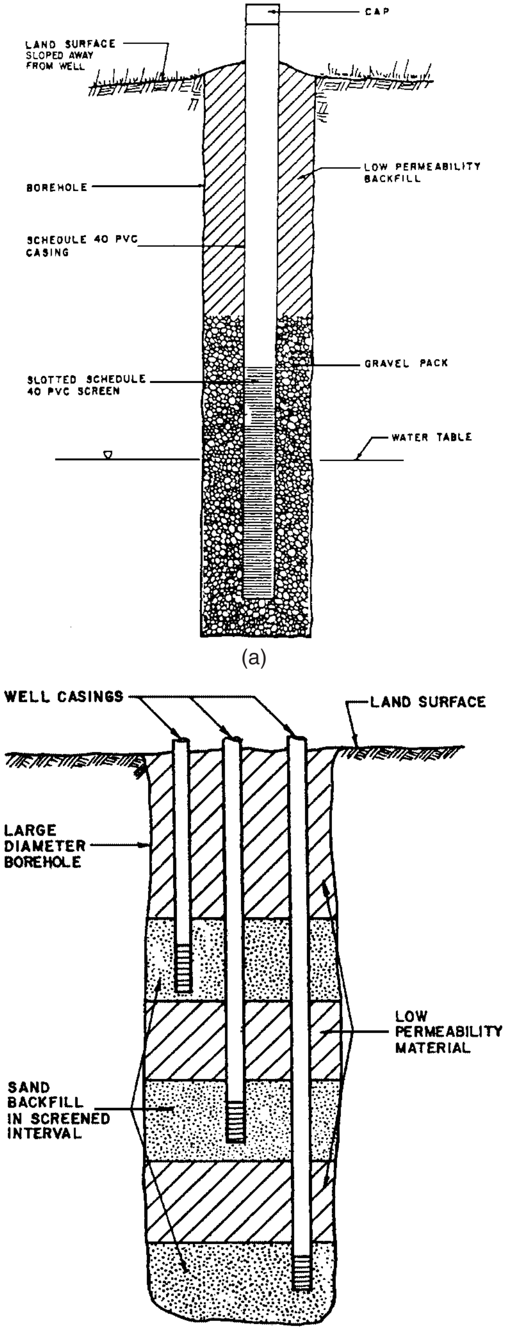
1.3.5 Pathogen and vector attraction reduction requirements

According to the 40 CFR Part 503 rule, biosolids can be discharged to an active biosolids surface-disposal unit only if certain pathogen and vector attraction reduction requirements are met. For pathogen reduction, the biosolids placed on an active biosolids surface-disposal unit must meet either (1) Class A or Class B pathogen-reduction requirements or (2) a cover (soil or other material) must be placed over the discharged biosolids at the end of each day.

For vector attraction reduction, one of the options in Table 1.31 must be met. In most cases, owners or operators of biosolids surface-disposal sites will place a daily cover on the unit to meet pathogen and vector attraction reduction requirements [32,34].

1.3.6 Frequency of monitoring

According to the 40 CFR Part 503 rule, the parameters listed in Table 1.32 must be monitored at biosolids surface-disposal sites. Monitoring of arsenic, chromium, and nickel in biosolids is required only at biosolids surface-disposal sites that are not equipped with liners or leachate collection systems. Compliance monitoring for pathogen reduction, vector attraction reduction requirement, and methane in any structure and at the property line at a covered surface-disposal site is required for both lined and unlined units.



(a)

(b)

Figure 1.22 Schematic diagram of groundwater monitoring systems: (*a*) single well; (*b*) nested-well system.

TABLE 1.31 Available Options for Meeting Pathogen and Vector Attraction Reduction Requirement for Surface Disposal of Biosolids*

Pathogens (must meet one of these)

- Place a daily cover on the active biosolids unit.
- Meet one of the six class A pathogen-reduction alternatives (see Sec. 1.2.4.1).
- Meet one of the three class B pathogen reduction alternatives (see Sec. 1.2.4.2).

Vector attraction reduction requirements

- Place a daily cover on the active biosolids unit.
- Reduce volatile solids content by a minimum of 38 percent or less under specific laboratory test conditions with anaerobically or aerobically digested biosolids.
- Meet a specific oxygen uptake rate (SOUR).
- Treat the biosolids in an aerobic process for a specified number of days at a specified temperature.
- Raise the pH of the biosolids with an alkaline material to a specified level for a specified time.
- Meet a minimum percent solids content.
- Inject or incorporate the biosolids into soil.

*Adapted from ref. [34].

The required frequency of monitoring for metals, pathogen reduction, and vector attraction reduction is determined by the annual amount of biosolids disposed at the site. POTWs disposing zero to less than 290 dry metric tons (dmt) of biosolids annually must test once per year, those disposing 290 to less than 1500 dmt annually must test once per quarter, those disposing 1500 to 15,000 dmt annually must test once per 60 days (six times per year), and those disposing 15,000 dmt or more annually must test once per month. The permitting authority may require more frequent monitoring, for example, if the pollutant and pathogen levels in the biosolids vary significantly [24,32].

After biosolids have been monitored for 2 years, the permitting authority may reduce the frequency of monitoring for arsenic, chromium, nickel, and, under limited circumstances, pathogens [24,32]. The monitoring frequency may be reduced, for example, if the pollutant levels in the biosolids do not vary significantly or if pathogens are never detected. However, at a minimum, monitoring for metals (for unlined sites), pathogens, and vector attraction reduction must be performed once per year.

For biosolids surface-disposal sites that are covered, methane concentrations must be monitored continuously both at the property line of the surface-disposal site and within each structure on the property. The continuous monitoring of methane must be maintained as long as any covered biosolids surface-disposal unit on the site is active and then for at least 3 years after the last biosolids surface-disposal unit has been closed.

TABLE 1.32 Parameters That Must Be Monitored at Surface Disposal Sites as Mandated by 40 CFR Part 503 Subpart C*

Parameter to monitored	Medium to be monitored
Arsenic	Biosolids
Chromium	Biosolids
Nickel	Biosolids
Pathogens	Biosolids for several options
Vector attraction reduction	Biosolids for several options
Methane gas	Air in each structure on site
Methane gas	Air at surface-disposal site property boundary

*Adapted from refs. [24,32].

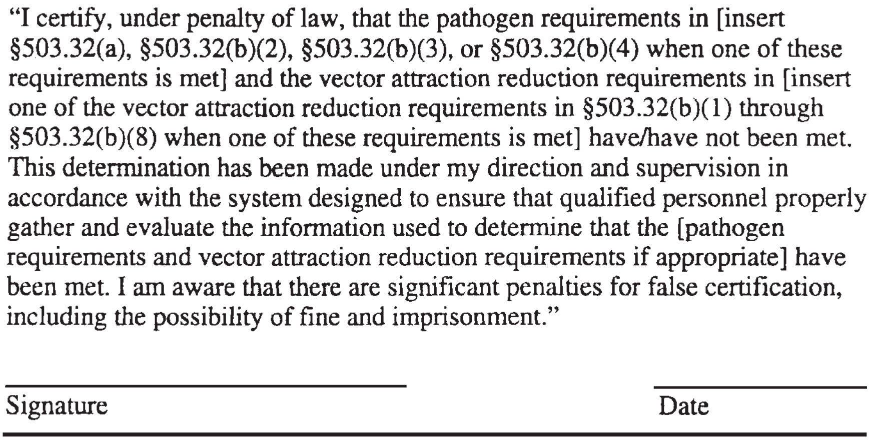
1.3.7 Recordkeeping requirements for surface-disposal sites

Under the 40 CFR Part 503 rule, certain information regarding the operation and maintenance of a surface-disposal unit must be recorded and retained for 5 years from the time that biosolids are first discharged to the facility. A separate set of records must be maintained by the biosolids preparer, and another set must be kept by the owner/operator of a surface-disposal site. The biosolids preparer must develop and retain the following records for 5 years:

1. The concentrations of arsenic, chromium, and nickel in biosolids for surface-disposal units not equipped with a liner and leachate collection system when the distance between the unit's boundary and the property line is 150 m (500 ft) or more.
2. A certification that pathogen and vector attraction reduction requirements have been met (Fig. 1.23).
3. Descriptions of how pathogen and vector attraction reduction requirements were achieved when the vector attraction reduction requirement was not met using subsurface injection, incorporation, or a daily cover.

Similarly, the owner/operator of a surface-disposal site on which biosolids have been discharged must develop and keep the following records for 5 years:

1. The concentrations of arsenic, chromium, and nickel in biosolids discharged to an active biosolids surface-disposal unit with a boundary less than 150 m (500 ft) from the facility's property line or for an active biosolids surface-disposal unit that is regulated by site-specific pollutant limits.



"I certify, under penalty of law, that the pathogen requirements in [insert §503.32(a), §503.32(b)(2), §503.32(b)(3), or §503.32(b)(4) when one of these requirements is met] and the vector attraction reduction requirements in [insert one of the vector attraction reduction requirements in §503.32(b)(1) through §503.32(b)(8) when one of these requirements is met] have/have not been met. This determination has been made under my direction and supervision in accordance with the system designed to ensure that qualified personnel properly gather and evaluate the information used to determine that the [pathogen requirements and vector attraction reduction requirements if appropriate] have been met. I am aware that there are significant penalties for false certification, including the possibility of fine and imprisonment."

Signature Date

Figure 1.23 Statement certifying that pathogen and vector attraction reduction requirements have been met. Record is maintained by the biosolids preparer.

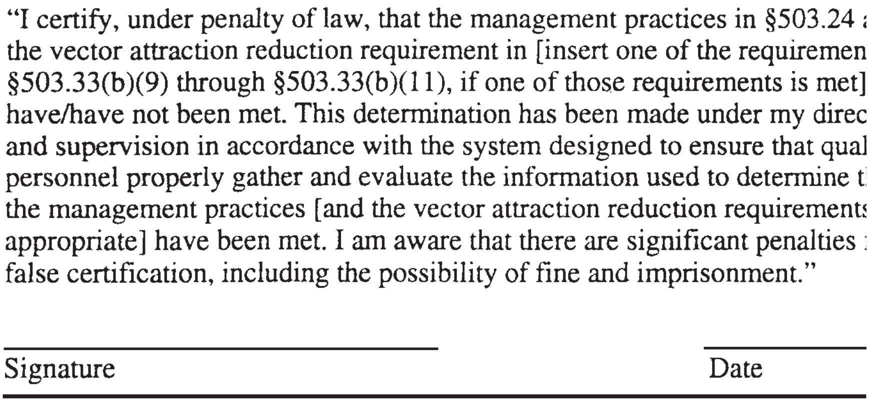
2. A certification that management practice requirements have been met together with descriptions of those practices (Fig. 1.24).
3. A certification that vector attraction reduction requirements have been met if injection, incorporation, or daily cover has been used to meet these requirements and a description of those processes.

1.3.8 Reporting requirements for surface-disposal sites

The 40 CFR Part 503 rule includes a reporting requirement only for those facilities designated as Class I (i.e., those which are required to have a permitted industrial pretreatment program) or for those facilities which process at least 1 million gallons of wastewater per day (1 MGD). Facilities that fall into at least one of these categories must present the information developed for recordkeeping purposes to the permitting authority by February 19 of each year [21,30,32].

1.3.9 Regulatory requirements for surface disposal of domestic septage

The regulatory requirements for the surface disposal of septage are not as extensive as the requirements for the surface disposal of biosolids. Although the management practices for the surface disposal of domestic septage are the same as the management practices required for the surface disposal of biosolids, the options for meeting the vector attraction reduction requirements of domestic septage



"I certify, under penalty of law, that the management practices in §503.24
the vector attraction reduction requirement in [insert one of the requiremen
§503.33(b)(9) through §503.33(b)(11), if one of those requirements is met]
have/have not been met. This determination has been made under my direc
and supervision in accordance with the system designed to ensure that qual
personnel properly gather and evaluate the information used to determine t
the management practices [and the vector attraction reduction requirement
appropriate] have been met. I am aware that there are significant penalties
false certification, including the possibility of fine and imprisonment."

Signature Date

Figure 1.24 Typical statement found in documents certifying that management practices have been met. Record is maintained by the owner/operator of the surface-disposal site.

include any one of the following: (1) injecting or incorporating (i.e., plowing) of domestic septage into soil, (2) raising the septage to a pH of 12 for 30 minutes by the addition of alkali, and (3) placing a daily cover over discharged biosolids. There are no pathogen reduction requirements for the surface disposal of domestic septage [21,30,32].

The owner/operator of the site where domestic septage is disposed must certify and describe the management practices and vector attraction reduction methods used if injection, incorporation, or daily cover is relied on to meet these requirements. Moreover, domestic septage haulers that use pH adjustment to meet vector attraction reduction requirements must certify and describe the procedure [30,32].

Domestic septage requires monitoring only if pH adjustment is chosen to meet vector attraction reduction requirements. Each container of domestic septage to which alkali has been added must be tested for pH before it is disposed to ensure that vector attraction reduction requirements are met. The applier of domestic septage to the surface-disposal site must certify that vector attraction reduction has been achieved and develop a description of how it was achieved. The certification and description must be kept on file for a minimum of 5 years (Fig. 1.25).

1.4 Incineration

Biosolids incineration generally is defined as the high-temperature combustion of biosolids within an enclosed vessel. Unlike land application and surface disposal, biosolids incineration takes advantage of the inherent energy value of the solids fraction of biosolids. The per capita energy value of biosolids may be estimated by noting that, on average, each per-

"I certify, under penalty of law, that the vector attraction reduction requirements in §503.33(b)(12) have/have not been met. This determination has been made under my direction and supervision in accordance with the system designed to ensure that qualified personnel properly gather and evaluate the information used to determine that the vector attraction requirements have been met. I am aware that there are significant penalties for false certification, including the possibility of fine and imprisonment."

(a)

"I certify, under penalty of law, that the management practices in §503.24 and the vector attraction reduction requirements in [insert §503.33(b)(9) through §503.33(b)(11) when one of those requirements is met] have/have not been met. This determination has been made under my direction and supervision in accordance with the system designed to ensure that qualified personnel properly gather and evaluate the information used to determine that the management practices [and the vector attraction requirements, if appropriate] have been met. I am aware that there are significant penalties for false certification, including the possibility of fine and imprisonment."

(b)

Figure 1.25 Statements certifying that vector attraction reduction requirements have been met for surface disposal of domestic septage: (*a*) statement to be kept by individual who disposes of septage; (*b*) statement to be kept by the owner/operator of the facility.

son generates approximately 19 kg of organic matter each year as biosolids. Since 1 kg of organic dry matter has an energy value of approximately 7 kW, the theoretical per capita energy value of biosolids is 133 kW (19·7 kW) per year. To effectively use this energy during biosolids incineration, the heating value of the biosolids feed must be optimized. Approaches to improving the heating value of the feed biosolids include (1) moisture removal through biosolids dewatering (and drying) and/or (2) addition of a combustible material (e.g., auxiliary fuel).

In addition to energy recovery, incineration of biosolids results in (1) significantly reducing the final volume of solids to be disposed (*ca.* 80 to 95 percent volume reduction is typical) and (2) destruction of toxic organic compounds. POTWs should consider biosolids by incineration in situations in which suitable land for land application or surface disposal of biosolids is scarce, stringent requirements for biosolids land disposal exist, and/or destruction of toxic materials is required [30].

Biosolids incineration in the United States is currently regulated under the 40 CFR Part 503 (Subpart E) rule. It is important to note that the 40 CFR Part 503 (Subpart E) rule does not regulate the incineration of (1) industrial biosolids, (2) hazardous biosolids, (3) sludges with high PCB concentrations, (4) grit, (5) screenings, (6) drinking water treatment sludges, or (7) commercial/industrial septage.

1.4.1 Use of auxiliary fuels

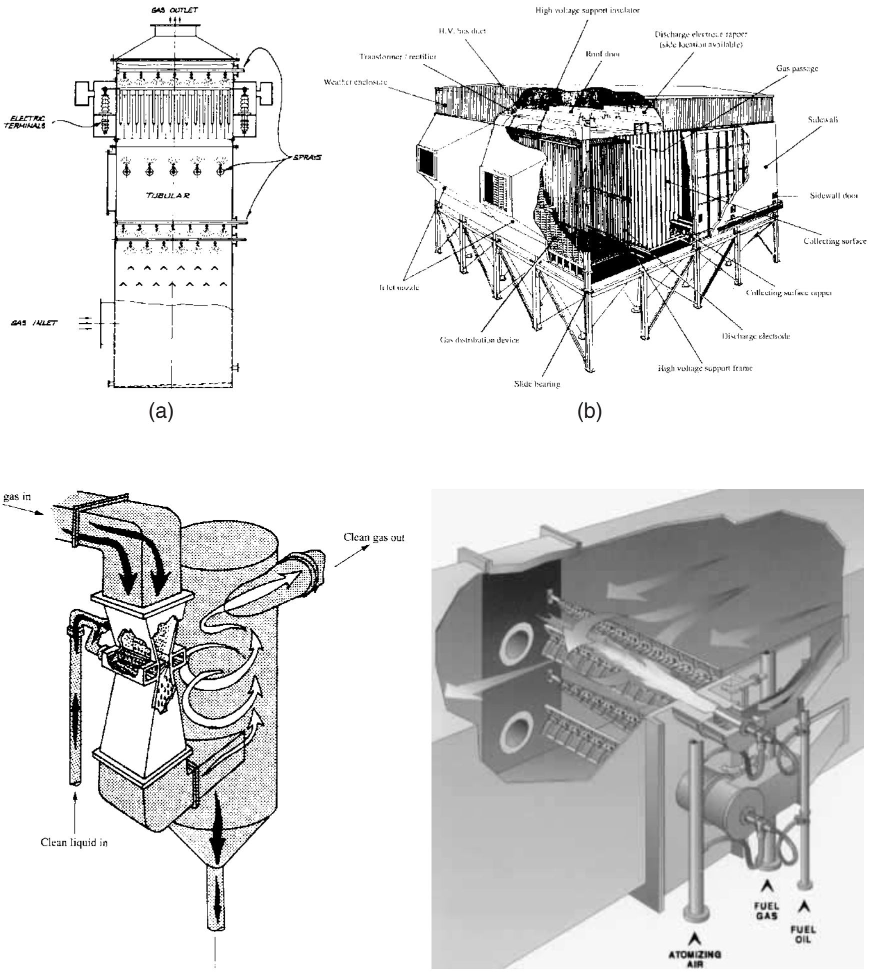
The addition of auxiliary fuels (e.g., woodchips, coal, municipal solid wastes, etc.) is often used to increase the efficiency of biosolids incineration by either increasing the organic matter content of the biosolids feed and/or reducing its overall moisture content. Although the cofiring of biosolids and municipal solid wastes (MSWs) is a common practice in metropolitan areas, the 40 CFR Part 503 rule specifies that if MSW accounts for more than 30 percent (dry weight) of the feed material, the MSW is no longer considered auxiliary fuel. Moreover, a biosolids incinerator that is firing MSW at greater than 30 percent of its feed composition would no longer be regulated under the 40 CFR Part 503 rule but rather would fall under the jurisdiction of the municipal waste incineration regulations (i.e., 40 CFR Parts 60 and 61). In addition to the limitations on the percentage of MSW fired to a biosolids incinerator, hazardous waste is not considered auxiliary fuel under the 40 CFR Part 503 rule. An incinerator that receives any hazardous wastes (either with or without biosolids) is considered a hazardous waste incinerator and must operate in accordance with the Resource Conservation and Recovery Act (RCRA) Subtitle C regulations (i.e., 40 CFR Parts 261–268).

1.4.2 Biosolids incineration systems

Incineration systems generally consist of an incinerator (furnace) and one or more air pollution control devices. Most air pollution control devices are used either to remove particulate matter in the exhaust gas or to increase the combustion efficiency of the organic matter. Examples of particulate-removing air pollution control devices include wet scrubbers, electrostatic precipitators, and fabric filters. Afterburners, another type of air pollution control device, are used to burn organics in exhaust gases more efficiently. Examples of these devices are illustrated in Fig. 1.26.

The most commonly used biosolids incinerator systems are the (1) multiple-hearth furnace, (2) cyclonic furnace, (3) fluidized-bed furnace, and (4) electric infrared furnace. Each of these systems is described briefly in the following sections. Readers interested in the design of these systems are referred to the following reference [36].

1.4.2.1 Multiple-hearth furnace (MHF). The MHF is a vertically oriented, cylindrically shaped steel vessel equipped with a series of horizontally positioned refractory brick hearths aligned one above the other. MHFs are available with diameters ranging from 1.4 m to over 8.8 m (4.5–29 ft) and generally have from 4 to 14 hearths [16,36]. The MHF is durable and can handle significant fluctuations in both feed



(a) (b)

(c) (d)

Figure 1.26 Emission control devices. (*a*) Wet electrostatic precipitator (*courtesy of Water Environment Federation*). (*b*) Dry electrostatic precipitator (*courtesy of Institute of Clean Air Companies*). (*c*) Venturi scrubber (followed by cyclone) (*courtesy of Wheelabrator Air Pollution Control, Inc.*). (*d*) Afterburner (*courtesy of Coen Company, Inc.*).

quality and loading rates [16,36]. A cross section of a typical MHF is shown in Fig. 1.27.

Within the MHF, a hollow central shaft that extends from the bottom to the top of the furnace supports two or four rabble arms per hearth. Each rabble arm contains several rabble teeth (or plows) that rake the biosolids across the hearth in a spiral pattern. To provide effective solids movement within each hearth, the central shaft is rotated at speeds ranging from approximately 0.3 to 3.0 revolutions per minute (rpm).

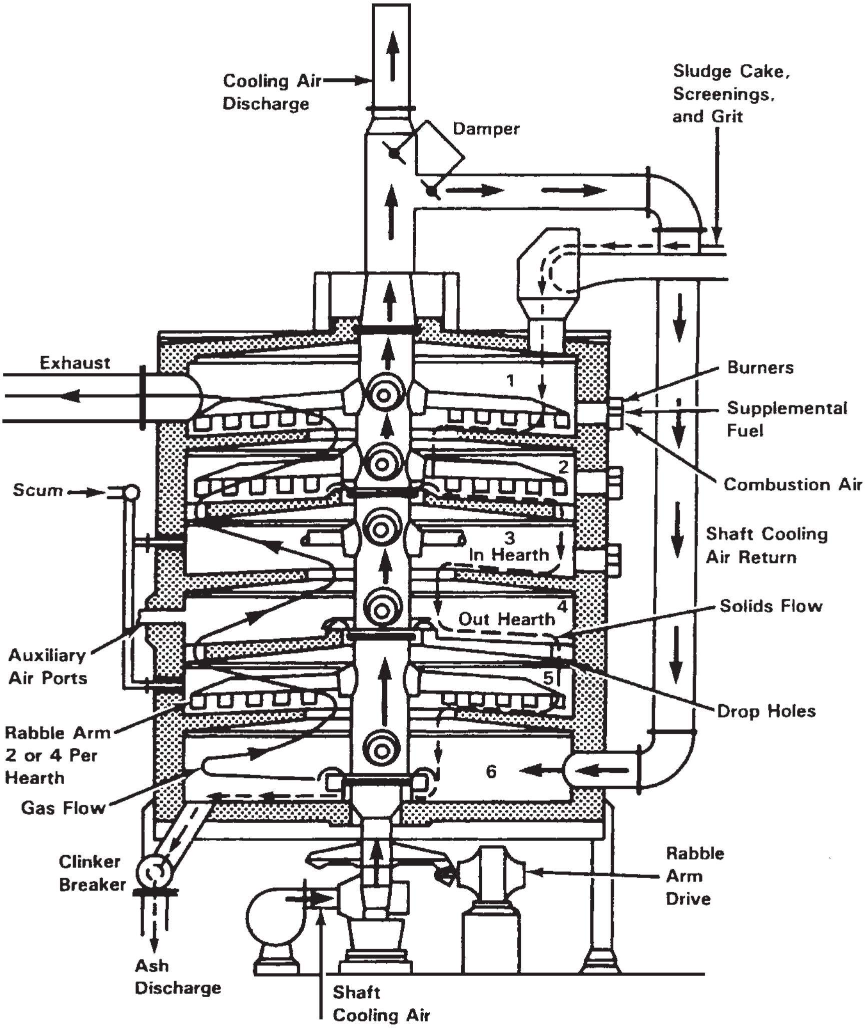


Figure 1.27 Schematic diagram of a multiple-hearth furnace.

During normal operation, biosolids are fed at the periphery of the top hearth and are rabbled toward the center, where they drop to the hearth below. On the second hearth, the biosolids are rabbled outward to the holes at the periphery, where they drop to the next hearth. The alternating drop-hole locations on each hearth together with the countercurrent flow of the rising exhaust gases ensures effective heat transfer within the incinerator system (Fig. 1.28).

Theoretically, the MHF can be divided into four operational zones, as illustrated in Fig. 1.29. The sequence of these operational

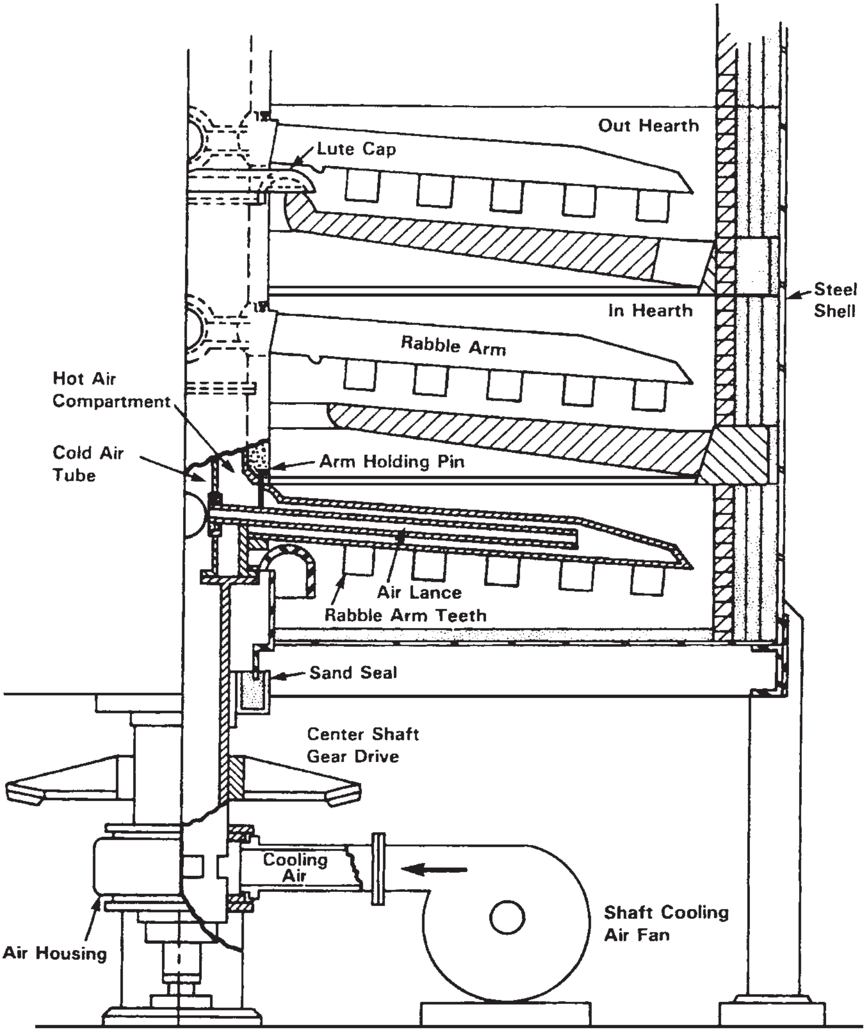


Figure 1.28 Schematic diagram of the rabble arm in a multiple-hearth furnace.

zones is always the same regardless as to the size or shape of the furnace [16]. The first zone, which is termed the *drying zone,* consists of the upper hearths. In the drying zone, the exhaust gas temperature is in the range of 600 to 900°F (315–480°C), which ensures rapid removal of any residual moisture from the feed biosolids. The second zone, which generally consists of the central hearths, is categorized as the *combustion zone.* In this zone, the majority of the combustibles are burned at temperatures in the range of 760 to 927°C (1400–1700°F).

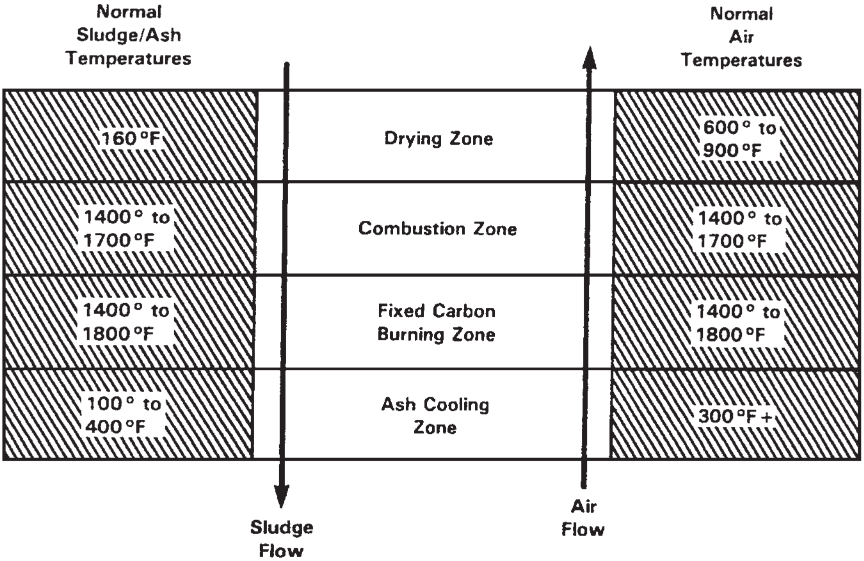


Figure 1.29 Treatment zones in a multiple-hearth furnace incinerator system.

When the heating value of the biosolids is insufficient to sustain autogenous (i.e., spontaneous) combustion, additional heat is supplied by firing supplemental fuel (e.g., natural gas) through burners located at various points within the furnace wall (Fig. 1.30). The third zone, which is the *fixed carbon burning zone,* is maintained at a temperature ranging from 1400 to 1800°F (760–980°C), which is suitable for oxidizing any remaining carbon to carbon dioxide (CO_2). Finally, the fourth zone, which is termed the *cooling zone,* includes the lower hearths. In the cooling zone, the incoming combustion air is typically preheated while the incinerator ash is cooled.

Given the length of time and fuel requirements needed for the hearths and internal equipment to achieve operational temperature from a cold condition, MHFs are best suited for continuous operation. Generally, the temperature of the MHF is maintained at "hot standby" (approximately 800°F, or 427°C) using auxiliary fuel during biosolids feed stoppages [16,36].

1.4.2.2 Cyclonic furnace. The cyclonic furnace is a refractory-lined cylindrical vessel equipped with a single hearth (Fig. 1.31). In contrast to the MHF, the cyclonic furnace hearth rotates while the rabble teeth remain stationary. During normal operation, dewatered biosolids are fed into the furnace with a screw feeder and deposited on the periphery of the rotating hearth. If the feed biosolids are fluid (i.e., below 15

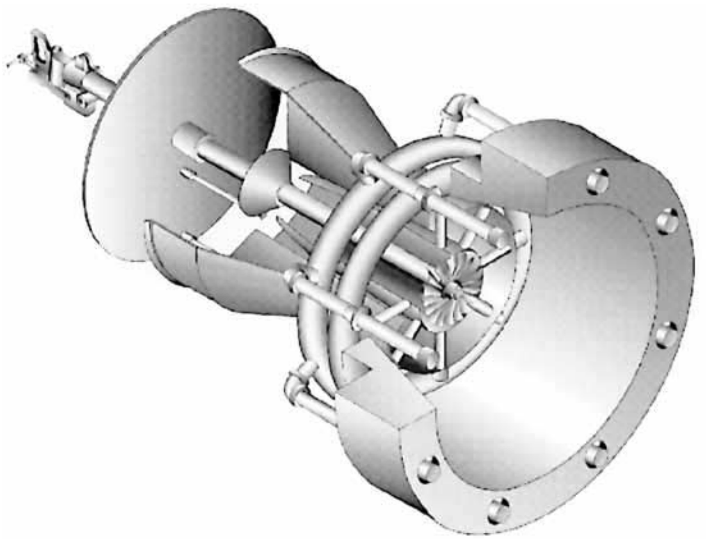


Figure 1.30 Diagram of venturi-type burner used to burn auxiliary fuel. (*Courtesy of Coen Company, Inc.*)

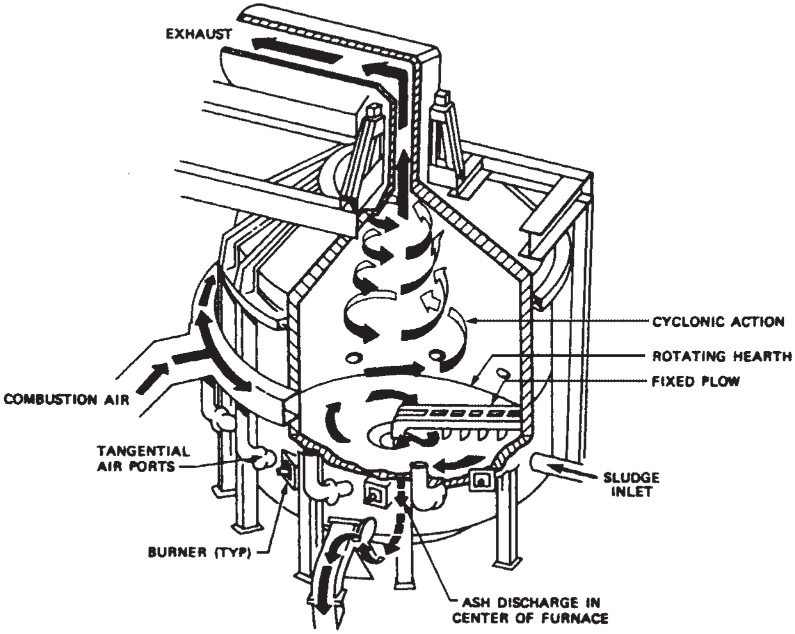


Figure 1.31 Schematic diagram of a cyclone furnace. (*Courtesy of Water Environment Federation.*)

percent solids content), a progressive cavity pump also may be used to discharge biosolids to the incinerator. As the combustion process progresses, biosolids are rabbled toward the center of the hearth, where, as ash, they are discharged.

Combustion air, which is introduced through tangential burner ports located on the shell of the furnace, creates turbulence that ensures effective mixing and heat transfer within the incinerator system. The exhaust gases are directed vertically in a spiral pattern through the discharge flue located in the center of the domed roof.

The temperature within the cyclonic furnace is maintained within the range of 1500 to 1600°F (820–870°C) using auxiliary fuel (if necessary). Cyclonic furnaces are relatively small and can be installed and normally made operational within an hour [36]. Because of their small size, cyclonic furnaces are best suited for installation at small wastewater treatment plants (i.e., those treating 2-MGD influent wastewater flows or less).

1.4.2.3 Fluidized-bed incinerators. The fluidized-bed incinerator is a vertically oriented, cylindrically shaped, refractory-lined steel vessel that contains a sand bed (Fig. 1.32). Bed material is commonly silica sand but also may be comprised of limestone, alumina, or ceramic material [16]. The fluidized-bed incinerator is operated with a continuous flow of fluidizing air that is introduced through ports called *tuyeres*. Although the bed normally expands 30 to 60 percent in volume when fluidized with air at a velocity of approximately 2 to 3 ft/s (0.6–0.9 m/s), in some applications, bed expansion can be as large as 200 percent [16].

The fluidized-bed incinerator normally is available in sizes ranging from 2.7 m to more than 7.6 m (9–25 ft) in diameter [16,36]. The sand bed is approximately 0.8 m (2.5 ft) thick and sits on a refractory-lined grid. The grid contains the tuyeres through which the air is injected into the furnace at pressures ranging from 21 to 34 kN/m^2 gauge (3–5 psig). At these influent air pressures, the sand undulates and has the appearance of a fluid in motion.

There are two basic incinerator designs that currently use the fluidized-bed furnace. In the hot windbox (HWB) design, the fluidizing air passes through the heat exchanger (or recuperator) prior to injection into the combustion chamber, whereas in the cold windbox (CWB) design, the fluidizing air is injected directly into the furnace. A schematic diagram of each of the fluidized-bed designs is provided in Fig. 1.33.

By preheating the combustion air, the HWB design has improved thermal efficiency and lowered auxiliary fuel costs compared with the

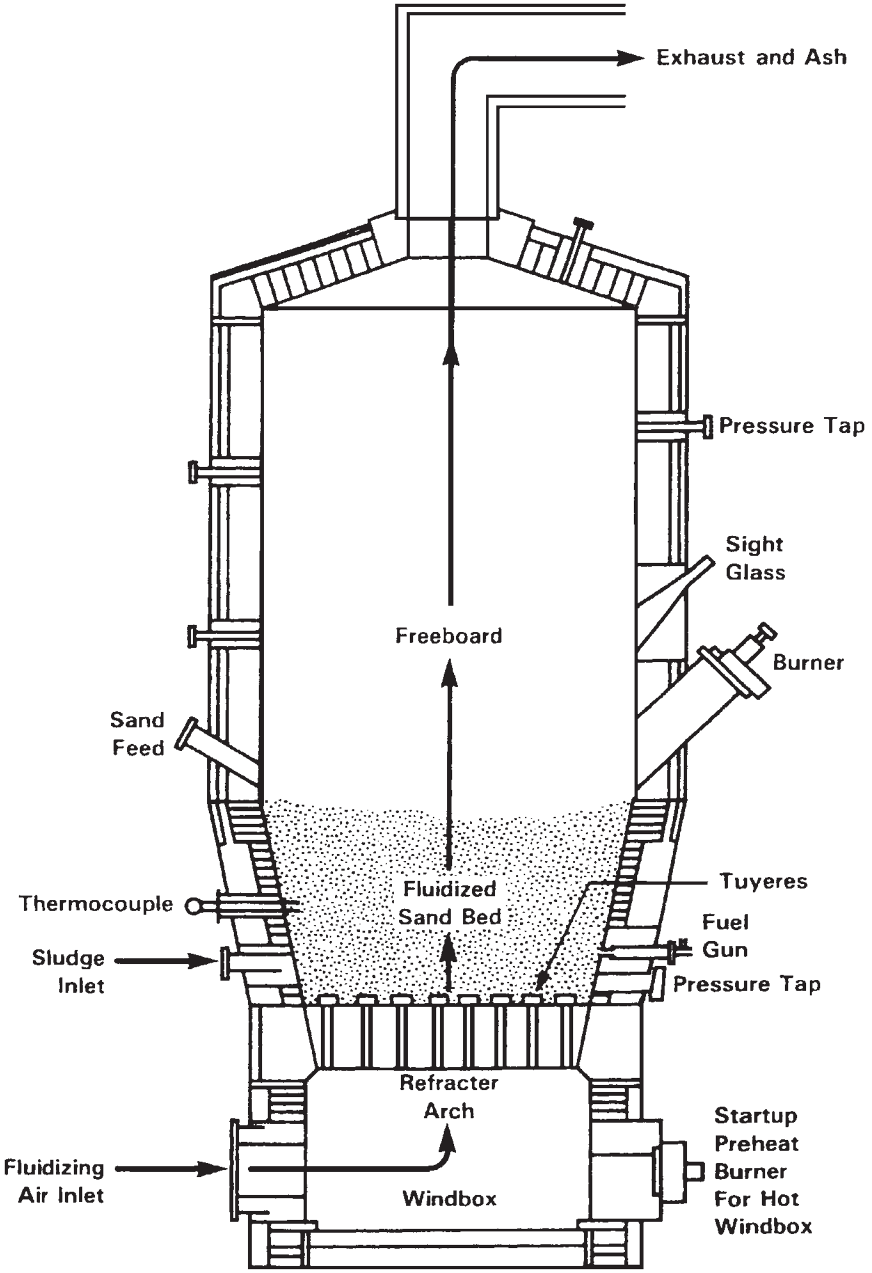


Figure 1.32 Schematic diagram of a fluidized-bed incinerator system.

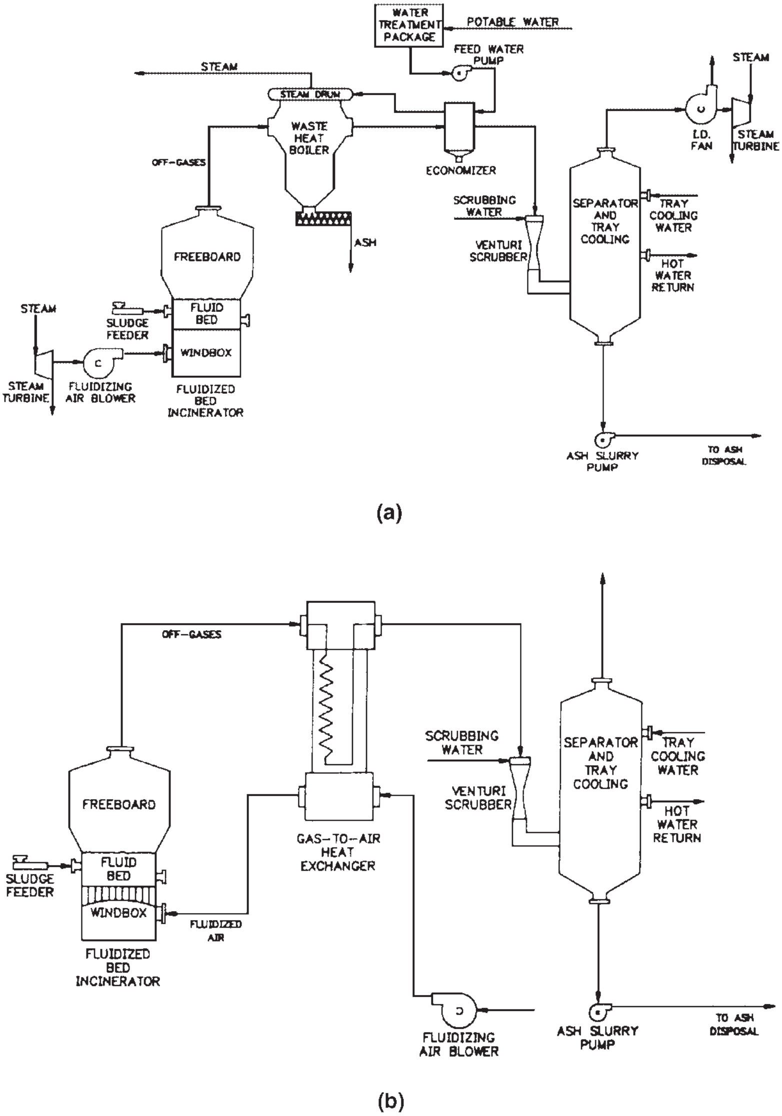


Figure 1.33 (*a*) Schematic diagram of cold windbox design; (*b*) schematic diagram of hot windbox design.

CWB design. However, the additional duct work and fans required by the HWB design add substantially to its capital costs [16].

During normal operation, the sand bed is maintained at temperatures of approximately 760 to 816°C (1400–1500°F). In addition to the sand bed, the large volume (i.e., freeboard) above the sand is maintained at temperatures in the range of 1500 to 1600°F (816–870°C). At these temperatures, an air residence time on the order of a few seconds is normally sufficient to obtain complete combustion of biosolids and odor elimination. In some fluidized-bed systems, a water spray or heat-removal system in the bed assists in controlling the furnace temperature [16,36].

Violent mixing in the fluidized bed ensures rapid and uniform distribution of fuel together with effective heat transfer and combustion. Moreover, the turbulent motion of the bed comminutes the ash material, minimizing the accumulation of clinkers [16,36]. The resulting fine ash is constantly stripped from the bed by the upflowing gases. Unfortunately, in addition to ash removal, the fluidizing airflow continuously removes a fraction of the sand medium. Sand losses (which must be replaced) are typically 5 percent of the bed volume for every 300 hours of operation [16]. To ensure effective removal of ash and elutriated sand from the exhaust gas stream, high-energy venturi scrubbing systems typically are employed with fluidized-bed incinerators. Fluidized-bed incineration systems generally require the use of venturi scrubbers rated with pressure drops of between 30 and 60 in H_2O (7–15 kPa) [16,36]. Figure 1.34 depicts a full-scale HWB fluidized-bed biosolids incinerator system.

Although the airflow rate in fluidized-bed incinerators must be sufficient to both expand the bed to a proper density and maintain the minimum oxygen requirements to ensure complete oxidation of all combustible materials, excessive airflow rates must be avoided. Excessive airflow rates not only deplete stored heat energy (which increases auxiliary fuel consumption) but also result in transferring

Figure 1.34 Full-scale hot windbox fluidized-bed incinerator system. (*Courtesy of Infilco-Degremont, Inc.*)

both sand and incomplete combustion products into the exhaust gas. Under normal operating conditions, the quantity of air added to the fluidized-bed furnace is adjusted to maintain approximately 20 to 45 percent excess oxygen [16].

1.4.2.4 Electric furnace incinerator. The electric furnace incinerator is a horizontally oriented, rectangular steel vessel containing a moving horizontal woven-wire belt. Electric furnace incinerators are available in sizes ranging from 1.2 m (4 ft) wide by 6.1 m (20 ft) long to greater than 2.9 m (9.5 ft) wide by 29.3 m (96 ft) long [36]. The electric furnace incinerator is divided into three zones: (1) feed zone, (2) drying and combustion zone, and (3) ash discharge zone. The length of each zone varies depending on the specific design. A typical cross-sectional view of an electric furnace incinerator is shown in Fig. 1.35.

During normal operation, dewatered and/or dried biosolids are fed into the electric furnace incinerator through a feed hopper that discharges onto a nickel-chromium–plated woven-wire belt [36]. Shortly after the biosolids are deposited on the belt, they are leveled by means of an internal roller to a layer approximately 2.5 cm (1 in) thick across the belt width. The biosolids are then subjected to a series of infrared heating elements, which provide supplemental energy for moisture removal and combustion.

After drying, the biosolids undergo combustion at temperatures ranging from 1300 to 1600°F (700–870°C). Combustion air flows countercurrently relative to biosolids conveyance, with most of the combustion air being introduced into the ash discharge end of the unit. Excess airflow rates for electric furnace incinerators vary from 29 to 70 percent [36]. Following biosolids combustion, the resulting ash is discharged from the end of the belt to the ash handling system.

Low capital costs combined with modular construction make the electric furnace an attractive incinerator design [36]. Moreover,

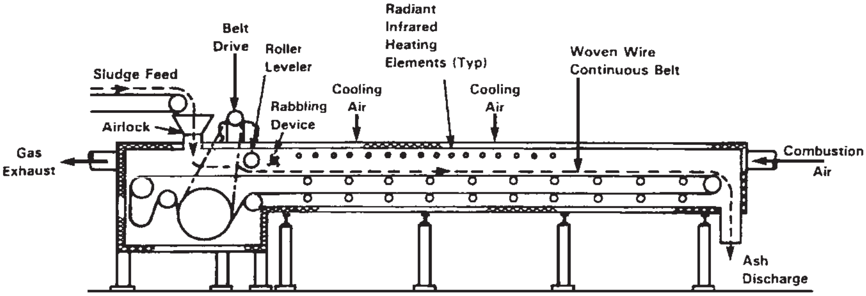


Figure 1.35 Schematic diagram of an electric furnace incinerator system.

because of the use of ceramic fiber blanket insulation instead of solid refractories, the electric furnace may be shut down and restarted without the refractory problems that occur in other incineration systems. Although this design feature makes the electric furnace incinerator suitable for intermittent operation, each restart requires supplemental energy (electricity) because no heat sink exists as found in refractory incineration systems.

Although the use of an electric furnace incinerator is a feasible biosolids disposal alternative for both small and large wastewater treatment plants, the electric furnace incinerator requires considerably more floor space than furnaces that are vertically oriented. Another concern in using the electric furnace incinerator is the replacement frequency for various system components such as the woven-wire belt (*ca.* 3- to 5-year life expectancy) and the infrared heaters (*ca.* 3-year life expectancy). The replacement costs of these system components alone typically amount to more than 50 percent of the capital costs for the entire unit [16,36]. Finally, electricity is generally a more expensive energy source than the fossil fuels used by the other incinerator designs. Therefore, unless the feed biosolids will burn autogenously, an electric furnace may not be a cost-effective biosolids disposal alternative.

1.4.3 General incinerator design requirements

The first step in the design of a biosolids incineration system is to determine both the feed rate (pound or kilograms per hour, wet basis) and the feed characteristics (percent solids, percent combustible matter, solids heating value, ultimate analysis, fusion point of the ash, etc.) that the system must handle. Unfortunately, in many instances, precise information regarding the characteristics of the feed biosolids is unknown. In these instances, the design engineer typically will base the size of the incinerator (together with the operational parameter values) on the anticipated range of the biosolids generation rate and feed quality. At a minimum, the following operational parameters must be specified to design an effective biosolids incineration system:

1. Biosolids feed rate
2. Minimum and maximum furnace exhaust temperatures
3. Minimum percent oxygen in exhaust gas (i.e., amount of excess air)
4. Biosolids combustion air requirement (mass and volume flow rate)
5. Auxiliary fuel requirement
6. Auxiliary fuel combustion air requirement
7. Furnace exhaust flue gas volume (cubic feet per minute)

1.4.3.1 Biosolids feed rate. The incinerator operator usually determines the biosolids feed rate based on the design capacity of the incinerator and the rate at which biosolids are generated and must be disposed. In general, the biosolids feed rate is determined as a function of either (1) the average daily design capacity for all biosolid incinerators within a site or (2) the average daily amount of biosolids fired in all incinerators within the property line of a site for a number of days that the incinerator operates during a 365-day period. Example 1.7 illustrates both approaches for estimating the biosolids feed rate.

Example 1.7 The Baldwin County Water Reclamation Facility is currently using four fluidized-bed incinerator systems to dispose of their dewatered biosolids. The design capacity of each unit is given as follows:

Incinerator	Design capacity (dry metric tons per day, dmt/day)
Unit I	100 dmt/day
Unit II	200 dmt/day
Unit III	200 dmt/day
Unit IV	200 dmt/day

For the first 60 days of the year, only unit I is operational. The average biosolids feed rate for this unit is 50 dmt/day. After 60 days, unit I is shut down. From day 61 through day 100, only unit II is operational. For the 40-day period, unit II is operated at a daily feed rate of 120 dmt/day, after which it is shut down. From day 101 through day 200, both unit III and unit IV operate at 120 dmt/day, after which both units are shut down. From day 201 through day 365, no incinerators are operational. Using these annual operational data, estimate the biosolids feed rate based on both the design capacity of the incinerators and the average daily amount of biosolids fired to all four incinerators during the 365-day period.

solution

Step 1. To determine the biosolids feed rate based on the design capacity of the incinerators, sum all the design capacities as follows:

$$\begin{aligned}\text{Biosolids feed rate} &= \text{unit I (design capacity)} + \text{unit II (design capacity)} + \\ &\quad \text{unit III (design capacity)} + \text{unit IV (design capacity)} \\ &= 100\ \text{dmt/yr} + 200\ \text{dmt/yr} + 200\ \text{dmt/yr} + 200\ \text{dmt/yr} \\ &= 700\ \text{dmt/yr}\end{aligned}$$

Step 2. To estimate the biosolids feed rate based on the daily amount of biosolids fired to all four incinerators over the 365-day period, sum all the biosolids fired at the site as follows:

$$\begin{array}{llll} \text{Unit I} & = 50\ \text{dmt/day} \cdot 60\ \text{days} & = & 3000\ \text{dmt} \\ \text{Unit II} & = 120\ \text{dmt/day} \cdot 40\ \text{days} & = & 4800\ \text{dmt} \\ \text{Unit III} & = 90\ \text{dmt/day} \cdot 100\ \text{days} & = & 9000\ \text{dmt} \\ \text{Unit VI} & = 90\ \text{dmt/day} \cdot 100\ \text{days} & = & \underline{9000\ \text{dmt}} \\ & \text{TOTAL} & & 25{,}800\ \text{dmt} \end{array}$$

Step 3. Estimate the average biosolids feed rate by dividing the total amount of biosolids fired to the incinerators by the total number of operating days during the 365-day period. Note that at least one incinerator is operational during a period of 200 days during the 365-day period.

$$\text{Biosolids feed rate} = \frac{\text{total amount of biosolids fired in 365-day period}}{\text{no. of operational days per year}}$$

$$= \frac{25{,}800\ \text{tons}}{200\ \text{days}} = \frac{129\ \text{tons}}{\text{day}}$$

NOTE: Basing the biosolids feed rate on the incinerator design capacity will provide greater flexibility in system operation.

As illustrated in Example 1.7, biosolids incinerator operators will have more process flexibility if they use the incinerator design capacity rather than the daily amount of biosolids fired to all incinerators to estimate the biosolids feed rate. Basing the biosolids feed rate on design capacity allows the incinerator operators to increase their biosolids throughput from a more typical less-than-design-capacity operation to a maximum-design-capacity operation without exceeding the permitted pollutant limits [24,30].

1.4.3.2 Airflow rates. To estimate the minimum airflow rates to the incinerator, the excess air requirement must be determined. The excess air requirement, in turn, is obtained by specifying the volume percent of oxygen in the exhaust gas. Equation (1.5) may be used to estimate the percent excess air given the desired percent oxygen content in the exhaust gas (dry basis).

$$\text{Percent excess air (\%)} = \left[\frac{O_2}{21 - O_2}\right] \cdot 100 \tag{1.5}$$

where O_2 = percent oxygen in exhaust gas (dry basis).

1.4.4 Regulatory considerations for biosolids incineration

The 40 CFR Part 503 (Subpart E) rule specifies the regulatory requirements that must be followed to legally dispose of biosolids through incineration. It should be noted that biosolids incinerator ash, which is the residue from the combustion process, is not regulated by the 40 CFR Part 503 rule when it is either used or disposed. Moreover, biosolids ash normally is exempted from federal regulation as a toxic waste and can be disposed in a sanitary landfill [7,10,11,24].

The 40 CFR Part 503 rule subjects the owner/operator of the biosolids incinerator to the following six types of regulatory requirements: (1) general requirements, (2) pollutant limits, (3) management practices, (4) monitoring requirements, (5) recordkeeping requirements, and (6) reporting requirements. Each of these requirements is discussed in further detail in the following sections.

1.4.5 General requirements

The general requirements mandate that no person shall fire a biosolids incinerator except in compliance with the regulations specified in Subpart E of the 40 CFR Part 503 rule. The 40 CFR Part 503 (Subpart E) rule regulates the concentrations of certain metal pollutants that may be contained in biosolids fired to a biosolids incinerator as well as the emission rate of certain metals and the concentration of total hydrocarbons (or carbon monoxide) contained in the exhaust gas.

1.4.6 Pollutant limits

Pollutant limits refer to the maximum concentration of a particular pollutant that may be legally found in biosolids fired to a biosolids incinerator. Pollutants that are regulated within biosolids fired to a biosolids incinerator include the metals arsenic, cadmium, chromium, lead, and nickel [24,30]. The following sections describe the regulatory framework used to establish each of the pollutant limits.

1.4.6.1 Estimating the pollutant limit for lead. Anyone firing biosolids to a biosolids incinerator must determine the pollutant limit for lead (C_{lead}) in biosolids using Eq. (1.6). In addition to employing the current National Ambient Air Quality Standard (NAAQS) for lead, use of Eq. (1.6) to estimate the pollutant limit for lead requires specifying the values of several incinerator system operational parameters, including (1) pollutant control efficiency, (2) pollutant dispersion factor, and (3) biosolids feed rate (dry metric tons of biosolids per day, dmt/day).

$$C_{\text{lead}} = \frac{0.1 \cdot \text{NAAQS} \cdot 86{,}400}{DF \cdot (1 - CE) \cdot SF} \tag{1.6}$$

where C_{lead} = pollutant limit for lead (mg/kg), dry basis

0.1 = allowable ground-level concentration of lead from biosolids is 10 percent of the NAAQS for lead

NAAQS = National Ambient Air Quality Standard for lead (currently 1.5 μg/m^3)

86,400 = constant (seconds per day)

DF = dispersion factor ($\mu g/m^3 \cdot g \cdot s$) based on air dispersion model

CE = biosolids incinerator control efficiency (percentage as a decimal)

SF = biosolids feed rate in dry metric tons per hour (dmt/h)

Pollutant control efficiency refers to the degree to which a biosolids incinerator furnace, in conjunction with an air pollution control system, removes a particular pollutant. For example, if a quantity of biosolids fed to an incinerator contains 100 g of lead and 1 g is released in the exhaust gas, the incinerator has a 99 percent control efficiency for lead. Pollutant control efficiency is determined through conducting a performance test of the incinerator under normal operating conditions. To provide confidence in establishing the pollution control efficiency, the permitting authority normally will require that the owner/operator of the incinerator employ a mass-balance approach. In other words, the pollutant control efficiency determination would be established based on the mass flow of regulated metals in the biosolids feed and ash, the mass flow of regulated metals in the incinerator air emissions, and the operating conditions set for the air pollution control device [21,30].

A *dispersion factor* is the ratio of the concentration of a pollutant in air at or beyond the property line of an incinerator site relative to the rate pollutants are emitted in the exhaust gases. The dispersion factor normally is determined through the use of a regulatory-approved air dispersion model. The air dispersion model will consider particular site conditions (e.g., temperature and velocity of gas from the stack, wind speed and direction, type of terrain, etc.) together with exhaust gas and ambient air pollutant measurements to estimate the dispersion factor [21,30].

1.4.6.2 Estimating pollutant limits for arsenic, cadmium, chromium, and nickel. The pollutant limits for arsenic, cadmium, chromium, and nickel in biosolids fired in a biosolids incinerator may be estimated using Eq. (1.7).

$$C = \frac{\text{RSC} \cdot 86{,}400}{DF \cdot (1 - CE) \cdot SF} \tag{1.7}$$

where C = average daily concentration of the pollutant in biosolids (mg/kg), dry weight

RSC = risk-specific concentration specified in Subpart E ($\mu g/m^3$)
86,400 = constant (seconds per day)
DF = dispersion factor ($\mu g/m^3 \cdot g \cdot s$) based on air dispersion model
CE = biosolids incinerator pollutant control efficiency (percentage as a decimal)
SF = biosolids feed rate in dry metric tons per hour (dmt/h)

Unlike lead, which uses the NAAQS value to estimate the pollutant limit, arsenic, cadmium, chromium, and nickel use the risk-specific concentrations (RSCs). RSCs, which are based on human health risk-assessment data, represent the allowable increase in the average daily ground-level ambient air concentrations of pollutants at or beyond the property line of the biosolids incinerator site. The RSC values for arsenic, cadmium, and nickel are given in Table 1.33.

Unlike arsenic, cadmium, and nickel, the RSC for chromium is based on either (1) the type of incinerator together with the particular air pollution control device used for biosolids disposal or (2) the concentration ratio of hexavalent (VI) to total chromium in the exhaust gases. The RSC values for chromium based on the incinerator type used are provided in Table 1.34.

When the exhaust gas analysis is used to determine the concentration ratio of hexavalent (VI) to total chromium, Eq. (1.8) may be employed to estimate the RSC for chromium. Example 1.8 illustrates the approach for estimating the biosolids pollutant limits.

$$\mathrm{RSC} = \frac{0.0085}{r} \tag{1.8}$$

where RSC = site-specific RSC for chromium in $\mu g/m^3$
r = decimal fraction of hexavalent chromium in the total chromium concentration

If measurements of pollutants in the biosolids are higher than those estimated from using Eqs. (1.6) and/or (1.7), the biosolids incinerator

TABLE 1.33 Risk-Specific Concentrations for Arsenic, Cadmium, and Nickel*

Pollutant	RSC ($\mu g/m^3$)
Arsenic	0.023
Cadmium	0.057
Nickel	2.000

*Adapted from refs. [22,30].

TABLE 1.34 Risk-Specific Concentrations for Chromium Based on Incinerator System*

Chromium	RSC (μg/m^3)
Fluidized bed with wet scrubber	0.650
Fluidized bed with wet scrubber and wet electrostatic precipitator	0.230
Other types of wet scrubbers	0.064
Other types of wet scrubbers and wet electrostatic precipitators	0.016

*Adapted from refs. [21,22,30].

will be in violation of the 40 CFR Part 503 (Subpart E) rule until system adjustments are made that allow the limits to be met. Such system adjustments include, but are not limited to, improvements in biosolids quality through pretreatment (see Chap. 4), reduction in the biosolids feed rate, improved furnace operation, or addition of an air pollution control device to improve pollutant control efficiency. It should be noted that if furnace or air pollution control device improvements are made, the performance test used to establish the pollutant control efficiency must be repeated and documented for the permitting authority.

In addition to pollutant limits, the USEPA established operational standards for certain pollutants included in the exhaust gases. The pollutants that are regulated in the exhaust gases include beryllium, mercury, and total hydrocarbons (or carbon monoxide). The following sections describe the regulatory framework used to establish the permissible pollutant emission levels in the exhaust gases.

Example 1.8 The Baldwin County Water Reclamation Facility (see Example 1.7) has scheduled to bring its biosolids incinerator system on line within 90 days. From performance tests on the fluidized-bed incinerator system, the wet scrubbers used to treat the exhaust gas have been found to have a pollutant control efficiency of 98 percent for all regulated pollutants. If the biosolids feed rate is 700 dmt/day and the dispersion factor is 3.5 μg/m$^3 \cdot$ g $\cdot$ s, estimate the biosolid pollutant limits for lead, arsenic, cadmium, nickel, and chromium. Assume that the current NAAQS for lead is 1.5 μg/m^3.

solution

Step 1. Estimate the pollutant limit for lead using Eq. (1.6).

$$C_{\text{lead}} = \frac{0.1 \cdot \text{NAAQS} \cdot 86{,}400}{DF \cdot (1 - CE) \cdot SF}$$

$$= \frac{0.1 \cdot (1.5\ \mu\text{g/m}^3) \cdot 86{,}400}{\left(\dfrac{3.5\ \mu\text{g}}{\text{m}^3 \cdot \text{g} \cdot \text{s}}\right) \cdot (1 - 0.98) \cdot \left(\dfrac{700\ \text{dmt}}{\text{day}}\right)}$$

$$= \frac{264.5\ \text{mg}}{\text{kg}}$$

Step 2. Estimate the pollutant limit for arsenic, cadmium, and nickel using Eq. (1.7) and Table 1.33. From Table 1.33, the RSCs for arsenic, cadmium, and nickel are 0.023, 0.057, and 2.000, respectively.

$$C_{\text{arsenic}} = \frac{\text{RCS} \cdot 86{,}400}{DF \cdot (1 - CD) \cdot SF}$$

$$= \frac{0.023 \cdot 86{,}400}{\left(\dfrac{3.5\ \mu\text{g}}{\text{m}^3 \cdot \text{g} \cdot \text{s}}\right) \cdot (1 - 0.98) \cdot \left(\dfrac{700\ \text{dmt}}{\text{day}}\right)}$$

$$= \frac{40.6\ \text{mg}}{\text{kg}}$$

$$C_{\text{cadmium}} = \frac{\text{RCS} \cdot 86{,}400}{DF \cdot (1 - CE) \cdot SF}$$

$$= \frac{0.057 \cdot 86{,}400}{\left(\dfrac{3.5\ \mu\text{g}}{\text{m}^3 \cdot \text{g} \cdot \text{s}}\right) \cdot (1 - 0.98) \cdot \left(\dfrac{700\ \text{dmt}}{\text{day}}\right)}$$

$$= \frac{100.5\ \text{mg}}{\text{kg}}$$

$$C_{\text{nickel}} = \frac{\text{RCS} \cdot 86{,}400}{DF \cdot (1 - CE) \cdot SF}$$

$$= \frac{2.000 \cdot 86{,}400}{\left(\dfrac{3.5\ \mu\text{g}}{\text{m}^3 \cdot \text{g} \cdot \text{s}}\right) \cdot (1 - 0.98) \cdot \left(\dfrac{700\ \text{dmt}}{\text{day}}\right)}$$

$$= \frac{3526.3\ \text{mg}}{\text{kg}}$$

Step 3. Estimate the pollutant limit chromium using the RSC for a fluidized-bed incinerator equipped from a wet scrubber. From Table 1.34, the RSC for chromium is 0.65.

$$C_{\text{chromium}} = \frac{\text{RCS} \cdot 86{,}400}{DF \cdot (1 - CE) \cdot SF}$$

$$= \frac{0.65 \cdot 86{,}400}{\left(\dfrac{3.5\ \mu\text{g}}{\text{m}^3 \cdot \text{g} \cdot \text{s}}\right) \cdot (1 - 0.98) \cdot \left(\dfrac{700\ \text{dmt}}{\text{day}}\right)}$$

$$= \frac{1146.1\ \text{mg}}{\text{kg}}$$

1.4.6.3 Emission limits for beryllium and mercury. The emission limits for beryllium and mercury contained in a biosolids incinerator exhaust gas are based on the National Emission Standards for Hazardous Air Pollutants (NESHAP) for incineration (40 CFR Part 61). The NESHAP for beryllium requires that the total quantity of beryllium emitted from a biosolids incinerator not exceed 10 g during any 24-hour period. The NESHAP emission limitation for beryllium can be waived if written approval has been obtained from the USEPA regional administrator, which may occur if (1) the ambient beryllium concentration in the proximity of the biosolids incinerator does not exceed 0.01 $\mu g/m^3$ when averaged over a 30-day period or (2) if the biosolids incinerator operator can demonstrate (with historical data) that the biosolids fired in the incinerator do not contain beryllium [21,22,24,30]. The NESHAP emission limitation for mercury requires that the total quantity of mercury emitted from each biosolids incinerator not exceed 3200 g during any 24-hour period [22,24,30].

1.4.6.4 Emission limits for total hydrocarbons and carbon monoxide. Organic compounds that are generated as a result of incomplete combustion or are produced as combustion by-products (e.g., benzene, phenol, vinyl chloride, etc.) can be present in biosolids incinerator emissions. Since these compounds can be harmful to public health, the 40 CFR Part 503 (Subpart E) rule limits the emission of total hydrocarbons (THCs) in the exhaust gases from biosolids incinerators. As an alternative approach for ensuring that the THC emission limits will be met, the USEPA has allowed the monitoring of carbon monoxide (CO) in the exhaust gas as a surrogate measurement for total hydrocarbons [24,30].

To be in compliance with the 40 CFR Part 503 rule, the maximum monthly average concentration of THCs (or CO) in the exhaust gas from a biosolids incinerator can be no greater than 100 parts per million (volume basis, ppmv). The 40 CFR Part 503 rule defines the monthly average THC (or CO) concentration as the arithmetic mean of the hourly pollutant measurement averages, each of which must be calculated based on at least two readings taken each hour that the incinerator operates [22,30]. In addition to the number of exhaust air samples that must be taken, the 40 CFR Part 503 rule specifies that the THC (or CO) concentration must be measured using a flame ionization detector (FID) with a sampling line heated to at least 150°C (300°F) (which minimizes pollutant condensation).

It should be noted that certain incinerator operational conditions will affect the THC (or CO) measurement. The two primary incinerator operational conditions that affect the THC (or CO) measurement are (1) feed biosolids moisture content and (2) excess airflow rates. To account for the effects of these incinerator operational conditions (as well as others) on the exhaust gas pollutant measurements, the USEPA requires that the measured THC (or CO) concentration be corrected to 0 percent moisture and 7 percent oxygen content before being compared with the 100-ppmv regulatory limit.

To correct the THC (or CO) exhaust gas concentrations to 0 percent moisture, Eq. (1.9) may be used, while THC (or CO) measurements may be normalized to 7 percent oxygen using Eq. (1.10). Example 1.9 illustrates the approach for standardizing the THC (or CO) stack measurements to the requisite regulatory conditions necessary for compliance verification.

$$\text{Correction factor}_{\text{moisture}} = \frac{1}{1 - X} \tag{1.9}$$

where X = decimal fraction of percent moisture in the stack gas

$$\text{Correction factor}_{\text{oxygen}} = \frac{14}{21 - Y} \tag{1.10}$$

where 14 = difference between percent oxygen in air (i.e., 21 percent) and 7 percent oxygen
21 = percent oxygen in air
Y = percent oxygen concentration in incinerator exhaust gas (volume/volume)

Example 1.9 The Baldwin County Water Reclamation Facility (see Example 1.7) has opted to monitor carbon monoxide as a surrogate for total hydrocarbons (THCs). If the measured monthly average CO concentration for each of the four incinerators is 23 ppmv (unit I), 72 ppmv (unit II), 49 ppmv (unit III), and 84 ppmv (unit IV), determine the compliance status of each of the units. Assume that the average moisture and oxygen contents of the exhaust gas of each unit are 15 and 10 percent, respectively.

solution

Step 1. Estimate the moisture and oxygen correction factors using Eqs. (1.9) and (1.10), respectively.

$$\text{Correction factor}_{\text{moisture}} = \frac{1}{1 - X} = \frac{1}{1 - 0.15} = 1.1765$$

$$\text{Correction factor}_{\text{oxygen}} = \frac{14}{21 - Y} = \frac{14}{21 - 10} = 1.2727$$

Step 2. Multiply each of the carbon dioxide stack measurements by both the moisture and oxygen correction factors to obtain the regulatory measurement.

Unit I:

Regulatory CO measurement

$$= \text{stack measurement} \cdot \text{correction factor}_{\text{moisture}} \cdot \text{correction factor}_{\text{oxygen}}$$

$$= 23\ \text{ppm} \cdot 1.1765 \cdot 1.2727$$

$$= 34.4\ \text{ppm}$$

Unit II:

Regulatory CO measurement

$$= \text{stack measurement} \cdot \text{correction factor}_{\text{moisture}} \cdot \text{correction factor}_{\text{oxygen}}$$

$$= 72\ \text{ppm} \cdot 1.1765 \cdot 1.2727$$

$$= 107.8\ \text{ppm}$$

Unit III:

Regulatory CO measurement

$$= \text{stack measurement} \cdot \text{correction factor}_{\text{moisture}} \cdot \text{correction factor}_{\text{oxygen}}$$

$$= 49\ \text{ppm} \cdot 1.1765 \cdot 1.2727$$

$$= 73.4\ \text{ppm}$$

Unit VI:

Regulatory CO measurement

$$= \text{stack measurement} \cdot \text{correction factor}_{\text{moisture}} \cdot \text{correction factor}_{\text{oxygen}}$$

$$= 84\ \text{ppm} \cdot 1.1765 \cdot 1.2727$$

$$= 125.8\ \text{ppm}$$

NOTE: Incinerators II and IV are out of compliance with the 40 CFR Part 503 (Subpart E) rule.

It should be noted that if the normalized monthly average THC (or CO) concentration were found to be above 100 ppmv, the biosolids incinerator would be in violation of the 40 CFR Part 503 (Subpart E) rule until adjustments are made to achieve the regulatory limits (typical adjustments include altering the furnace temperature and/or improving the pollutant control efficiency).

1.4.7 Management practices for biosolids incineration

The 40 CFR Part 503 rule specifies several management practices that must be followed to legally operate a biosolids incinerator.

Management practices include (1) instrument operation and maintenance, (2) temperature requirements, (3) operation of air pollution control devices, and (4) protection of threatened or endangered species.

Biosolids incinerator operators must use instruments to continuously measure and record certain information, including (1) THCs (or CO) in the exhaust gas, (2) oxygen content in the exhaust gas, (3) information used to calculate moisture content of the exhaust gas, and (4) furnace combustion temperatures. Biosolids management practices require that each of the instruments used for these measurements be installed, calibrated, operated, and maintained according to guidance provided by the permitting authority. Examples of instruments used to monitor the incineration process include (1) flame ionization detector (FID, for measurement of THCs or CO), (2) extractive or in situ oxygen analyzers (for O_2 measurement in exhaust gas), (3) thermocouples (temperature measurements), and (4) dew point detectors (for moisture content estimation of exhaust gas). In addition to using an FID, management practices require that the THC (or CO) monitoring system employ a sampling line heated to at least 150°C (300°F) and that the FID detector be calibrated using propane at least once every 24-hour operating period [24,30].

1.4.7.1 Temperature requirements. Because of its impact on both the pollutant control efficiency and auxiliary fuel requirements, incinerator temperature is a critical system operational parameter. The permitting authority, based on performance test data, establishes the maximum combustion temperature allowed in the incinerator furnace. A limit on combustion temperature is necessary to ensure that the daily performance of the incinerator is similar to what was recorded during the performance test. If biosolids were incinerated at higher temperatures than those recorded during the performance test, the pollutant control efficiency may be significantly different during normal operation.

1.4.7.2 Air pollution control devices. In addition to the maximum combustion temperature, the regulatory authority is responsible for determining the permissible operational conditions for any air pollution control device treating exhaust gases from the biosolids incinerator. The range of acceptable values for the operational parameters (e.g., liquid flow rate, pressure drop, temperature, etc.) is determined by the permitting authority using data obtained during the system performance test. The operational conditions are established to ensure that the air pollution control device will achieve the desired level of pollutant control efficiency. Examples of operational parameters that are

TABLE 1.35 Operational Parameters for Air Pollution Control Devices*

Operational parameter	Air pollution control device	Measuring instrument
Pressure drop	Venturi scrubber Fabric filter Mist eliminator Impingement scrubber	Differential pressure gauge
Liquid flow rate	Venturi scrubber Impingement scrubber Wet electrostatic precipitator	Orifice plate with differential pressure gauge
Gas temperature	Venturi scrubber Impingement scrubber Fabric filter Dry scrubber	Thermocouple
Compressed air pressure	Dry scrubber	Pressure gauge
Opacity	Fabric filter	Transmissometer
Liquid/reagent flow	Dry scrubber	Magnetic flowmeter
Atomized motor power	Dry scrubber	Wattmeter

*Adapted from refs. [24,30].

typically used to control the performance of air pollution control devices are summarized in Table 1.35.

1.4.7.3 Protection of threatened or endangered species. The 40 CFR Part 503 (Subpart E) rule does not allow biosolids to be incinerated if a threatened or endangered animal or plant species or its “critical habitat” is likely to be adversely affected. *Critical habitat* is defined as any place where a threatened or endangered species lives and grows during any stage of its life cycle [21,24,30]. Any direct or indirect action (or the result of any direct or indirect action) in a critical habitat that diminishes the likelihood of survival and recovery of a listed species is considered destructive of a critical habitat [24]. A list of endangered and threatened species is published annually by the U.S. Fish and Wildlife Service as mandated by the Endangered Species Act (PL 99-625). Specific regulations pertaining to the protection of endangered and threatened plants and animals may be found in 50 CFR Part 17 [21].

1.4.8 Monitoring frequency

The owner/operator of a biosolids incinerator must monitor at specified intervals for various biosolids quality parameters as well as pollutants in incinerator emissions. A summary of these monitoring requirements is provided in Table 1.36.

TABLE 1.36 Parameters That Must Be Monitored during Biosolids Incineration*

1. Concentration of metals in biosolids (arsenic, cadmium, chromium, lead, and nickel)
2. Concentration of beryllium and mercury in exhaust gas
3. Concentration of THCs (or CO) in exhaust gas
4. Concentration of oxygen in exhaust gas
5. Moisture content of exhaust gas
6. Combustion temperature in the furnace
7. Operating conditions of the air pollution control device
8. Biosolids feed rate

*Adapted from refs. [24,30].

For the regulated metals contained in feed biosolids (i.e., arsenic, cadmium, chromium, lead, and nickel), the minimum frequency for monitoring is based on the amount of biosolids incinerated. POTWs incinerating zero to less than 290 dry metric tons (dmt) annually must test for these metals once per year. Those POTWs incinerating 290 to less than 1500 dmt of biosolids per year must evaluate for these metals in biosolids once per quarter, whereas POTWs incinerating 1500 to less than 15,000 dmt of biosolids per year must test for these metals once every 60 days. POTWs incinerating 15,000 dmt or more of biosolids must evaluate for the concentration of these metals once per month.

Continuous emissions monitoring (CEM) of the exhaust gas is required to quantify the (1) THC (or CO) concentration, (2) oxygen content, and (3) moisture content. CEM is also required for ensuring that the combustion temperature in the furnace remains within the permissible range. In addition, the permitting authority typically requires that certain operating conditions of air pollution control devices be monitored routinely. The specific parameters that must be monitored are based on the type of air pollution control device used and the operating parameters that are important for maintaining the established pollutant control efficiency. Finally, the permitting authority will determine how often the facility operator must monitor for beryllium and mercury in the exhaust gas.

1.4.9 Recordkeeping

The owner/operator of the biosolids incinerator must develop and maintain certain records for a minimum of 5 years. The recordkeeping requirements, which include information on the pollutant limits, management practices, and monitoring requirements, are summarized in Table 1.37.

TABLE 1.37 Records That Must Be Kept by Owner/Operator of Biosolids Incinerator for 5 Years*

1. The concentrations of arsenic, chromium, cadmium, lead, and nickel in the biosolids fed to the incinerator.
2. The concentrations of THCs, moisture, and oxygen in the exhaust gases and information used to measure these parameters.
3. Information indicating that the beryllium and mercury emission limits specified in 40 CFR Part 61 are met.
4. Combustion temperature range including the maximum temperature as set by the permitting authority.
5. Operating parameter values for the air pollution control device.
6. The biosolids feed rate for each incinerator.
7. The stack height and dispersion factor for the site.
8. The pollutant control efficiency for lead, arsenic, cadmium, chromium, and nickel.
9. The risk specific concentration (RSC) for chromium, if calculated.
10. A calibration and maintenance log for monitors used to measure the combustion temperature, THC concentration, oxygen concentration, and moisture content.

*Adapted from refs. [24,30].

1.4.10 Reporting requirements

All Class I treatment works, treatment works serving a population of 10,000 or more, and treatment works with a 1 million gallon per day (1 MGD) or greater design wastewater influent flow must report the following information to the permitting authority: (1) pollutant concentration in biosolids fired to the incinerator, (2) THCs (or CO) in exhaust gas, (3) oxygen and moisture contents of exhaust gas, (4) mercury and beryllium emission data, (5) furnace combustion temperature, and (6) operational data for the air pollution control device(s). POTW reporting information must be submitted to the permitting authority in an annual report, which is due by February 19 of the following calendar year.

1.4.11 USEPA Biosolids Data Management System

Although the 40 CFR Part 503 rule requires all Class I facilities to submit biosolids annual reports, the USEPA currently does not have a centralized database to collect and review the data quality. The Biosolids Data Management System (BDMS), which has been developed by the USEPA (Region VIII), will be employed for assessing the quality of biosolids data collected throughout the United States. The BDMS is basically an electronic file cabinet designed for the storage and retrieval of biosolids management data. The system enables the user to store, search, retrieve, and review all information necessary to determine a facility's compliance status. BDMS allows the user to identify discrepancies in the data submitted by facilities with its error-checking capabilities. With the data in the BDMS, biosolids quality together with the compliance status of biosolids management systems can be evaluated within a state, USEPA region, or across the United States. By increasing access to biosolids data, it is anticipated that public confidence in the biosolids program will increase.

The BDMS consists of nine active databases and ten libraries [5]. The active databases include (1) general facilities information (e.g., point of contact, addresses, phone numbers), (2) biosolids treatment provided, (3) use/disposal method, (4) land-application site information, (5) cumulative loading tracking, (6) monitoring data (yearly averages and maximums), (7) monitoring data tracking (individual data points), (8) pathogen reduction and vector attraction reduction, and (9) findings. The libraries are a repository for the data that are common to all databases.

1.4.12 Criticisms of the 40 CFR Part 503 rule

The 40 CFR Part 503 rule has provided a set of risk-based and/or operational standards designed to promote regulatory uniformity for biosolids beneficial use. Although some critics have argued that the 40 CFR Part 503 rule is unnecessarily restrictive, most published criticisms have focused on claims of inadequate protection of public health and the environment [6]. The majority of complaints regarding the 40 CFR Part 503 rule fall into one of the following three categories: (1) alternative protection paradigms, (2) unaddressed issues, or (3) methodologic arguments.

Paradigm-based criticisms reflect the fact that an individual or organization has a fundamentally different philosophical approach to biosolids recycling than does the USEPA. An example of an alternative protection paradigm is exemplified by the policy-formulated regulations of some European countries, which are based on soil-protection goals that balance soil metal inputs and losses. This approach to establishing biosolids land-application rates would result in numerical soil loadings that are significantly more restrictive than the 40 CFR Part 503 rule [2,3].

With regard to unaddressed issues, most objections of the 40 CFR Part 503 rule have focused on the additive effects of metals relative to phytotoxicity and/or pathogen regrowth in stored biosolids. In addition, there have been criticisms pertaining to the existence of unregulated contaminants in biosolids that may pose a significant human health and environmental risk. The unregulated contaminants in question include (1) synthetic organics, (2) certain inorganics (e.g., barium), and (3) radioactivity [4,33].

By far the largest number of objections of the 40 CFR Part 503 rule have been focused on the methodology employed to develop the regulatory limits for pollutants. Critics of the 40 CFR Part 503 rule have identified potential flaws in the risk-assessment process including claims that the USEPA used inappropriate extrapolation of published research data [28]. Some of these same critics argue that the risk-assessment process was biased toward low risk determination through underestimation of soil ingestion, inaccurate dietary assumptions, use of geometric mean to average crop uptake coefficients, high phytotoxicity thresholds, and questionable assumptions regarding long-term trace element bioavailability.

Although criticisms of the 40 CFR Part 503 rule stem from different philosophical positions and others reflect gaps in the existing knowledge base, in both cases addressing and resolving these concerns will require long-term studies. As field experience pertaining to the application of the 40 CFR Part 503 rule is gained, the broadening scope of knowledge will allow refinement of regulations and more precise balancing of resource conservation and environmental protection.

1.5 Problems

1.1 The Malindi County Sewer Improvement District is considering land application of its biosolids onto adjacent agricultural land. Based on the nitrogen content of the biosolids and the nitrogen demand of the crop, the biosolids land-application rate is to be maintained at 35 dry metric tons per hectare per year. If the lead content of the biosolids is 45 mg/kg (dry weight), estimate the land-application site life (in years) based on lead if the cumulative pollutant loading rate (CPLR) for lead is 300 kg/ha.

1.2 The Tororo City Wastewater Treatment Plant has signed a 10-year contract with a local tree farmer to land apply biosolids as a supplemental fertilizer at a rate of 18 dry metric tons per hectare per year. The owner of the tree farm has recently expressed concern that the accumulated nickel concentration in the soil may be adversely affecting tree growth. If the maximum observed nickel concentration in biosolids was found to be 69 mg/kg (dry weight), estimate the land-application site life (in years) if the cumulative pollutant loading rate (CPLR) for nickel is 420 kg/ha.

1.3 Eldoret City Water Reclamation Plant anticipates selling its biosolids to the public in 75-lb prepackaged sacks. If the biosolids have the following metal concentrations, what annual whole sludge application rate (dry metric tons/ha·yr and lb/1000 ft^2·yr) must be included on the label affixed to each sack?

Pollutant	Concentration (mg/kg)
Arsenic	14.5
Cadmium	70.0
Copper	1400.0
Lead	420.0
Mercury	7.8
Nickel	260.0
Selenium	39.0
Zinc	1850.0

1.4 Eldoret City Water Reclamation Plant (see Prob. 1.3) has instituted a new industrial pretreatment program. If the resulting biosolids have the following metal concentrations, what annual whole-sludge application rate (dry metric tons/ha·yr and lb/1000 ft^2·yr) must be included on the label affixed to each sack of biosolids sold to the public?

Pollutant	Concentration (mg/kg)
Arsenic	10.0
Cadmium	15.0
Copper	720.0
Lead	110.0
Mercury	1.4
Nickel	130.0
Selenium	19.0
Zinc	1120.0

1.5 The Garissa County Botanical Gardens is considering using biosolids as a low-cost fertilizer for its border hedgerows. Based on the metals content of the biosolids, an annual whole-sludge application rate of 24 dry metric tons per hectare per year was estimated. If the nitrogen requirement for the hedgerows is estimated to be 180 lb of nitrogen per acre per year, calculate the fraction of nitrogen supplied by the biosolids relative to vegetative requirements during the first year. Assume that the nitrogen content of the biosolids is 1.2 percent, of which 20 percent is available during the first year of application.

1.6 The Turkana City Wastewater Treatment Plant has decided to purchase a biosolids dryer to meet Class A pathogen-reduction requirements. If the biosolids have an initial solids content of 18 percent and are pelletized prior to entering the dryer, what is the minimum length of time that the biosolids must remain in the dryer if they are heated by warm air to a temperature of 155°F (68.3°C)?

1.7 The Turkana City Wastewater Treatment Plant (see Prob. 1.6) has decided to purchase a new biosolids dryer that will allow the biosolids processing time to be reduced to 12 minutes. To ensure that the biosolids meet Class A quality, what is the minimum temperature to which the biosolids must be subjected during the 12-minute period if heating regime B is followed?

1.8 The Tsavo City Sewage Works would like to employ surface disposal of its biosolids in an isolated section of the county. If the proposed surface disposal site is 80 acres, estimate the site life if the biosolids generation rate is 100 cubic yards (yd^3) per day. Assume that a wide-trench disposal design is to be employed with trench dimensions of 250 ft (length), 50 ft (width), and 10 ft (depth) and that there is a minimum spacing requirement of 10 ft between trenches.

1.9 The Lodwar County Solids Waste Control Board has approved the construction of a surface-disposal site for locally generated biosolids. If the solids content of the discharged biosolids is 35 percent, estimate the total trench volume, usable area, and minimum total area if the local biosolids generation rate is 120 cubic yards (yd^3) per day. Assume that the following conditions apply:

Trench life: 20 years
Trench dimensions: 40 ft wide by 10 ft deep by 250 ft long
Trench spacing: 15 ft between trenches
Buffer: 250 ft minimum from usable filling area to property line
Other assumptions: 25 percent additional area for access roads, dumping pad, and miscellaneous uses

1.10 The Butere County Solids Waste Control Board has decided to install a fluidized-bed incinerator to be used exclusively for biosolids disposal. If the local wastewater treatment plant is generating 550 dry metric tons of biosolids per day, estimate the biosolids pollutant limits for lead, arsenic, cadmium, nickel, and chromium assuming that the following conditions apply:

Wet electrostatic precipitator control efficiency: 95.0 percent
Dispersion factor: 2.6 μg/m$^3\cdot$s
National Ambient Air Quality Standard for lead: 1.5 μg/m^3

1.11 The Garsen City Water Reclamation Plant has decided to contract with the county to collect and incinerate all locally generated biosolids. If the county is currently generating 2200 dry metric tons of biosolids per day and is planning to employ a multiple-hearth furnace equipped with a wet scrubber air-pollution-control system, estimate the biosolids pollutant limits for lead, arsenic, cadmium, nickel, and chromium assuming that the following conditions apply:

Wet scrubber control efficiency: 99.0 percent
Dispersion factor: 3.8 μg/m$^3\cdot$s
National Ambient Air Quality Standard for lead: 1.5 μg/m^3

1.12 The Kisumu County Solids Waste Control Board has decided to monitor carbon monoxide (CO) as a surrogate for total hydrocarbon (THC) in meeting the 40 CFR Part 503 emissions requirements for biosolids incineration. If the monthly average CO concentration from the incinerator is 48 parts per million (volume basis), determine the compliance status of the incinerator if the average moisture and oxygen contents of the exhaust gas are 18 and 12 percent, respectively.

1.6 References

1. American Society of Civil Engineers (1990), *Waste Containment Systems,* Geotechnical Special Publication No. 26, edited by Rudolph Bonaparte.
2. Bastian, R. K. (1997), "Biosolids Management in the United States," *Water Environment and Technology,* pp. 45–50.
3. Bastian, R. K. (1997), "The Biosolids (Sludge) Treatment, Beneficial Use and Disposal Situation in the USA," *European Water Pollution Control,* pp. 62–79.
4. Bastian, R. K., P. Sobel, L. Setlow, D. Sauders, and D. Condra (1998), "Radiation in Biosolids and Ash: Guidance for POTWs and Initial Survey Results," Water Environment Federation, CP3805.
5. Brobst, R. B. (1999), "EPA's Biosolids Data Management System and Plans for Evaluating Biosolids Quality," WEF/ASSA Joint Residuals and Biosolids Management Conference, Charlotte, N.C.
6. Harrison, E. Z., M. B. McBride, and D. R. Bouldin (1997), "The Case for Caution: Recommendations for Land Application of Sewage Sludges and an Appraisal for the USEPA's Part 503 Sludge Rules," Cornell Waste Management Institute.
7. LaGrega, M. D., P. L. Buckingham, and J. C. Evans (1994), *Hazardous Waste Management,* New York, McGraw-Hill.
8. Loehr, R., W. Jewell, J. Novak, W. Clarkson, and G. Friedman (1979), *Land Application of Wastes,* vol. 2, New York, Van Nostrand Reinhold.
9. Luthin, J. N. (1973), *Drainage Engineering,* Krieger Publishing.
10. Parkin, W. P. (1993), *The Complete Guide to Environmental Law,* Specialty Technical Publishers.
11. Tchobanoglous, G., H. Theisen, and S. Vigil (1993), *Integrated Solid Waste Management: Engineering Principles and Management Issues,* New York, McGraw-Hill.
12. U.S. Army Corps of Engineers (1987), "Wetlands Delineation Manual. Technical Report Y-87-1," Waterways Experiment Station, Vicksburg, Miss.
13. U.S. Department of the Interior (1993), *Drainage Manual: A Water Resources Technical Publication,* 3d ed., Washington, U.S. Government Printing Office.
14. U.S. Environmental Protection Agency (1983), "Process Design Manual for Land Application of Municipal Sludge," EPA 625/1-83-016.
15. U.S. Environmental Protection Agency (1984), "Handbook: Septage Treatment and Disposal," EPA/625/6-84/009.
16. U.S. Environmental Protection Agency (1985), "Seminar Publication: Municipal Wastewater Sludge Combustion Technology," EPA/625/4-85/015.
17. U.S. Environmental Protection Agency (1989), "POTW Sludge Sampling and Analysis Guidance Document," NTIS PB93227957.
18. U.S. Environmental Protection Agency (1990), "40 CFR Part 503, National Sewage Sludge Survey: Availability of Information and Data and Anticipated Impacts on Proposed Regulations," *Federal Register* 55(218):47210–47283.
19. U.S. Environmental Protection Agency (1991), "Cooperative Testing of Municipal Sewage Sludges by the Toxicity Characteristic Leaching Procedure and Compositional Analysis," EPA 430/09-91-007.
20. U.S. Environmental Protection Agency (1993), "Domestic Septage Regulatory Guidance: A Guide to the EPA 503 Rule," EPA/832/B-92/005.
21. U.S. Environmental Protection Agency (1993), "Regulatory Impact Analysis of the Part 503 Sewage Sludge Regulation," EPA/821/R-93/006.
22. U.S. Environmental Protection Agency (1993), "Standards for Use or Disposal of Sewage Sludge. Final Rule, 40 CFR Part 503," *Federal Register* 58(32):9248–9415.
23. U.S. Environmental Protection Agency (1993a), *Technical Support Document for Land Application of Sewage Sludge,* vol. I, PB93-110575; vol. II, PB93110583.
24. U.S. Environmental Protection Agency (1994), "A Plain English Guide to the EPA Part 503 Biosolids Rule," EPA/832/R-93/003.
25. U.S. Environmental Protection Agency (1994), "Biosolids Recycling: Beneficial Technology for a Better Environment," EPA/832/R-94/009.
26. U.S. Environmental Protection Agency (1994), "Guide to Septage Treatment and Disposal," EPA/625/R-94/002.

27. U.S. Environmental Protection Agency (1994), "Land Application of Sewage Sludge: A Guide for Land Appliers on the Requirements of the Federal Standards for the Use or Disposal of Sewage Sludge, 40 CFR Part 503," EPA/831/B-93/002b.
28. U.S. Environmental Protection Agency (1995), "A Guide to the Biosolids Risk Assessments for the EPA Part 503 Rule," EPA/832/B-93/005.
29. U.S. Environmental Protection Agency (1995), "Ground-Water and Leachate Treatment Systems," EPA/625/R-94/005.
30. U.S. Environmental Protection Agency (1995), "Part 503 Implementation Guidance," EPA/833/R-95/001.
31. U.S. Environmental Protection Agency (1995), "Process Design Manual: Land Application of Sewage Sludge and Domestic Septage," EPA/625/R-95/001.
32. U.S. Environmental Protection Agency (1995), "Process Design Manual: Surface Disposal of Sewage Sludge and Domestic Septage," EPA/625/R-95/002.
33. U.S. Environmental Protection Agency (1996), "Technical Support Document for the Round Two Sewage Sludge Pollutants," EPA/822/R-96/003.
34. U.S. Environmental Protection Agency (1999), "Environmental Regulations and Technology: Control of Pathogens and Vector Attraction in Sewage Sludge," EPA/625/R-92/013.
35. Volpe, R. L., and W. E. Kelley (1985), *Seepage and Leakage from Dams and Impoundments,* New York, American Society of Civil Engineers.
36. Water Pollution Control Federation (1988), "Incineration, Manual of Practice," OM-11.

Chapter 2

Biosolids Characteristics and Production Rates

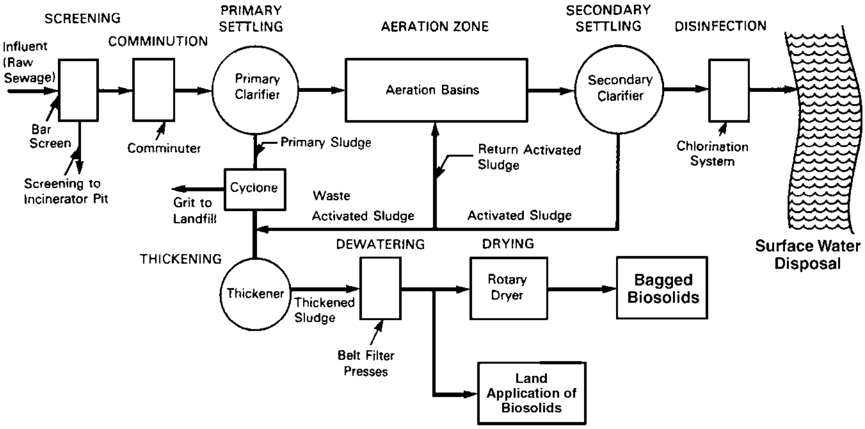
2.0 Introduction

Biosolids are the residual solids generated from the processing of municipal wastewater (excluding grit and bar screenings) and domestic septage that meet the regulatory requirements for recycling (i.e., beneficial use) specified in the 40 CFR Part 503 rule. Once generated, beneficial use (i.e., land application of biosolids for agricultural or aesthetic purposes) represents a cost-effective disposal option for biosolids. To properly design biosolids beneficial-use systems, it is important to understand the impact of wastewater quality objectives on biosolids characteristics and production rates (Fig. 2.1).

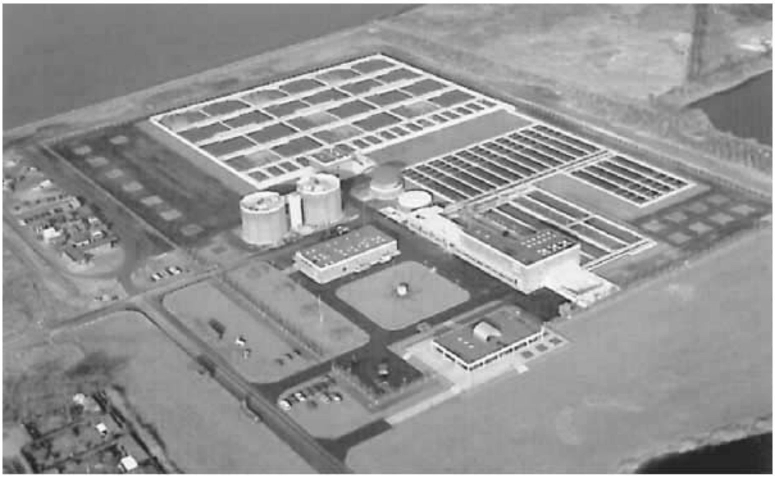
Municipal wastewater consists of liquid wastes produced in residences, commercial establishments, industries, and any subsurface, surface, or storm water that enters the municipal wastewater collection system. Depending on the type and extent of wastewater treatment, any of the materials that enter the municipal wastewater collection system ultimately may find their way into biosolids.

2.1 Wastewater Quality

Effective treatment of wastewater requires an accurate knowledge of its influent quality. Since wastewater influent is not constant in character from place to place nor from time to time, the biosolids resulting from its treatment would be expected to vary as well. The principal constituents of wastewater are summarized in Table 2.1.



(a)



(b)

Figure 2.1 (*a*) Schematic diagram of municipal wastewater treatment facility. (*b*) Photograph of oxidation ditch wastewater treatment plant. (*Courtesy of I. Kruger, Inc.*)

Depending on the concentrations of these constituents (also called pollutants), a wastewater may be designated as weak, medium, or strong. Typical pollutant concentrations for domestic wastewater are given in Table 2.2.

The following sections provide brief descriptions of wastewater characteristics that affect the quantity and quality of biosolids generated at municipal wastewater treatment plants. For additional information pertaining to the measurement of specific wastewater parameters, the reader is referred to the following references [21,25,28,31,58].

TABLE 2.1 Physical, Chemical, and Biological Constituents of Domestic Wastewater*

	Chemical constituents		
Physical constituents	Organic	Inorganic	Biological constituents
Color	Carbohydrates	Alkalinity	Animals
Odor	Oils and grease	Chlorides	Plants
Solids	Pesticides	Heavy metals	Bacteria
Temperature	Priority pollutants	Nitrogen	Viruses
	Surfactants	Phosphorus	Protozoa
	VOCs	Sulfur	
		pH	

*Adapted from refs. [21,25].

TABLE 2.2 Typical Pollutant Concentrations of Domestic Wastewater*

	Concentration (mg/liter)		
Pollutant	Weak	Medium	Strong
Total solids	350	720	1200
Dissolved	250	500	850
Suspended	100	220	350
Settable solids	5	10	20
Biochemical oxygen demand	110	220	400
Chemical oxygen demand	250	500	1000
Nitrogen (total as N)	20	40	85
Organic	8	15	35
Ammonia	12	25	50
Nitrite	0	0	0
Nitrate	0	0	0
Phosphorus (total as P)	4	8	15
Organic	1	3	5
Inorganic	3	5	10
Alkalinity (as $CaCO_3$)	50	100	200
Total coliforms (per 100 ml)	10^6–10^7	10^7–10^8	10^7–10^9

*Adapted from ref. [28].

2.1.1 Wastewater solids

The *total solids content* of wastewater is defined as all the residue that remains after heating a wastewater sample for at least 6 hours at a temperature in the range of 103 to 105°C (217–221°F) [28]. In this temperature range, all free water evaporates from the sample, leaving only the wastewater solids.

Alternatively, the wastewater solids can be differentiated first as either nonfilterable (suspended) or filterable by passing a known volume of wastewater through a 1.2-μm filter [60]. The residue caught on the filter and weighed after drying at 103 to 105°C (217–221°F) is known as *suspended solids* and typically is reported in units of milligrams per liter (mg/liter). The *filterable solids* fraction is that portion of the solids that passes through the filter and is weighed after drying at 103 to 105°C for at least 6 hours. Filterable solids typically consist primarily of colloidal material and dissolved solids [60].

Both suspended and filterable solids may be further classified on the basis of their volatility at 550°C (1022°F). At this temperature, the organic fraction will oxidize and be driven off as carbon dioxide (CO_2) and water vapor (H_2O), whereas the nonvolatile inorganic fraction remains behind as ash. The terms *volatile suspended solids* and *fixed suspended solids* refer to the organic and inorganic (or ash) content of the suspended solids, respectively.

Another common term used to describe the volume of solids removed in solid-liquid separation devices is *settleable solids*. Settleable solids represent the volume of solids that will settle within a given time (typically 30 minutes) within a precalibrated vessel. The settleable solids measurement is often used as a process control parameter in the operation of clarification systems (see Sec. 2.5) and in sizing biosolids-handling equipment. Settleable solids normally are reported in units of milliliters of solids per liter of wastewater (ml/liter). Figure 2.2 is a photograph of a settleometer (a device used to measure settleable solids).

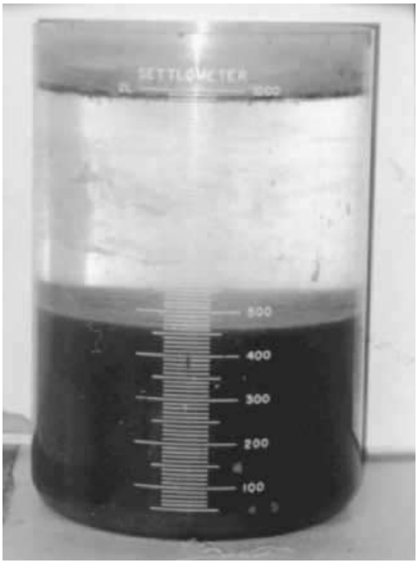


Figure 2.2 Settleometer used in wastewater treatment.

2.1.2 Odors

Gases that are produced by the decomposition of organic material normally generate odors in municipal wastewater. The most characteristic odor of anaerobic wastewater is that of hydrogen sulfide, which is produced by dissimilatory sulfate reduction [41]. During the evaluation of wastewater pretreatment programs, it should be recognized that industrial sewer discharges also may contain compounds that release odors during wastewater treatment [8,31,40].

Historically, the potential for generating objectionable odors has typically been cited as the primary concern of the public with regard to the siting of wastewater treatment facilities. Similarily, proposals for the land application of biosolids are routinely met with public skepticism regarding odor generation [48,52].

2.1.3 Organic matter

Organic matter in wastewater is composed of various combinations of carbon, hydrogen, oxygen, and nitrogen and, in some cases, sulfur and phosphorus. The principal groups of organic matter found in wastewater are proteins (40–60 percent), carbohydrates (25–50 percent), and fats and oils (10–15 percent). Along with these groups, municipal wastewater contains trace quantities of synthetic organic compounds. Typical examples of synthetic organic compounds include industrial surfactants (i.e., detergents), hazardous organic pollutants, and volatile organic compounds (VOCs) [8,31].

2.1.3.1 Measurement of organic matter. The parameter used most widely to describe the organic matter content of a municipal wastewater is the 5-day *biochemical oxygen demand* (BOD_5). This parameter requires monitoring the change in the dissolved oxygen concentration during the microbial oxidation of organic matter [28]. Nonbiodegradable, or recalcitrant, organic material will not be quantified using the BOD_5 test because these compounds will not exhibit a microbial oxygen demand.

In addition to the biodegradable organic matter, other species in wastewater (e.g., ammonia) can exhibit a microbial oxygen demand. The microbial oxygen demand associated with the oxidation of ammonia to nitrate is called the *nitrogenous biochemical oxygen demand* (NBOD). The effect of nitrogenous biochemical oxygen demand on BOD_5 measurement is depicted in Fig. 2.3. The interference to the BOD_5 test caused by the presence of nitrifying bacteria can be eliminated by pretreatment of the sample. Typical pretreatment procedures include pasteurization, chlorination, and acid treatment [7,21,25].

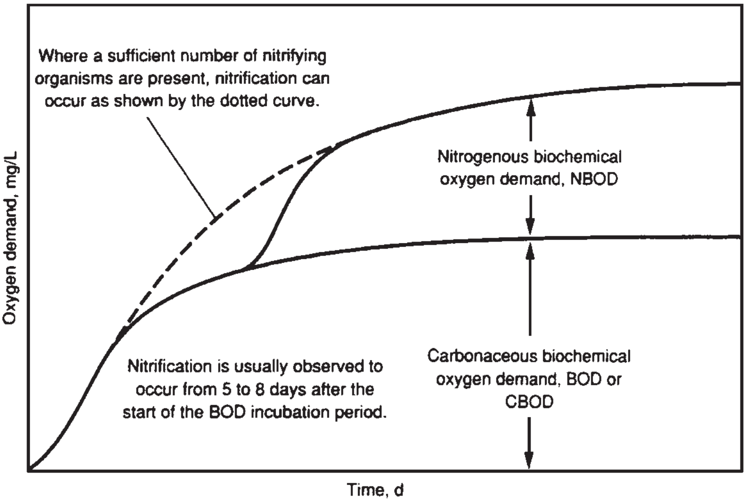


Figure 2.3 Behavior of carbonaceous versus nitrogenous oxygen demand during a typical BOD_5 test.

The *chemical oxygen demand* (COD) is another method used to estimate the organic content of municipal wastewater. In this approach, the oxygen equivalent of the organic matter is estimated by quantifying the reduction of a strong chemical oxidizing agent (e.g., potassium dichromate). The test is performed at elevated temperatures, and a catalyst (e.g., silver sulfate) is required to enhance the chemical oxidation of certain organic compounds [25]. The COD level of a wastewater is always equal to or greater than its BOD_5 measurement because a greater number of compounds can be oxidized chemically than can be oxidized biologically. However, for many wastewaters, it is possible to correlate the COD level with the BOD_5 measurement. This correlation can be a cost-effective alternative for monitoring wastewater treatment plant operations since a COD measurement is less expensive to conduct than the BOD_5 test and results can be reported in approximately 3 hours compared with 5 days for BOD_5 [25].

2.1.4 Inorganic wastewater parameters

The hydrogen ion activity is one of the most important inorganic wastewater quality parameters. The hydrogen ion activity normally is expressed as pH, which is defined as the negative logarithm of the hydrogen ion activity [Eq. (2.1)]. Under most circumstances, the ionic strength of the water is relatively low and therefore, the use of the hydrogen ion concentration rather than activity in estimating the pH of wastewater leads to negligible errors.

$$\mathrm{pH} = -\log \{\mathrm{H}^+\} \tag{2.1}$$

The importance of wastewater pH on the performance of biological wastewater treatment operations stems from the fact that the range of pH over which the proper microbial population in secondary wastewater treatment operations can exist is quite narrow (e.g., pH 6–9). Wastewaters with pH levels that vary significantly from neutrality (i.e., pH of 7.0) are difficult to treat using biological unit operations. Moreover, wastewaters with extreme pH values (i.e., below 5.5 or above 9.5) are not only difficult to treat biologically but require substantially greater amounts of neutralization chemicals during primary wastewater treatment. Finally, wastewater pH also enhances the volatilization loss of some constituents. For example, at pH levels above 8.0, ammonia will tend to volatilize from wastewater, whereas at pH levels below 6.0, hydrogen sulfide will partition into the gas phase.

A parameter that is related to wastewater pH is alkalinity. *Alkalinity* is defined as the acid-neutralizing, or buffering capacity, of the wastewater and normally is reported in equivalent units of calcium carbonate (e.g., mg/liter $CaCO_3$). Alkalinity in wastewater results from the presence of the hydroxides, carbonates, and bicarbonates species. In most cases, calcium and magnesium bicarbonates are the main contributors to wastewater alkalinity. Because of the inherent alkalinity in most water supplies and the materials added during domestic use, influent wastewater is normally alkaline with a significant buffering capacity [25,28]. However, if insufficient alkalinity is present for adequate chemical treatment, supplemental alkalinity (normally in the form of lime) must be added.

2.1.5 Nutrient levels in wastewater

Under normal conditions, the municipal wastewater treatment plant influent flow typically contains 20 to 85 mg/liter of total nitrogen [28]. Municipal wastewater nitrogen consists of approximately 60 percent ammonium nitrogen, 40 percent organic nitrogen, and trace quantities of nitrate [25]. Ammonia nitrogen exists in aqueous solution as either the ammonium ion (NH_4^+) or free ammonia (NH_3) depending on the pH of the solution. At wastewater pH levels above 8.0, free ammonia predominates, whereas below this value, the dominant form of ammonia is the ammonium ion.

Although nitrate concentrations in influent wastewater are small, effluent nitrate concentrations can be significant. A major environmental concern pertaining to wastewater treatment plant effluent nitrate levels is eutrophication. *Eutrophication* (i.e., undesirable fertilization of surface waters) results in excessive aquatic plant growth, leading to the deterioration in surface water appearance and quality. In addition to eutrophication, nitrate and nitrite in discharged wastewater treatment plant effluent constitute a public health concern related to methemoglobinemia and carcinogenesis [15,60,61]. While methemoglobinemia, commonly referred to as "blue baby disease" because of the color that any afflicted infant will turn if affected, is caused by the preferential uptake of nitrate by hemoglobin, carcinogenesis (primarily gastric cancer) has been associated with the ingestion of *N*-nitroso compounds from water supplies contaminated with nitrates [28,60].

Phosphorus is present in influent wastewater primarily in the forms of orthophosphate, polyphosphate, and organic phosphate. Municipal wastewater may contain from 4 to 15 mg/liter of total phosphorus [25]. Like nitrogen, significant concentrations of phosphorus in the wastewater treatment plant effluent can result in eutrophication of surface waters.

Sulfate occurs naturally in most water supplies and therefore is present in municipal wastewater. Sulfate is reduced to sulfide in the anaerobic digestion process and may cause severe biological inhibition as well as odor complaints [41]. The presence of organic sulfur in wastewater occurs as a result of protein degradation. Organic sulfur may be metabolized by microorganisms, leading to the release of malodorous volatile compounds, including hydrogen sulfide [28].

2.1.6 Toxic inorganic compounds

Some toxic anions (i.e., negatively charged species), including cyanides, fluoride, and chromates, are present in industrial wastes. These materials normally are found in wastewater discharged from metal plating, electronics manufacturing, and paint-finishing industries [8,36,38]. To ensure an adequate level of microbial activity at the wastewater treatment plant, excess quantities of these toxic materials must be removed through industrial pretreatment programs (i.e., establishment of local wastewater limits; see Chap. 4).

Heavy metals, such as nickel, manganese, lead, chromium, cadmium, zinc, copper, and mercury, are important inorganic constituents that may be present in municipal wastewater as cations (i.e., positively charged species) or as inorganic salts. The presence of these toxic metals in raw wastewater will result in their presence in biosolids. If present in excessive quantities, they can limit the beneficial use of biosolids. Pretreatment programs therefore must be designed to limit the loadings of these materials to the wastewater treatment plant to a level that is protective of plant unit operations as well as effluent wastewater and biosolids quality.

2.1.7 Wastewater pathogens

Pathogens are organisms known to cause disease. The organisms in raw wastewater that are of greatest concern to human health are the enteric bacteria, viruses, and the intestinal parasites (Table 2.3).

TABLE 2.3 Principal Pathogens of Concern in Municipal Wastewater*

Organism	Disease/symptom
Bacteria	
Salmonella sp.	Salmonellosis (food poisoning), typhoid fever
Shigella sp.	Bacillary dysentery
Yersinia sp.	Acute gastroenteritis
Vibrio cholerae	Cholera
Campylobacter jejuni	Gastroenteritis
Escherichia coli	Gastroenteritis
Enteric viruses	
Hepatitis A virus	Infections hepatitis
Retaviruses	Acute gastroenteritis
Echoviruses	Meningitis, paralysis, diarrhea
Reovirus	Respiratory infections
Astrovirus	Gastroenteritis
Protozoa	
Cryptosporidium	Gastroenteritis
Entamoeba histolytica	Acute enteritis
Giardia lamblia	Giardiasis
Balantidium coli	Dysentery
Helminth worms	
Ascaris lumbricoides	Abdominal pain, vomiting
Ascaris suum	Chest pain, fever
Trichuris trichiura	Abdominal pain, diarrhea
Toxacara canis	Abdominal pain, fever

*Adapted from refs. [28,33,52].

Typical removal efficiencies of human pathogens in primary and secondary wastewater treatment operations are summarized in Table 2.4. It should be noted that the total and fecal coliforms are not, by definition, pathogenic organisms but, rather, are categorized as indicator organisms. In other words, their detection in wastewater indicates the potential presence of pathogens.

2.2 Biosolids Quality

Reliable information on biosolids quality is essential when developing beneficial-use programs. A number of factors influence biosolids quality, including (1) the proportion of industrial and residential discharges to the municipal wastewater collection system, (2) the amount of urban runoff into the municipal wastewater collection system, and (3) the combination of wastewater and biosolids treatment processes employed at the wastewater treatment plant [16,49,52]. Therefore,

TABLE 2.4 Pathogen Removal Efficiencies in Wastewater Treatment Processes*

Microorganisms	Primary treatment removal (%)	Secondary treatment removal (%)
Total coliforms	<10	90–99
Fecal coliforms	35	90–99
Shigella sp.	15	91–99
Salmonella sp.	15	96–99
Escherichia coli	15	90–99
Viruses	<10	76–99
Entamoeba histolytica	10–50	10

*Adapted from refs. [28,58].

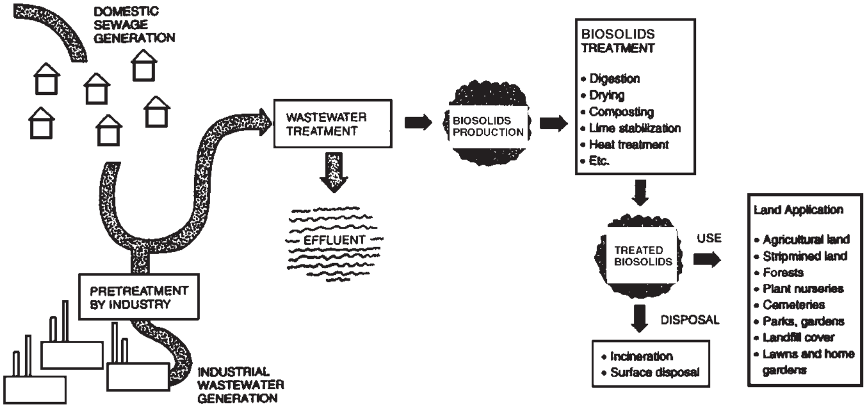


Figure 2.4 Biosolids generation, treatment, use, and disposal.

like wastewater, biosolids quality is variable from one location to another and over time at a specific wastewater treatment plant. Figure 2.4 illustrates the generation, treatment, use, and disposal of biosolids within a community.

The variability of biosolids composition emphasizes the need for a reliable sampling and analysis program. After a biosolids beneficial-use program has been initiated, a continuing biosolids sampling and analysis program is required to verify compliance with the 40 CFR Part 503 regulations.

Organic content, nutrients, metal content, and pathogens are important characteristics of biosolids that affect the final disposal options. The following sections examine each of these characteristics with respect to their impact on beneficial use of biosolids. It should be noted that, in most cases, the composition of biosolids does not follow a normal distribution because of the variability in the specific nature of industrial and other nondomestic inputs into the wastewater treatment plant. Several studies have shown that the median and geometric mean are better measures of "typical" biosolids pollutant concentrations than the arithmetic mean [32,52].

2.2.1 Organic content

The organic content of biosolids, which consists of microbial degradation products, chemical compounds present in the wastewater influent (i.e., proteins, carbohydrates, greases, fats, etc.), and synthetic organic compounds, normally ranges from 6 to 48 percent (dry-mass basis) [7,28]. Most of the organic carbon found in biosolids consists of various proteinaceous, carbohydrate, and lipid-type materials in various stages of decomposition.

Because of their impact on biosolids beneficial use, a number of synthetic organic compounds, primarily industrial in origin, have been receiving greater emphasis as potential pollutants of soils and crops [57]. Chlorinated hydrocarbons, phenols, pesticides, and polychlorinated biphenyls (PCBs) have all been detected in biosolids (Table 2.5).

TABLE 2.5 Toxic Organic Compounds Detected (mg/kg dry weight) in Biosolids from 238 Treatment Plants in Michigan*

Compound	Range (ppm)	Mean (ppm)	Median (ppm)
Acrylonitrile	4–82	16	7
Chlorobenzene	60–846	337	106
p-Chlorotoluene	93–324	153	121
o-Dichlorobenzene	6–108	89	16
m-Dichlorobenzene	6–1651	119	22
p-Dichlorobenzene	10–633	77	23
1,2-Dichloropropane	0.09–66	1.91	0.66
1,3-Dichloropropane	0.6–309	18	3.2
Ethyl benzene	1.2–66	25	20
Hexachloroethane	0.05–16.5	0.7	0.2
Pentachloroethane	0.4–9.2	2.7	1.3
Styrene	99–5848	1338	405
Tetrachloroethylene	1–1218	68	29
1,2,3-Trichlorobenzene	1–152	25	1
1,2,4-Trichlorobenzene	3–51	14	13
1,2,3-Trichloropropane	9–19	14	14
m-Chlorophenol	0.1–93	9	0.9
p-Chlorophenol	0.1–90	18	3.6
o-Cresol	0.2–183	25	2.0
2,4-Dichlorophenol	0.2–203	25	4.8
2,4-Dimethylphenol	0.09–87	6.5	2.2
4,6-Dinitro-*o*-cresol	0.2–187	12.7	2.3
Hydroquinone	0.1–223	8	2.6
Pentachlorophenol	0.2–8495	81	5.0
Phenol	0.05–288	9	2.0
2,4,6-Trichlorophenol	0.2–1333	42	4.8

*Adapted from ref. [32].

During the development of the 40 CFR Part 503 rule, the U.S. Environmental Protection Agency (USEPA) conducted extensive human and environmental risk assessments of biosolids land-application systems [48,52]. Their results indicated that the most important pathways of environmental exposure to toxic organic compounds contained in biosolids included (1) direct ingestion of biosolids, (2) human consumption of meat from animals grazing in pastures treated with wastewater biosolids, and (3) predators consuming biota living in biosolids-amended soil. However, with regard to plant uptake, the USEPA concluded that most toxic organic compounds are so strongly adsorbed to the biosolids-soil matrix following land application that their bioavailability to plants was negligible [5].

As a result of the USEPA research efforts, toxic organics were unregulated in the first round of development of the 40 CFR Part 503 biosolids land-application criteria [39]. During round two of the rule development, additional risk-assessment studies have focused on the environmental and human health risks associated with (1) chlorinated dibenzo-*p*-dioxins and (2) dibenzofurans (CDFs) found in biosolids [57]. At the time of this text preparation, the USEPA had issued a proposed standard for dioxin in land-applied biosolids. The standard, which limits the concentration of dioxin to no more than 300 parts per trillion (ppt) toxic equivalents (TEQ), has yet to be codified into law.

2.2.2 Nutrients

In many cases, biosolids can be used as a low-cost alternative to chemical fertilizers. The rate at which biosolids are applied (known as the *agronomic rate*) is a function of the biosolids' nutrient content and the crop uptake rate of nutrients [13,16,48,52]. Table 2.6 lists typical nutrient values of wastewater biosolids. Although all the nutrients listed in Table 2.6 are required for vegetative growth, it is nitrogen

TABLE 2.6 Nutrient Levels in Wastewater Biosolids*

Nutrient	No. of samples	Range	Median†	Mean†
Total nitrogen	191	<0.1–17.6	3.30	3.90
Ammonia	103	5×10^{-4}–6.76	0.09	0.65
Nitrate	43	2×10^{-4}–0.49	0.01	0.05
Phosphorus	189	<0.1–14.3	2.30	2.50
Potassium	192	0.02–2.64	0.30	0.40
Sodium	176	0.01–3.07	0.24	0.57
Calcium	193	0.1–25.00	3.90	4.90
Iron	165	<0.1–15.30	1.10	1.30

*Adapted from refs. [32,33,52].
†Percent dry-solids basis.

and, in some cases, phosphorus that have the greatest impact on biosolids land-application programs.

2.2.2.1 Nitrogen content. The concentrations of organic nitrogen, ammonia, and nitrate in biosolids are affected by the type of biosolids treatment and handling process employed at the wastewater treatment plant. Most of the organic nitrogen in biosolids is associated with the solid fraction, and thus organic nitrogen levels are not changed significantly by mechanical thickening or dewatering processes. In contrast, the inorganic forms of nitrogen (i.e., NH_4^+ and NO_3^-) are water-soluble, and their concentrations will decrease significantly during some conditioning (e.g., elutriation) and mechanical dewatering processes (e.g., filtration, centrifugation, etc.). In some passive dewatering operations (e.g., drying beds, biosolids lagoons), nitrogen may be lost through ammonia volatilization, whereas the nitrate levels in these systems remain unaffected.

The organic nitrogen compounds found in primary biosolids are derived principally from the biodegradation of proteins, whereas the organic nitrogen associated with secondary biosolids is generated from the biodegradation of proteins as well as the synthesis of new microbial cells [28]. After biosolids application to soils, soil microbes are responsible for metabolizing the organic matter contained in biosolids, resulting in the release of ammonia (a process known as *nitrogen mineralization*). Ammonia may be assimilated by vegetation or transformed into nitrite/nitrate by aerobic soil microorganisms. The degree to which organic nitrogen is mineralized in soils depends on the extent of biosolids processing (e.g., digestion, composting, etc.) occurring at the wastewater treatment plant. In general, land application of less stabilized biosolids (i.e., those biosolids that have not been processed for pathogen and/or volatile solids reduction) will result in greater organic nitrogen mineralization [32,33,52].

Although the application of biosolids to agricultural land has been demonstrated to be an environmentally acceptable means of supplying plant nutrients, a potential limitation to this practice is nitrate leaching to the underlying groundwater (see Chap. 5). Of critical importance is the recognition that the organic nitrogen mineralization process occurs whenever soil conditions are favorable, which is normally a longer period than the duration of nitrogen uptake by vegetation (i.e., growing period). Therefore, nitrate leaching may be significant during the nongrowing period if the biosolids application rate does not appropriately account for the crop nutrient requirements. For sites receiving repeated biosolids applications, measurement of residual soil nitrate levels prior to biosolids application and adjusting the application rate, if necessary, are an appropriate nitrogen management strategy [52].

The biosolids land-application rates described in the 40 CFR Part 503 rule dictate that bulk biosolids cannot be applied at a rate that exceeds the agronomic rate. The *agronomic rate* is defined as the minimum biosolids application rate that results in meeting the crop or vegetation nutrient requirements. By applying biosolids at the agronomic rate, the amount of nitrogen that leaches to the underlying groundwater is minimized [16,52].

2.2.2.2 Phosphorus content. Phosphorus in biosolids exists in both inorganic and organic forms. Organic phosphorus must undergo mineralization in the soil before it is available for plant uptake [16,45]. Total phosphorus application rates generally are much higher than crop requirements when the biosolids application rate is based on the biosolids' nitrogen content [52]. The imbalance between nitrogen and phosphorus in biosolids can result in significantly increased soil phosphorus levels when application rates are based on nitrogen. The accumulation of phosphorus in soil may have several negative consequences, including (1) increasing the potential of phosphorus runoff into surface waters and (2) immobilization of plant micronutrients. To avoid soil phosphorus accumulation, it has been suggested that biosolids application rates be based on the phosphorus content of the biosolids and crop phosphorous demand rather than on the biosolids nitrogen content [52]. This approach would alleviate the increase in soil phosphorus concentrations but would greatly increase the amount of land required for biosolids disposal.

Finally, because of the impact of microbial activity on nutrient mobility, biosolids subjected to aerobic stabilization processes at the wastewater treatment facility (e.g., aerobic digestion, composting) will yield significantly different nutrient levels than biosolids subjected to anaerobic stabilization processes (e.g., anaerobic digestion). Table 2.7 provides data on the typical nutrient concentrations found in biosolids as a function of the general type of biosolids stabilization processing occurring at the wastewater treatment plant.

2.2.3 Metal content

Biosolids contain varying concentrations of metals, including potassium (K), calcium (Ca), magnesium (Mg), sodium (Na), copper (Cu), zinc (Zn), molybdenum (Mo), iron (Fe), manganese (Mn), and cobalt (Co). Some metals (e.g., sodium and potassium) enter the wastewater treatment plant as soluble cations and do not form either insoluble chemical precipitates or stable complexes. As a result, the majority of these metals are discharged in the treated wastewater effluent. Moreover, since these metal species are water-soluble, certain dewatering processes (e.g., centrifugation, belt filter press, filter press) will lower their concentrations

TABLE 2.7 Biosolids Nutrient Levels from Various Treatment Processes*

Nutrient	Biosolids stabilization process†	Mean concentration‡	Range‡
Organic carbon (%)	Anaerobic	27.6	18–39
	Aerobic	31.7	27–37
Total nitrogen (%)	Anaerobic	5.0	0.5–17.6
	Aerobic	4.9	0.5–7.6
Ammonia (mg/kg)	Anaerobic	9400	120–67,600
	Aerobic	950	30–11,300
Nitrate (mg/kg)	Anaerobic	520	2–4900
	Aerobic	300	7–830
Total phosphorus (%)	Anaerobic	3.3	0.5–14.3
	Aerobic	2.9	1.1–5.5
Potassium (%)	Anaerobic	0.52	0.02–2.64
	Aerobic	0.46	0.08–1.10
Sodium (%)	Anaerobic	0.70	0.01–2.19
	Aerobic	1.11	0.03–3.07
Calcium (%)	Anaerobic	5.8	1.9–20.0
	Aerobic	3.3	0.6–13.5

*Adapted from ref. [33].

†Aerobic processes are those in which molecular oxygen is present (e.g., composting), whereas anaerobic processes are those devoid of oxygen (e.g., anaerobic digestion).

‡Concentrations and percent composition are on a dry-solid basis.

significantly in biosolids. Figure 2.5 is a schematic diagram of a typical filter-press device used to dewater thickened sludge or biosolids.

Conversely, dewatering processes that rely on water evaporation (e.g., drying bed, biosolids lagoon) will result in increased concentrations of these metals in the final biosolids product. Figure 2.6 shows a typical sludge/biosolids drying bed.

Depending on pH, redox potential, oxygen availability, and composition, a number of inorganic metal precipitates can form in biosolids, including hydroxides, oxides, carbonates, phosphates, and sulfides [32]. Many heavy metals, such as cadmium, coprecipitate to form insoluble compounds in biosolids. For example, cadmium can be enmeshed within aluminum hydroxide [$Al(OH)_3$] or calcium carbonate ($CaCO_3$) precipitates during chemical treatment processes [32,33].

Potential human health and environmental risks from metals in biosolids have been evaluated by the USEPA. USEPA risk-assessments studies have found that the adsorption and precipitation process in soil limit plant uptake of metals [5,9,52]. Moreover, since phytotoxicity (i.e., plant toxicity) from zinc, copper, and nickel occurs in plants well below levels posing a significant risk to livestock or humans, consumption of crops exposed to the typical levels of these metals found in biosolids was determined to represent a minimal risk to public health and the

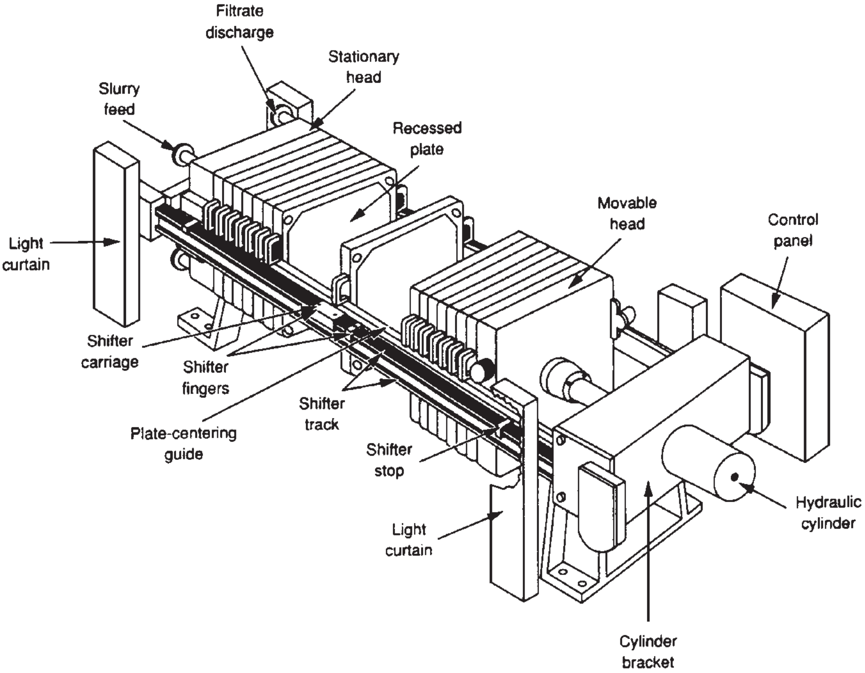


Figure 2.5 Schematic diagram of a filter-press dewatering device. (*Courtesy of EIMCO Process Equipment Company.*)

Figure 2.6 A biosolids/sludge drying bed.

TABLE 2.8 Average Concentration of Heavy Metals in Biosolids Relative to 40 CFR Part 503 Ceiling Concentration Limits*

Metal	Mean concentration† (mg/kg)	40 CFR Part 503 ceiling concentration limits (mg/kg)
Arsenic	9.9	75
Cadmium	6.9	85
Copper	741.0	4300
Lead	134.4	840
Mercury	5.2	57
Molybdenum	9.2	75
Nickel	42.7	420
Selenium	5.2	100
Zinc	1202.0	7500

*Adapted from refs. [45,52].
†Dry-solids basis.

environment. Field and laboratory results on metal uptake by crops and soil biota formed the basis for the metal concentration limits promulgated in the 40 CFR Part 503 regulations. Table 2.8 lists the average concentrations of heavy metals found in biosolids (dry-mass basis) relative to the 40 CFR Part 503 ceiling concentration limits.

2.2.4 Pathogens in biosolids

Pathogenic microorganisms present in raw wastewater would be expected to be found in biosolids. However, their concentrations in biosolids will be reduced significantly due to the stabilization processes to which biosolids are subjected at the wastewater treatment facility. For example, Table 2.9 provides data that illustrate the extent of pathogen reduction that occurs during anaerobic digestion. Additional data on pathogen reduction during other types of biosolids stabilization processes are provided in the following references [28,32,33,58].

TABLE 2.9 Typical Pathogen Levels in Unstabilized and Anaerobically Digested Liquid Biosolids*

Pathogen	Typical concentration in unstabilized liquid biosolids (no./100 ml)	Typical concentration in anaerobically digested biosolids (no./100 ml)
Viruses	2500–70,000	100–1000
Fecal coliforms	1×10^9	3×10^4–6×10^6
Salmonella	8000	3–62
Ascaris lubricoides	200–1000	0–1000

*Adapted from ref. [32].

2.2.4.1 Pathogen survival in biosolids land-application systems. Organisms applied to soil are subjected to a variety of environmental conditions that impact their survivability. Of greatest concern to public health is their potential to survive in soil for extended periods of time. Temperature is an important factor in the survival of bacteria and viruses in the environment. In the case of bacteria, the dieoff rate is approximately doubled with each 10°C (50°F) rise in temperature between 5 and 30°C (41 and 86°F) [58]. In addition to high temperatures, ultraviolet radiation rapidly kills most pathogens exposed to sunlight. Representative survival times for important groups of pathogens are summarized in Table 2.10.

Risk-assessment studies sponsored by the USEPA have resulted in the present harvesting/grazing schedules associated with biosolids application regulations specified in the 40 CFR Part 503 rule [52]. For the various biosolids land-application scenarios, the 40 CFR Part 503 rule specifies the time duration required following biosolids land application after which land can be used for animal grazing or have any human contact.

The level of pathogen reduction achieved during biosolids treatment depends on the type of stabilization processes employed at the wastewater treatment plant as well as their operational conditions (e.g., time, temperature, pH, etc.). For example, it has been reported that heat drying of biosolids can result in reducing the concentration of viruses by as much as five orders of magnitude when the final biosolids contain less than 10 percent moisture [32]. Similarly, in the biosolids composting process, temperatures of 55 to 70°C (140–165°F) will result in the inactivation of pathogenic microorganisms, including protozoan cysts, helminth eggs, and pathogenic bacteria [32,58]. Radiation processing of biosolids is also effective in reducing the numbers of pathogenic microorganisms and viable helminth eggs found in biosolids [58].

TABLE 2.10 Survival Times of Pathogens in Soil*

Organism	Approximate survival time (days)
Fecal coliforms	8–55
Leptospira	15
Streptococcus faecalis	26–77
Enterovirus	70–170
Poliovirus	70–90
Helminth eggs	>1000

*Adapted from ref. [58].

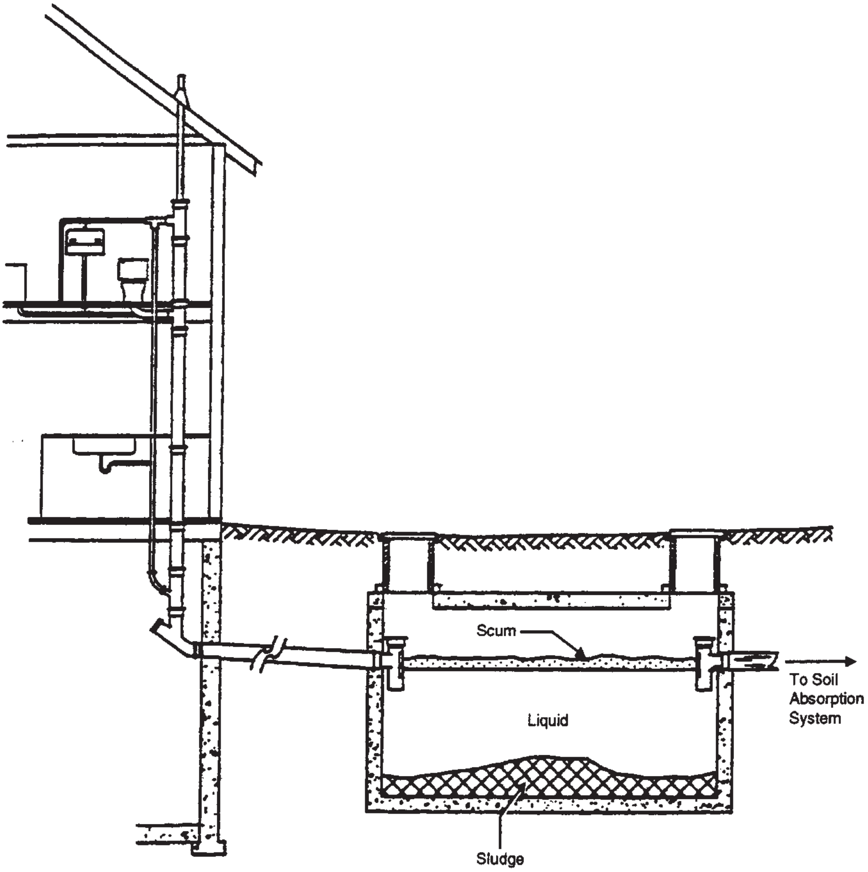


Figure 2.7 Diagram of typical domestic septic tank system.

2.2.5 Septage

Federal regulations (40 CFR Part 503, Subpart O) define *septage* as either a liquid or solid material removed from a septic tank, cesspool, portable toilet, type III marine sanitation device, or similar treatment works that receives only domestic wastes [39]. A typical domestic septic tank system is depicted in Fig. 2.7.

Within the septic tank system, heavy solids (e.g., grit and sand) contained in domestic wastewater settle to the bottom of the tank, while scum (e.g., grease, fats, and floatable matter) accumulates on the wastewater surface. The rate of accumulation of heavy solids and scum in a septic tank is highly variable and depends on many factors, including (1) the number of household members, (2) their personal hygiene, (3) personal eating habits, (4) tank dimensions, (5) the appliances used in the home (e.g., garbage disposals, water softeners, etc.), and (6) local climat [46,50].

As the domestic waste undergoes decomposition within the septic tank, a relatively clear liquid zone called *effluent* is formed in between the scum layer and the heavy solids (i.e., sludge). It is from this effluent zone that wastewater from the septic tank is discharged into a soil-absorption system (termed a *leach* or *drain field*). The soil-absorption system provides the final polishing step for the discharged septic tank wastewater.

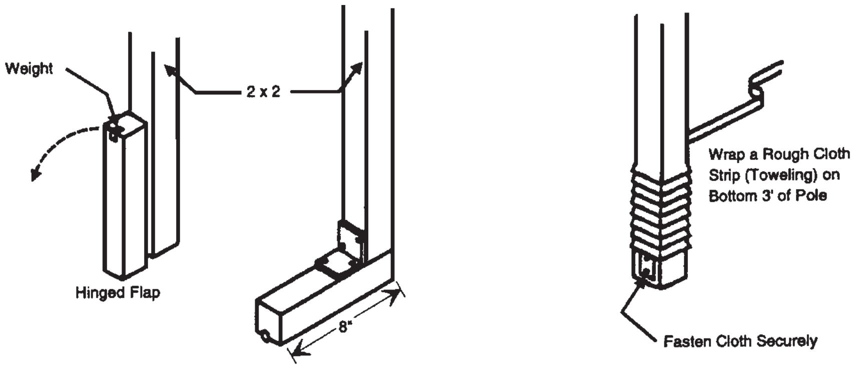
If excessive accumulation of sludge or scum occurs within the septic tank, solids will be discharged together with the partially treated wastewater into the soil-absorption system, resulting in the plugging of drain pipes and/or the clogging of soil. Once drain pipes or soils become clogged, the septic tank system will cease to function properly until a new soil-absorption system is constructed. Therefore, to maintain an effective waste treatment system, regular inspection and pumping of the septic tank are required. It is the material that is pumped from the septic tank that is characterized as *domestic septage*.

A relatively simple and inexpensive inspection program can determine whether or not septic tank pumping is required. Inspection consists of measuring both the depth of the scum and sludge layers and assessing the physical condition of the tank and its components. The depth of the scum layer may be measured using a stick with a hinged flap (Fig. 2.8). The stick is pushed through the scum layer until the flap shifts into the horizontal position. The stick is then raised until it meets resistance at the bottom of the scum layer. By marking the stick at the top of the scum layer, the thickness of the scum layer can be measured. If the bottom of the scum layer is less than 3 in above the bottom of the baffle or outlet tee, the septic tank should be pumped.

Similarly, the depth of the sludge layer may be measured with a long stick to which a cloth strip is fastened (see Fig. 2.8). The measuring stick is lowered through the scum layer near the outlet baffle to the bottom of the septic tank. The sludge depth is estimated by the length of the cloth containing black sludge particles [50]. If the sludge layer is within 3 in of the bottom of the outlet baffle, the septic tank should be pumped.

When septic tank pumping is warranted, septage is normally removed by a specially equipped pumper truck. Pumper trucks typically range in volume capacity from 1000 to 4000 gal (3800–15,000 liters), although multiaxle trucks may have capacities of over 6000 gal (23,000 liters). Figure 2.9 shows a typical septage pumper truck.

Pumps used to remove septage are either vacuum or centrifugal pumps. Vacuum pumps, the most common system employed by septage haulers, have the following advantages over centrifugal pumps: (1) liquid does not flow through the pump, which reduces wear, (2) the pump is less likely to freeze, and (3) the tank contents can be discharged under pressure. Centrifugal pumps used to transfer septage



Device for Measuring Scum Thickness

Device for Measuring Sludge Depth

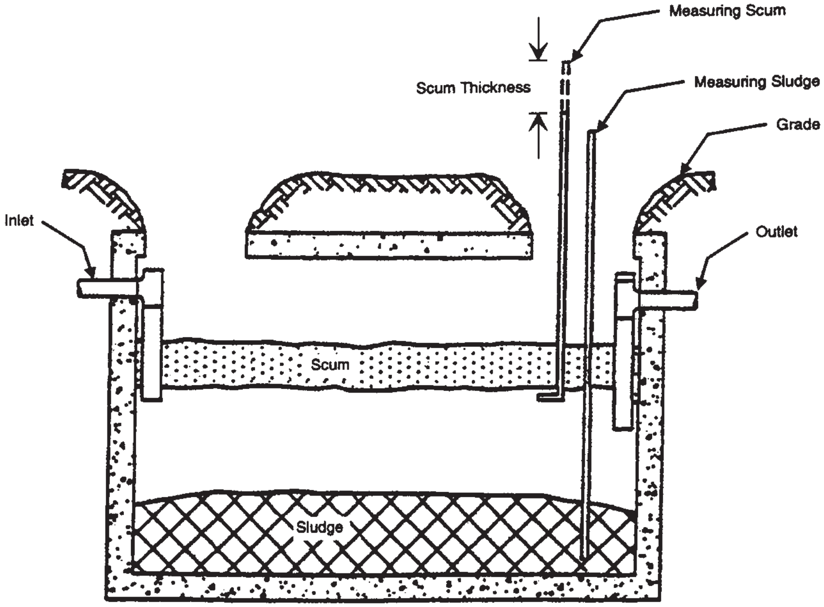


Figure 2.8 Schematic diagram of septic scum and sludge measuring devices.

typically are open-impeller or recessed-impeller pumps, which are preferred for handling high solids flows [34].

Hoses used to transfer septage from septic tanks to pumper trucks should be high-vacuum black rubber or synthetic material with a minimum diameter of 3 in (8 cm). Discharge valves on the hauler trucks should be driptight, and a discharge nipple should accommodate a quick disconnect coupling.

2.2.5.1 Septage characteristics. The characteristics of the pumped septage are highly variable, but typically, septage contains large

(a)

(b)

Figure 2.9 (*a*) A septage pumper truck. (*b*) A septage pumper truck equipped with a subsurface injection system. (*Courtesy of Lely-Pacific, Inc.*)

amounts of grease, grit, hair, and debris. Other characteristics of domestic septage include (1) an objectionable odor, (2) solids that are resistant to settling and dewatering, and (3) a significant potential to foam during aeration. Septage characteristics for conventional wastewater quality parameters, metals, toxic organic compounds, and pathogens are presented in Tables 2.11 through 2.14.

Under federal regulations (40 CFR Part 503), domestic septage is categorized as biosolids and, when properly managed, is a valuable resource. Domestic septage contains nutrients that can reduce reliance on chemical fertilizers as well as organic matter that can improve soil moisture retention capacity [34,46,50]. A good septage management program recognizes the potential benefits of septage and

TABLE 2.11 **Characteristics of Septage: Conventional Wastewater Quality Parameters***

Parameter	Concentration (mg/liter) Average	Minimum	Maximum
Total solids	34,105	1,132	130,475
Total volatile solids	23,100	353	71,402
Total suspended solids	12,862	310	93,378
Volatile suspended solids	9,027	95	51,500
Biochemical oxygen demand (BOD_5)	6,480	440	78,600
Chemical oxygen demand	31,900	1,500	703,000
Total Kjeldahl nitrogen	588	66	1,060
Ammonia nitrogen	97	3	116
Total phosphorus	210	20	760
Alkalinity (as mg/liter $CaCO_3$)	970	522	4,190
Grease	5,600	208	23,368
pH	—	1.5	12.6

*Adapted from ref. [34].

TABLE 2.12 **Typical Metal Concentrations in Septage***

Parameter	Concentration (mg/liter) Average	Minimum	Maximum
Iron	39.3	0.2	2740
Zinc	9.97	<0.001	444
Manganese	6.09	0.55	17.1
Barium	5.76	0.002	202
Copper	4.84	0.01	261
Lead	1.21	<0.025	118
Nickel	0.526	0.01	37
Chromium (total)	0.49	0.01	34
Cyanide	0.469	0.001	1.53
Cobalt	0.406	<0.003	3.45
Arsenic	0.141	0	3.5
Silver	0.099	<0.003	5
Cadmium	0.097	0.005	8.1
Tin	0.076	<0.015	1
Mercury	0.005	0.0001	0.742

*Adapted from ref. [34].

employs practices to maximize these benefits. Table 2.15 provides guidelines for selecting a septage-disposal alternative.

For small rural communities with adequate land area available, land application is clearly the recommended septage-disposal alternative due to its low cost, simplicity, and environmental benefit. Even for larger metropolitan municipalities, land application may be the most cost-effective solution, although land availability is often the major limitation to this alternative. Disposal at an existing wastewater treatment plant is relatively simple and economical, but the long-term

TABLE 2.13 Toxic Organic Compounds Identified in Septage*

Parameter	Concentration (mg/liter) Average	Minimum	Maximum
Methyl alcohol	15.8	1	396
Isopropyl alcohol	14.1	1	391
Acetone	10.6	0	210
Methyl ethyl ketone	3.65	1	240
Toluene	0.17	0.005	1.95
Methylene chloride	0.101	0.005	2.2
Ethylbenzene	0.067	0.005	1.7
Benzene	0.062	0.005	3.1
Xylene	0.051	0.005	0.72

*Adapted from refs. [34,50].

TABLE 2.14 Pathogens Detected in Domestic Septage*

Viruses
  Adenoviruses
  Poliovirus
  Reoviruses
  Hepatitis virus A

Bacteria
  Bacillus cereus
  Vibrio cholera
  Clostridium perfringens
  Clostridium tetani
  Escherichia coli
  Myobacterium tuberculosis
  Salmonella sp.
  Shigella
  Yersinia

Protozoa
  Entamoeba hystolytica
  Giardia lamblia

Nematodes
  Ascaris lumbricoides
  Enterobius vermicularis

Helminths
  Echinococcus multilocularis
  Taenia solium

*Adapted from ref. [34].

viability of this option depends on available plant capacity and projected increases in sewage and septage flows. Independent septage treatment facilities are expensive to build and operate and, therefore, usually are the last resort for a municipality.

2.3 Biosolids Production Rates

To properly develop biosolids beneficial-use programs, the daily rate of biosolids production (on both a volumetric and mass basis) must be estimated. Knowledge of the biosolids production rate is critical in the design of biosolids handling facilities and the establishment of land area requirements. The wastewater treatment processes that are principally responsible for the generation of biosolids are the primary and secondary wastewater unit operations at municipal wastewater treatment

TABLE 2.15 Guidelines for Selecting a Septage-Disposal Alternative*

Community profile	Conditions	Recommended alternative
Small, unsewered rural community	Remote land area available with suitable site and soil conditions	Land application of untreated septage
	Land available but relatively close to neighbors	Land application of alkali-stabilized septage
	Inadequate land area available; wastewater treatment plant with available capacity within 20 miles	Disposal at wastewater treatment plant
Medium-sized, partially sewered community	Remote land area available but relatively close to neighbors	Land application of alkali-stabilized septage
	Inadequate land area available; wastewater treatment plant with available capacity within 20 miles	Disposal at wastewater treatment plant
	Inadequate land area available; no wastewater treatment plant with available capacity	Disposal at independent septage treatment plant
Large, sewered municipality	Wastewater treatment plant with available capacity	Disposal at wastewater treatment plant
	Inadequate land area available; no wastewater treatment plant with available capacity	Disposal at independent septage treatment plant

*Adapted from ref. [50].

plants as well as the settling of solids in domestic septic systems. The solids removed from bar screens and grit-removal systems at wastewater treatment plants are not characterized as biosolids and typically are landfilled or incinerated by the wastewater treatment plant rather than recycled. A typical automatic bar-screen/comminutor system employed at wastewater treatment facilities is illustrated in Fig. 2.10.

Average biosolids quantities generated in the United States as a function of disposal practice are given in Table 2.16. It should be noted that of the five disposal practices listed, only land application is considered a beneficial-use option.

The final volume of biosolids to be disposed from the wastewater treatment plant will be a function of the following parameters: (1) characteristics of the influent wastewater, (2) efficiency of the solid-

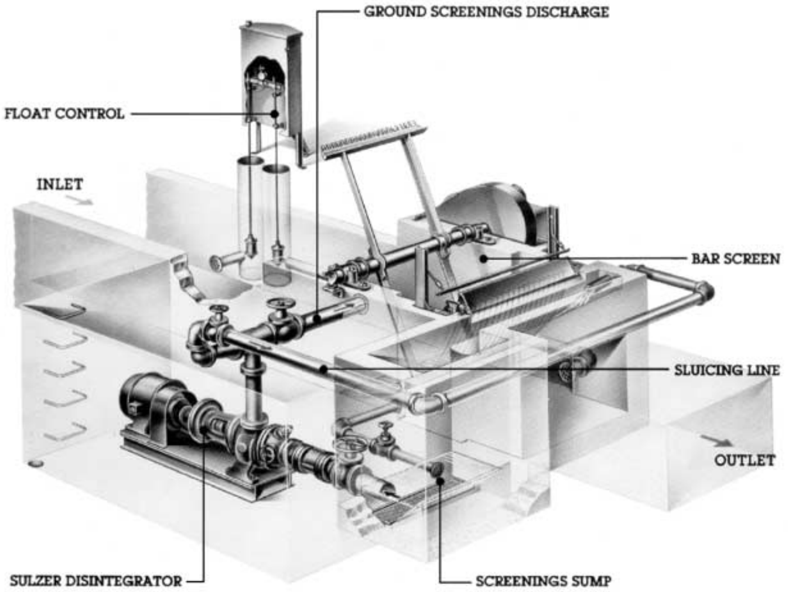


Figure 2.10 Diagram of a bar-screen/comminutor system. (*Courtesy of Dorr-Oliver.*)

TABLE 2.16 Quantity of Biosolids Generated Annually in the United States by Disposal Practice*

Use/disposal practice	Quantity (dry metric tons)	Percentage
Land application	1785.3	33.3
Incineration	864.7	16.1
Landfill	1818.7	33.9
Surface disposal	553.7	10.3
Ocean disposal†	335.5	6.3
TOTAL	5375.2	100.0

*Adapted from refs. [39,47,54].

†The National Sewage Sludge Survey on which these figures are based was conducted prior to the passage of the Ocean Dumping Ban of 1988. Ocean dumping of biosolids ended in June 1992.

liquid separation processes, (3) physical conditions of the deposited solids (e.g., specific gravity and water content), and (4) the period between biosolids removal operations. The following sections describe the various sources of biosolids and provide standard approaches to estimating the biosolids production rate. It should be noted that the residual solids generated in the various unit operations will be referred to as *wastewater sludge.* The term *biosolids* will only apply to that material that meets the specific regulatory criteria described in the 40 CFR Part 503 rule for beneficial use.

TABLE 2.17 Specific Gravity and Concentration of Sludge from Gravity Sedimentation Operations*

Type of sludge	Specific gravity	Settled sludge concentration: Range	Settled sludge concentration: Typical
Primary sludge only			
Medium-strength wastewater	1.03	4–12	6
From combined sewer	1.05	4–12	6.5
Primary and waste-activated sludge	1.03	2–6	3
Primary and trickling-filter sludge	1.03	4–10	5

*Adapted from ref. [28].

2.4 Primary Wastewater Treatment

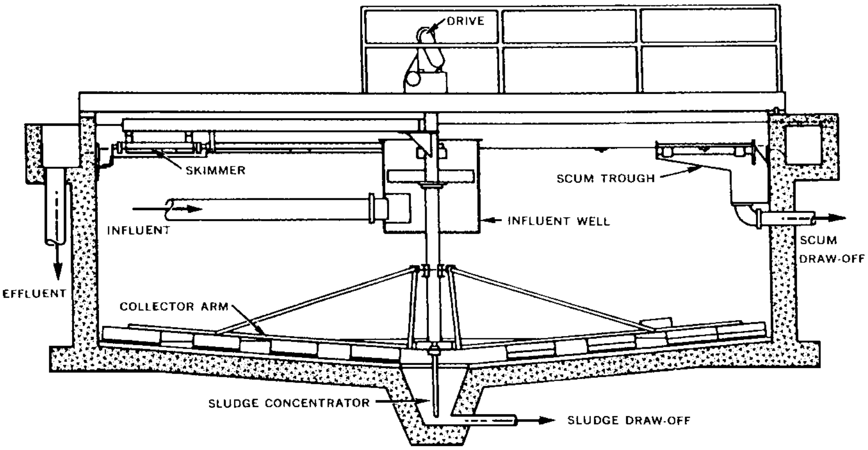
The principal objective of primary wastewater treatment is the removal of suspended solids from incoming wastewater. The removal of suspended solids by settling is termed *sedimentation,* whereas the removal of suspended solids by rising is defined as *flotation*. Most municipal primary wastewater treatment operations employ gravity sedimentation to remove suspended solids. In this process, the difference in specific gravity between liquid water and the suspended solids particles is responsible for facilitating the solid-liquid separation. Data on the specific gravity and solids content of various settled solids (i.e., sludge) are summarized in Table 2.17.

Although flotation is rarely used in the primary treatment of municipal wastewater, it is used frequently in both industrial pretreatment operations (e.g., food processing) and the thickening of secondary sludge. Flotation operations are described under sludge-thickening processes (see Chap. 3).

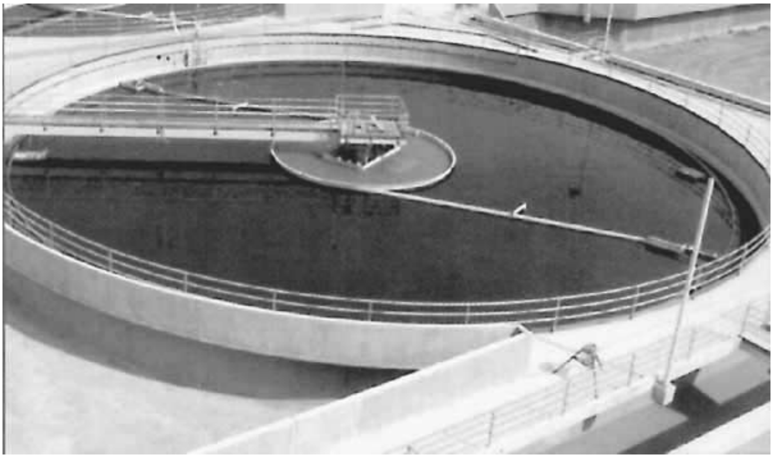
2.4.1 Design of gravity sedimentation systems

A typical gravity sedimentation system consists of a primary clarifier, although it also may include a coagulation/flocculation pretreatment process when chemical addition is employed (Fig. 2.11). The design basis of primary clarification systems is the type of gravity settling behavior that is expected within the sedimentation tank. On the basis of the concentration of suspended solids and their tendency to interact, there are four types of gravity settling that can occur in clarification systems, including (1) discrete-particle settling, (2) flocculent settling, (3) hindered (or zone) settling, and (4) compression settling (Fig. 2.12). During primary sedimentation operations, it is possible to have all four types of settling behavior occurring simultaneously.

2.4.1.1 Discrete-particle settling. Discrete-particle settling characterizes the sedimentation behavior of particles such as sand and silts.



(a)



(b)

Figure 2.11 (*a*) Schematic diagram of a gravity sedimentation system. (*Courtesy of Walker Process Equipment.*) (*b*) Photograph of a primary settling tank. (*Courtesy of WesTech Engineering.*)

Although the removal of these particles normally occurs in grit chambers at wastewater treatment plants, depending on the hydraulic conditions of the plant, a significant portion of these particles may find their way into the primary clarifier. The settling of discrete, nonflocculating particles can be analyzed by the classic sedimentation laws formulated by Stokes and Newton [66]. By equating the gravitational force acting on a falling particle to its frictional drag force, the terminal velocity of the particle can be obtained [29]. If the particle settling under quiescent conditions can be assumed spherical, Stokes' law yields the following terminal settling velocity [Eq. (2.2)]:

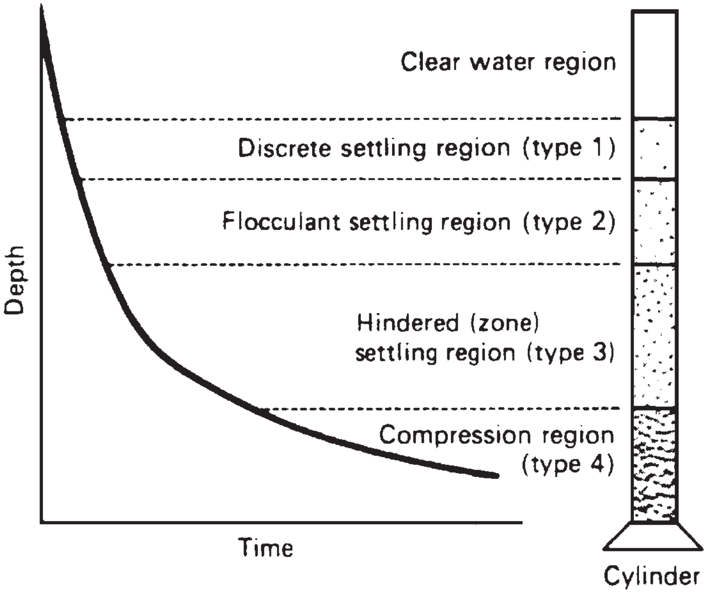


Figure 2.12 Impact of particle concentration on the type of gravity settling.

$$V_c = \frac{g\,(\rho_s - \rho)\,d^2}{18\mu} \tag{2.2}$$

where V_c = terminal velocity of settling particle
ρ_s = density of particle
ρ = density of water
g = acceleration due to gravity
d = diameter of particle
μ = viscosity of water at given temperature

When the design of the sedimentation system is based on discrete-particle settling behavior, the basic approach for sizing the system involves selecting a particle with a terminal velocity V_c and estimating the cross-sectional area of the basin that will result in all particles that have a terminal velocity equal to or greater than V_c being removed. The theoretical relationship between V_c and the overflow rate (or surface-loading rate) of a clarifier is given by Eq. (2.3):

$$V_c = \frac{Q}{A} \text{ (overflow rate or surface loading rate)} \tag{2.3}$$

where V_c = terminal velocity of settling particle (ft/min, m/min)
Q = rate at which clarified water is produced (gal/day, m^3/day)
A = surface area of primary clarifier (ft^2, m^2)

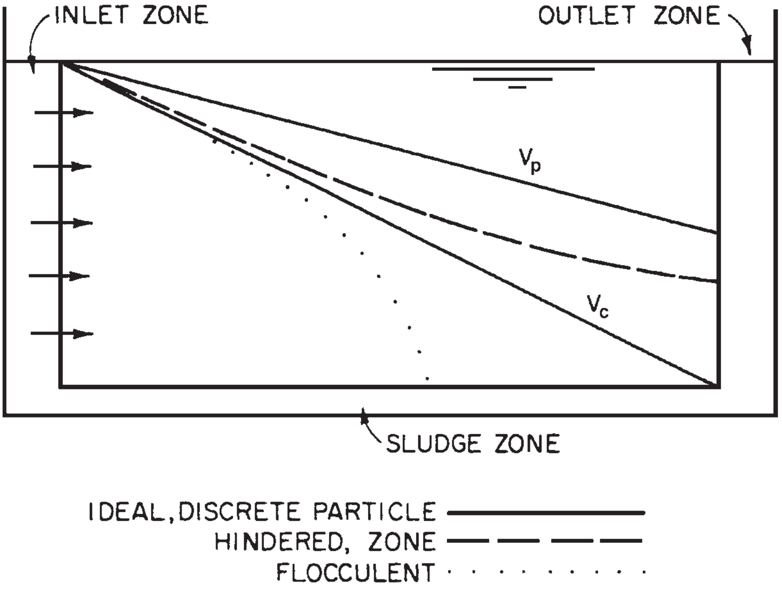


Figure 2.13 Trajectories of particles removed in a primary clarifier. (*Reproduced by permission from ref. [66].*)

It should be noted that although Eq. (2.3) does not explicitly account for basin depth, the depth of the clarifier and the fluid detention time should be maintained such that all particles with the design velocity V_c will settle to the bottom of the tank (Fig. 2.13). In practice, design factors must be adjusted to mitigate the effects of inlet and outlet turbulence, short circuiting, and velocity gradients, as well as to provide volume for sludge storage [60].

Although the clarification basin is designed to completely remove particles having a settling velocity of V_c or greater, a portion of particles with a velocity less than V_c will be removed. This occurs because, in clarifier design, it is assumed that suspended solid particles of various sizes are uniformly distributed over the entire depth of the basin at the clarifier inlet. Particles with a settling velocity less than V_c may be removed if they enter the settling zone at lower depths. Suspended solid particles with settling velocities less than V_c that settle in the primary clarifier are removed in the ratio expressed by Eq. (2.4):

$$X_r = \frac{V_p}{V_c} \tag{2.4}$$

where X_r = fraction of particles with velocity V_p that are removed ($V_p < V_c$)

V_p = particle velocity
V_c = design velocity

For a given surface-loading rate Q/A ($= V_c$), only those particles with settling velocities equal to or greater than V_c will be removed completely, whereas those particles with a velocity less than V_c will be removed in the ratio V_p/V_c. The total fraction of influent suspended solids particles removed in a gravity sedimentation system by discrete particle settling is given by Eq. (2.5).

$$\text{Total fraction of particles removed} = (1 - X_c) + \int_0^{X_c} \frac{V_p}{V_c}\, dx \quad (2.5)$$

where X_c = fraction of particles with velocity less than V_c
$1 - X_c$ = fraction of particles with velocity greater than V_c
$\int_0^{X_c} \frac{V_p}{V_c}\, dx$ = fraction of particles removed with settling velocity less than V_c

To use Eq. (2.5) in estimating the fraction of influent suspended solids particles removed, a laboratory column test must be normally conducted. In the design of the laboratory column, the length should be equivalent to the proposed depth of the sedimentation tank, with the outlet positioned at a depth just above where the actual settled sludge-removal outlet will be located (Fig. 2.14).

Using the laboratory column, the percentage of suspended solids removal is measured as a function of time. These data can then be used to develop a settling-velocity analysis curve that can be employed to estimate the overall suspended solids removal for a given detention

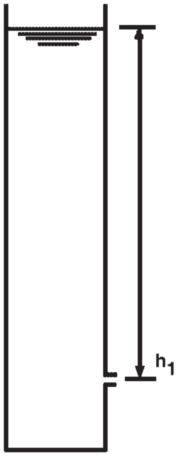


Figure 2.14 Schematic diagram of a laboratory column used for estimating the fraction of suspended solids that settle through a discrete-particle settling mechanism.

time. The development of a settling-velocity curve and its use in estimating the overall fraction of suspended solids removed assuming discrete-particle settling behavior are illustrated in Example 2.1.

Example 2.1 The Walcott County Wastewater Treatment Facility receives a significant portion of its influent wastewater from industrial mining operations. Because of the discharge of fine silty material into the municipal wastewater collection system, the wastewater treatment plant is planning to install a series of new gravity sedimentation units to reduce the solids loading to the downstream unit operations. Assuming that the plant anticipates a maximum surface-loading rate of 3600 gal/ft^2·h, estimate the fraction of particles removed assuming discrete-particle settling behavior if laboratory column tests provided the following data:

Settling velocity (ft/min)	Weight fraction remaining
20	0.60
10	0.50
5	0.32
3	0.25
2	0.10
1	0.05

solution

Step 1. Based on the maximum overflow rate, estimate the critical velocity V_c for particle removal.

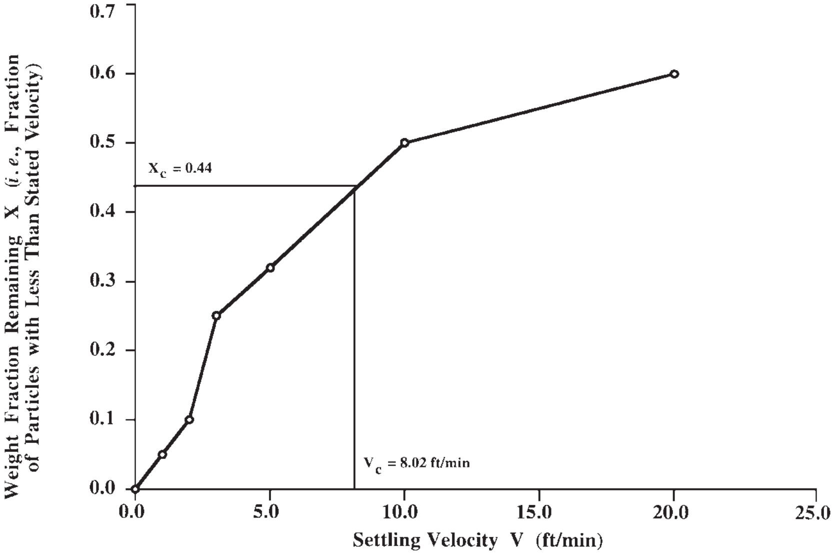
$$V_c = \frac{Q}{A} = \frac{3600 \text{ gal}}{\text{ft}^2 \cdot \text{h}}$$

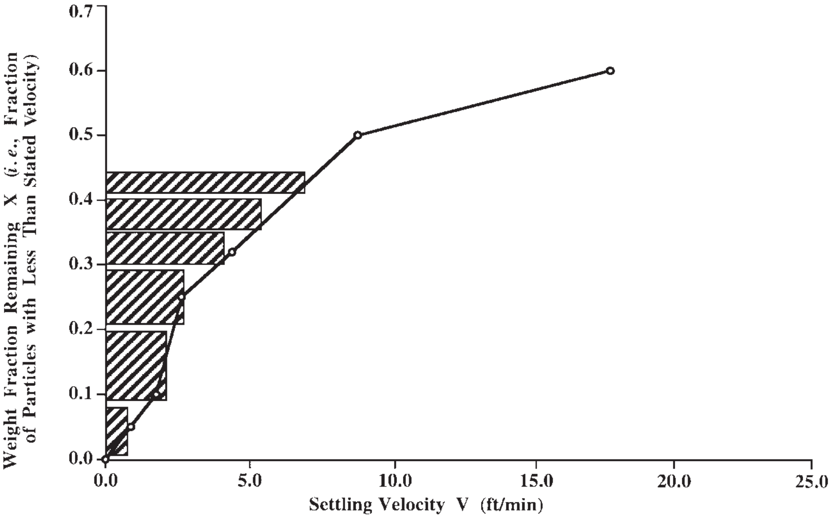
$$= \frac{3600 \text{ gal}}{\text{ft}^2 \cdot \text{h}} \cdot \frac{\text{ft}^3}{7.48 \text{ gal}} \cdot \frac{\text{h}}{60 \text{ min}}$$

$$= \frac{8.02 \text{ ft}}{\text{min}}$$

Step 2. Using the column data, plot a curve of the fraction of particles remaining versus settling velocity, and find the fraction of particles that has a settling velocity of less than 8.02 ft/min (see the first curve on the next page).

From the curve, the fraction of particles with settling velocities less than V_c (8.02 ft/min) is 44 percent. In other words, 66 percent of the particles will have velocities equal to or greater than V_c and will be removed completely. Therefore, X_c is 0.44. The fraction of particles with velocities less than V_c but which are also removed can be evaluated by estimating the area under the curve from X_c equal to 0.44 to X_c equal to 0.00 (see the second curve on the next page).





The following table summarizes the procedure for estimating the area under the curve.

V_p	dX	$V_p\,dX$
8.0	0.03	0.240
6.0	0.04	0.240
4.8	0.05	0.240
3.1	0.08	0.250
2.2	0.10	0.220
0.9	0.06	0.054
TOTAL		1.244

With the area under the curve, the total fraction of particles removed may be estimated using Eq. (2.5).

$$\text{Total fraction of particles removed} = (1 - X_c) + \int_0^{X_c} \frac{V_p}{V_c}\,dx$$

$$= (1 - X_c) + \frac{1}{V_c}\int_0^{X_c} V_p\,dx$$

$$= (1 - 0.44) + \frac{1.244}{8.02}$$

$$= 0.66 + 0.155 = 0.815 \text{ (or 81.5 percent)}$$

2.4.1.2 Flocculent settling. Although some suspended solids will settle as discrete particles, most will flocculate (i.e., coalesce) during the gravity sedimentation process. As flocculation occurs, the mass of the particle increases, resulting in an increase in settling velocity. Therefore, the fraction of solids removed in the primary sedimentation tank (i.e., clarifier) will depend on both detention time and depth.

The extent to which flocculation occurs depends on the opportunity for particle contact, which varies with surface-loading rate, depth of the basin, velocity gradients, particle concentration, and range of particle sizes [66]. The cumulative effect of these variables on particle settling behavior is normally determined by laboratory column tests. Since the extent of particle removal varies with depth, the laboratory column used in flocculent settling tests should have a depth equal to the proposed tank depth, with sampling ports located every 1 to 2 ft (Fig. 2.15).

Prior to the beginning of the settling test, wastewater is introduced into the column and kept continuously stirred. At the initiation of the test, the stirrer is turned off, and the solids are allowed to settle under quiescent conditions. Samples, which are withdrawn from each sampling port at various times, are analyzed for percentage solids removal. Particle removal data are used to generate isoconcentration curves (Fig. 2.16). From these curves, the efficiency of suspended solids removal at various depths and times may be estimated.

The percentage solids removal during flocculent settling for a given settling time can be estimated using Fig. 2.16 by drawing a horizontal line from the bottom of the settling basin to the top. The average removal percentages within each section of the basin are summed to estimate an overall removal percentage using Eq. (2.6).

$$\text{Percent removal} = \frac{\Delta h_1}{h_{\text{total}}} \cdot \frac{R_1 + R_2}{2} + \frac{\Delta h_2}{h_{\text{total}}} \cdot \frac{R_2 + R_3}{2} + \frac{\Delta h_3}{h_{\text{total}}} \cdot \frac{R_3 + R_4}{2} + \cdots \tag{2.6}$$

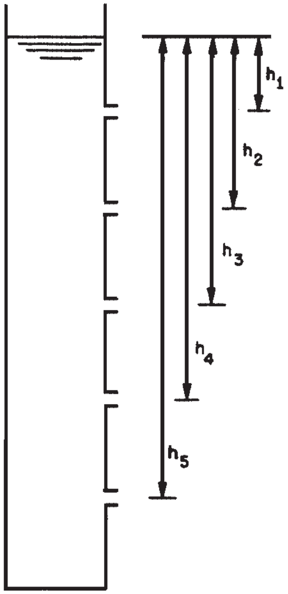


Figure 2.15 Laboratory column used in estimating fraction of flocculent particle removal.

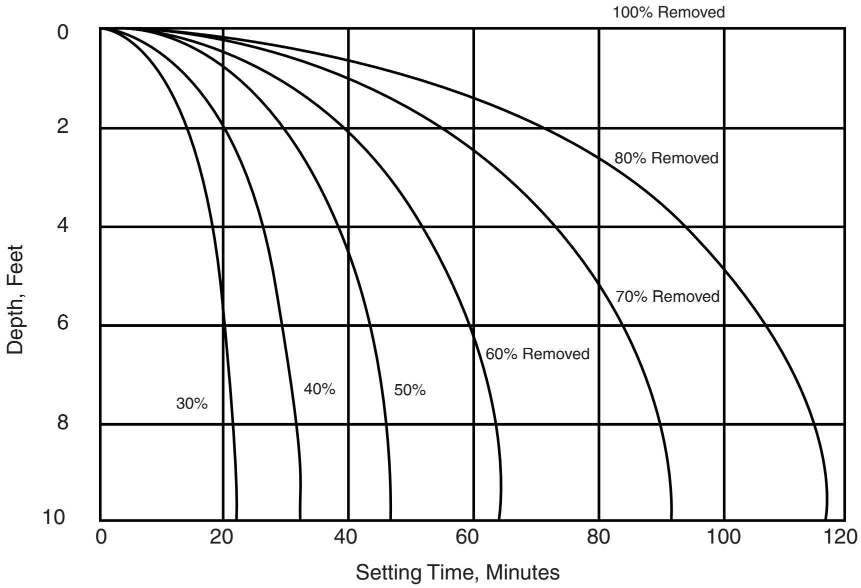


Figure 2.16 Isoconcentration curves used in flocculent settling analysis.

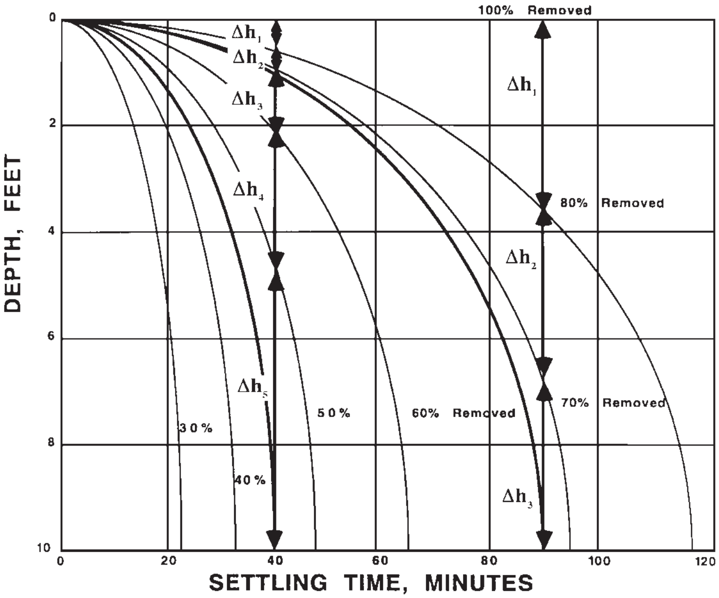
where Δh_i = distance of basin section i between two isoconcentration lines
h_{total} = total depth of basin
R_i = removal percentage (isoconcentration line)

Example 2.2 illustrates the use of Fig. 2.16 in estimating the percentage suspended solids removal during flocculent settling.

Example 2.2 Wastewater samples from the Warren County Wastewater Treatment Plant grit chamber effluent were evaluated in laboratory column tests to determine the optimal hydraulic retention time for suspended solids removal in the primary clarifier. Assuming that the clarifier has a depth of 10 ft, estimate the percentage solids removal at hydraulic retention times of 40 and 90 minutes. Assume that the isoconcentration curves in Fig. 2.16 were generated from laboratory column tests with Warren County wastewater samples.

solution

Step 1. Using Fig. 2.16, draw vertical lines at settling times of 40 and 90 minutes.



Step 2. At each settling time, estimate the depth between the various isoconcentration lines and the average solids removal percentage in these sections. This information is then substituted into Eq. (2.6) to estimate the total suspended solids removal. The distance of the various basin sections between isoconcentration lines for the 40-minute settling time is estimated as follows: $\Delta h_1 = 0.20$ ft, $\Delta h_2 = 0.16$ ft, $\Delta h_3 = 1.30$ ft, $\Delta h_4 = 2.20$ ft, and $\Delta h_5 = 6.14$ ft.

$$\text{Percent removal} = \frac{\Delta h_1}{h_\text{total}} \cdot \frac{R_1 + R_2}{2} + \frac{\Delta h_2}{h_\text{total}} \cdot \frac{R_2 + R_3}{2} + \frac{\Delta h_3}{h_\text{total}} \cdot \frac{R_3 + R_4}{2} + \frac{\Delta h_4}{h_\text{total}} \cdot \frac{R_4 + R_5}{2} + \frac{\Delta h_5}{h_\text{total}} \cdot \frac{R_5 + R_6}{2}$$

$$= \frac{0.20}{10.0} \cdot \frac{100 + 80}{2} + \frac{0.16}{10.0} \cdot \frac{80 + 70}{2} + \frac{1.3}{10.0} \cdot \frac{70 + 60}{2} + \frac{2.2}{10.0} \cdot \frac{60 + 50}{2} + \frac{6.14}{10.0} \cdot \frac{50 + 45}{2}$$

$$= 1.8 + 1.2 + 8.5 + 12.1 + 29.2 = 52.8 \text{ percent}$$

The distance of the various basin sections between isoconcentration lines for the 90-minute settling time is estimated as follows: $\Delta h_1 = 3.6$ ft, $\Delta h_2 = 4.0$ ft, and $\Delta h_3 = 2.4$ ft.

$$\text{Percent removal} = \frac{\Delta h_1}{h_\text{total}} \cdot \frac{R_1 + R_2}{2} + \frac{\Delta h_2}{h_\text{total}} \cdot \frac{R_2 + R_3}{2} + \frac{\Delta h_3}{h_\text{total}} \cdot \frac{R_3 + R_4}{2}$$

$$= \frac{3.6}{10.0} \cdot \frac{100 + 80}{2} + \frac{4.0}{10.0} \cdot \frac{80 + 70}{2} + \frac{2.4}{10.0} \cdot \frac{70 + 69}{2}$$

$$= 32.4 + 30.0 + 16.7 = 79.1 \text{ percent}$$

NOTE: There is a significant improvement in suspended solids removal when the hydraulic retention time is increased to 90 minutes from 40 minutes.

2.4.1.3 Hindered or zone settling. In sedimentation systems that receive wastewater containing high concentrations of suspended solids and/or in the lower depths of the clarifier, particles tend to settle as a zone or "blanket" in which they maintain the same relative position with respect to one another. This type of particle sedimentation is known as *hindered* or *zone settling*. The rate of hindered settling is a function of the concentration of solids and their settling characteristics [66]. The most common approach for design of sedimentation systems in which hindered settling is important is the limiting solids flux method.

Before applying the concept of limiting solids flux to clarifier design and operation, the design engineer must have a thorough understanding of the concept of solids flux (Fig. 2.17). The *solids flux* is the mass of solids transferred through a unit surface area of clarifier over a given time (typical units of flux are given in lb/ft^2·h or kg/m^2·h).

In a continuously operating clarifier, the total solids flux SF_T is a function of both gravitational forces and bulk fluid movement (i.e., advection). The solids flux due to gravity at any point in the clarifier may be estimated by using Eq. (2.7):

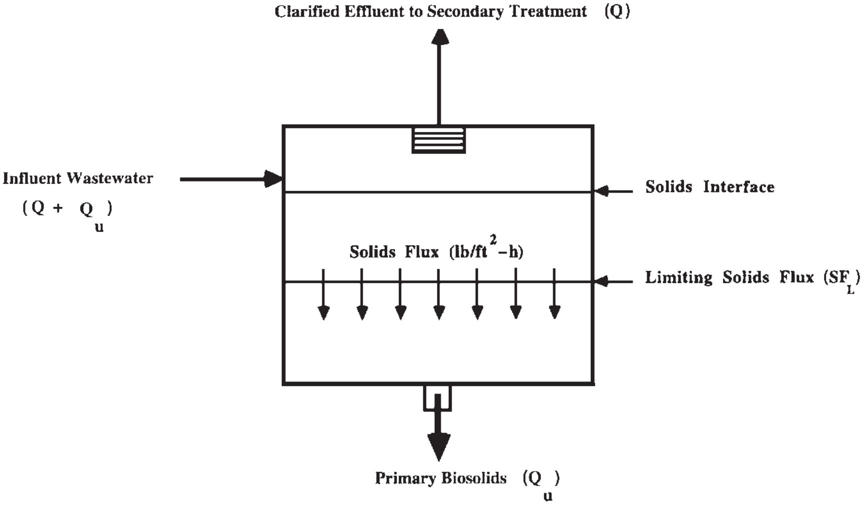


Figure 2.17 Conceptual illustration of the limiting solids flux SF_L.

$$SF_g = kC_iV_i \tag{2.7}$$

where SF_g = solids flux due to gravity, lb/ft² · h
C_i = concentration of solids at any point in the clarifier, mg/liter
V_i = settling velocity of solids at concentration C_i, ft/h
k = conversion constant, 1/16,030
MG = million gallons

$$k = \left(\frac{7.48\ \text{gal}}{\text{ft}^3}\right)\left(\frac{8.34\ \text{lb} \cdot \text{liter}}{\text{MG} \cdot \text{mg}}\right)\left(\frac{\text{MG}}{10^6\ \text{gal}}\right) = \frac{1}{16{,}030}$$

Figure 2.18 illustrates a typical gravity (i.e., batch) flux curve that characterizes the gravitational settling of solids within a clarifier. The procedure for developing the batch flux curve is illustrated in Fig. 2.19.

As illustrated in Fig. 2.19, the gravitational solids flux depends on both the concentration of suspended solids and the solids' settling characteristics (i.e., settling velocity V_i) at that concentration. At low solids concentrations (e.g., less than 0.01 percent solids), the flux of solids due to gravity is small because the settling velocity is independent of solids concentration. If the solids settling velocity remains constant and the solids concentration (i.e., C_i) increases, the solids flux (i.e., kC_iV_i) will increase. At high solids concentrations (i.e., above 1 percent), hindered settling occurs, and the solids settle in a zone or blanket.

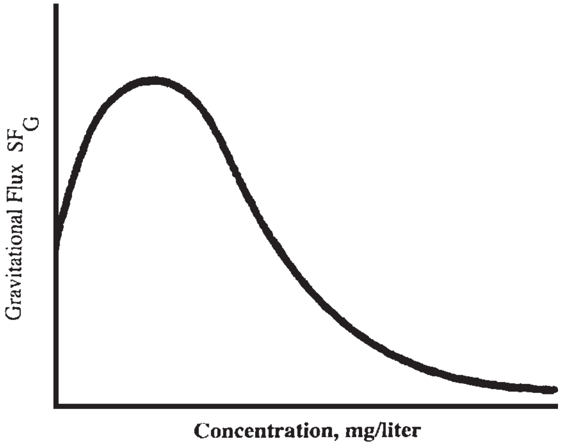


Figure 2.18 Sludge gravitational (batch) flux curve.

When the solids concentration reaches a level where hindered settling becomes predominant, the average solids settling velocity begins to decrease rapidly with any further increase in solids concentration. The overall impact of a declining solids settling velocity is a decrease in the solids flux. The increase and then decrease in the solids flux due to the changing solids concentration indicates that a maximum solids flux occurs within the sedimentation system. The maximum solids flux is observed during the development of the batch flux curve (see Fig. 2.19).

The solids flux due to bulk fluid transport, SF_U, is described by Eq. (2.8). The solids flux due to bulk transport (i.e., advection) at any point in the clarifier is a linear function of the solids concentration C_i and the underflow velocity U_B.

$$SF_U = kC_iU_B \tag{2.8}$$

where SF_U = solids flux due to underflow, lb/ft² · h
C_i = concentration of solids at any point in the clarifier, mg/liter
U_B = settling velocity due to bulk fluid movement ft/h
k = conversion constant, 1/16,030

$$k = \left(\frac{7.48\text{ gal}}{\text{ft}^3}\right)\left(\frac{8.34\text{ lb}\cdot\text{liter}}{\text{MG}\cdot\text{mg}}\right)\left(\frac{\text{MG}}{10^6\text{ gal}}\right) = \frac{1}{16{,}030}$$

By adding the gravitational flux SF_g and bulk fluid transport flux SF_U, an expression describing the total mass flux can be generated [Eq. (2.9)]:

$$\text{Total mass flux} = \text{gravity flux} + \text{bulk transport flux} \tag{2.9}$$

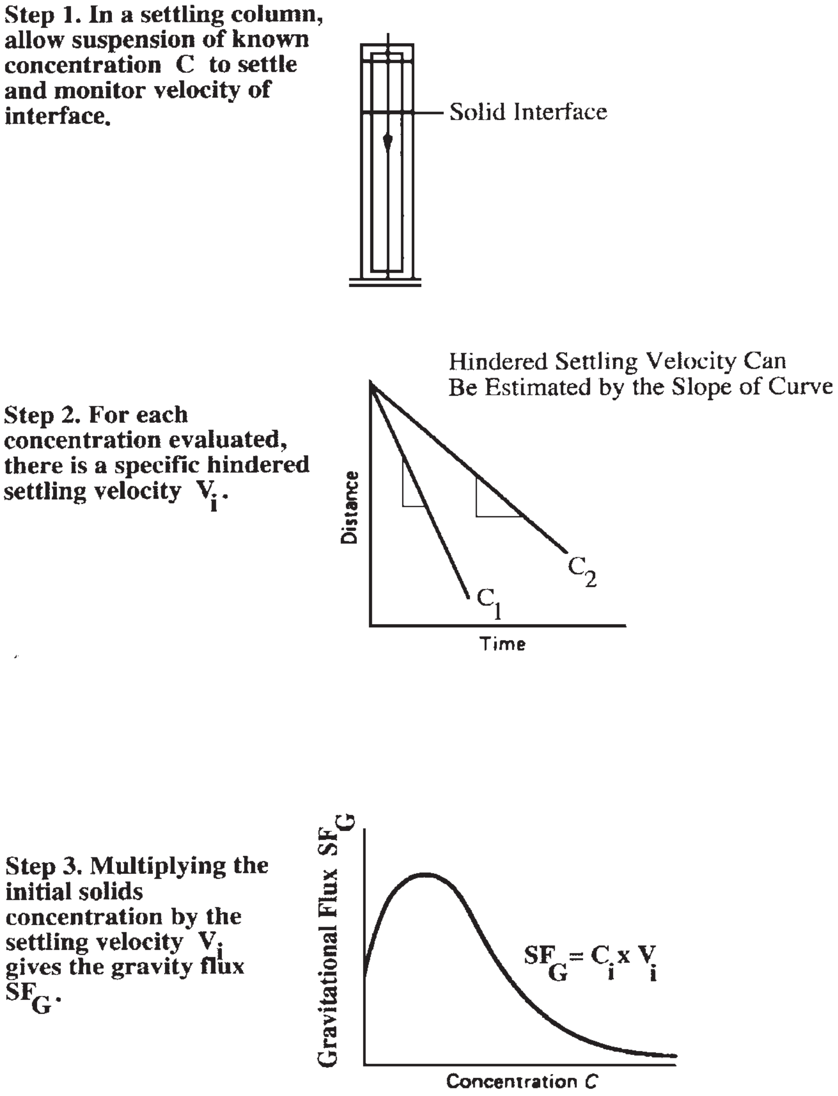


Figure 2.19 Method of developing a gravitational (e.g., batch) flux curve.

$$
\begin{aligned}
SF_T &= SF_g + SF_U \\
SF_T &= kC_iV_i + kC_iU_B
\end{aligned}
\qquad (2.9)
$$

where SF_T = total solids flux, lb/ft^2 · h
SF_g = solids flux due to gravity, lb/ft^2 · h
SF_U = solids flux due to underflow, lb/ft^2 · h

C_i = concentration of solids at any point in the clarifier, mg/liter

V_i = settling velocity of solids at concentration C_i, ft/h

U_B = settling velocity due to bulk fluid movement ft/h

k = conversion constant, 1/16,030

$$k = \left(\frac{7.48\text{ gal}}{\text{ft}^3}\right)\left(\frac{8.34\text{ lb}\cdot\text{liter}}{\text{MG}\cdot\text{mg}}\right)\left(\frac{\text{MG}}{10^6\text{ gal}}\right) = \frac{1}{16{,}030}$$

The cumulative effect of the gravitational and underflow flux on the total solids flux SF_T is depicted graphically in Fig. 2.20. It should be noted that, within the range of hindered zone settling, there is a minimum or limiting solids flux SF_L that develops within the clarifier.

To estimate the limiting solids flux graphically from the total flux curve, draw a horizontal line tangent to the low point of the total flux curve. Its intersection with the *y* axis represents the limiting solids flux SF_L for the clarifier (Fig. 2.21). The corresponding underflow sludge concentration in the clarifier can be obtained by dropping a vertical line at the intersection of the line of tangency to the limiting solids flux and the underflow velocity (i.e., U_B). This approach to evaluating the underflow concentration is permissible because, at the bottom of the clarifier, the bulk transport sludge flux is the predominant suspended solids removal mechanism (i.e., gravity settling is negligible at the bottom of the clarifier).

The limiting solids flux represents the maximum rate at which solids can be transmitted through the clarifier. At steady-state conditions, if the incoming solids loading (e.g, lb/ft²·h) were greater than the limiting solids flux, solids would begin to accumulate at the depth where the limiting solids flux occurs. Moreover, if sufficient solid storage capacity (i.e., tank depth) were unavailable to accommodate accumulated sludge, solids eventually would overflow the effluent weir and be discharged in the wastewater effluent.

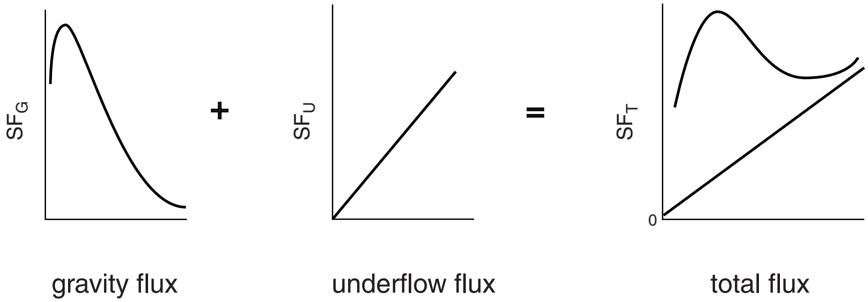


Figure 2.20 Total solids flux SF_T in a continuously operating clarifier.

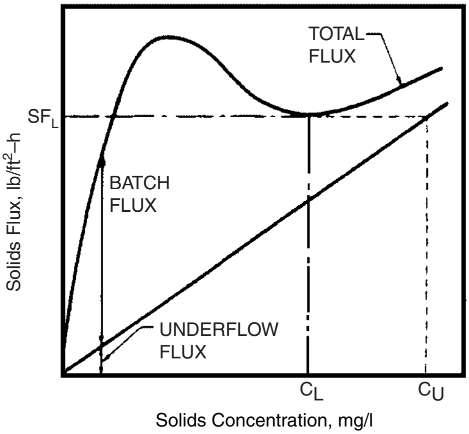


Figure 2.21 Obtaining the limiting solids flux SF_L from the total flux curve. (*Adapted by permission from ref. [66].*)

Figure 2.21 illustrates that the magnitude of both the total and limiting solids fluxes may be adjusted by controlling the underflow velocity U_B. In other words, by increasing or decreasing the underflow velocity U_B, the total and limiting solids fluxes will increase or decrease. Since the underflow velocity is equal to the underflow volumetric flow (or pumping) rate Q_u divided by the cross-sectional area of the clarifier (i.e., Q_u/area), the total and limiting solids fluxes can be varied by the wastewater treatment operator through a simple underflow pumping rate adjustment.

From Fig. 2.21, if a more concentrated underflow solids concentration were desired, the underflow velocity should be decreased. This operational change would, in effect, increase the solids settling (i.e., detention) time within the clarifier. An increase in settling time would have the net effect of increasing the solids content and reducing the limiting solids flux SF_L.

The minimum cross-sectional area required to achieve a desired underflow solids concentration may be estimated by performing a mass balance around the clarifier using the principle of limiting solids flux SF_L. This approach, which is described by Eq. (2.10), results in obtaining the desired underflow solids concentration while minimizing the potential for inadvertent loss of solids over the effluent weir.

$$\text{Incoming solids (lb/h)} = \text{maximum rate of solids transmitted through clarifier (lb/h)} \tag{2.10}$$

$$Q_0C_0 \times 8.34 = \text{area of clarifier (ft}^2\text{)} \times \text{limiting solids flux } SF_L \text{ (lb/ft}^2 \cdot \text{h)}$$

or

$$\text{Area of clarifier (ft}^2) = \frac{Q_0 C_0 \times 8.34}{SF_L} \tag{2.10}$$

where Q_0 = volumetric flow rate entering the clarifier, million gallons, MG, per hour
C_0 = concentration of solids entering the clarifier, mg/liter
SF_L = limiting solids flux, lb/ft$^2 \cdot$ h
8.34 = (lb · liter/MG · mg)

Example 2.3 illustrates the use of Eq. (2.10) in estimating the minimum required settling area for a primary clarifier.

Example 2.3 The Garvey County Wastewater Treatment Facility desires to operate each of its four primary clarifiers at an overflow rate of 800 gal/ft^2·day. If the volumetric flow rate to the plant were estimated to be approximately 4 million gallons per day and the suspended solids concentration entering the primary clarifier is 250 mg/liter, determine the following:

1. Area of each clarifier based on overflow rate
2. Underflow solids concentration for each clarifier
3. Minimum cross-sectional area of clarifier for solids transmission

Assume that the underflow volumetric flow rate is to be maintained at 2 percent of the overflow rate and that the following laboratory solids settling results have been obtained from laboratory column tests:

Solids concentration (mg/liter)	Initial settling velocity (ft/h)
100	24.00
200	24.00
500	20.00
1,000	18.00
2,500	12.00
5,000	10.00
7,500	2.00
10,000	0.80
20,000	0.20
30,000	0.10
40,000	0.05
50,000	0.03

solution

Step 1. Estimate the clarifier area based on the overflow rate. Assume that each clarifier receives one-quarter (25 percent) of the flow (i.e., 1 million gallons per day, MGD).

$$\text{Area of clarifier (ft}^2) = \frac{\text{flow rate (gal/day)}}{\text{overflow rate (gal/ft}^2 \cdot \text{day)}}$$

$$= \frac{1 \cdot 10^6 \text{ gal/day}}{800 \text{ gal/ft}^2 \cdot \text{day}} = 1250 \text{ ft}^2$$

Although the overflow rate can be used to determine the required area for clarification, the solids flux must be examined to determine whether the clarifier area will be sufficient to accommodate the solids loading rate.

Step 2. Using Eq. (2.7), estimate the gravity flux:

$$SF_g = kC_iV_i$$

For a $C_i = 100$ mg/liter, SF_g is calculated as follows:

$$SF_g = \frac{1}{16{,}030} \cdot 100 \text{ mg/liter} \cdot 24 \text{ ft/h}$$

$$= 0.15 \frac{\text{lb}}{\text{ft}^2 \cdot \text{h}}$$

The gravity flux at each solids concentration may be estimated using the same approach and the results presented in tabular form.

Solids concentration (mg/liter)	Initial settling velocity (ft/h)	Gravity flux (lb/ft^2·h)
100	24.00	0.15
200	24.00	0.30
500	20.00	0.62
1,000	18.00	1.12
2,500	12.00	1.87
5,000	10.00	3.12
7,500	2.00	0.94
10,000	0.80	0.50
20,000	0.20	0.25
30,000	0.10	0.19
40,000	0.05	0.12
50,000	0.03	0.09

Step 3. Estimate the underflow flux SF_U using Eq. (2.8). To estimate the underflow flux, the underflow velocity U_B may be estimated employing the fact that the underflow volumetric flow rate is 2 percent of the overflow rate.

$$\text{Overflow rate} \cdot 0.02 = \text{underflow rate}$$

$$800 \text{ gal/ft}^2 \cdot \text{h} \cdot 0.02 = 16 \text{ gal/ft}^2\text{h}$$

or

$$U_B = \frac{16 \text{ gal}}{\text{ft}^2 \cdot \text{h}} \cdot \frac{\text{ft}^3}{7.48 \text{ gal}} = \frac{2.1 \text{ ft}}{\text{h}}$$

With an estimate of the underflow velocity U_B, the flux due to bulk transport may be obtained using Eq. (2.8).

$$SF_U = kC_iU_B$$

For a $C_i = 10{,}000$ mg/liter, SF_U is calculated as follows:

$$SF_U = \frac{1}{16{,}030} \cdot 10{,}000 \text{ mg/liter} \cdot 2.1 \text{ ft/h}$$

$$= 1.31 \frac{\text{lb}}{\text{ft}^2 \cdot \text{h}}$$

The underflow flux at each solids concentration may be estimated using the same approach and the results added to the gravity flux to estimate the total flux SF_T of solids in the primary clarifier.

Solids concentration (mg/liter)	Underflow flux ($\text{lb/ft}^2\cdot\text{h}$)	Gravity flux ($\text{lb/ft}^2\cdot\text{h}$)	Total flux ($\text{lb/ft}^2\cdot\text{h}$)
100	0.01	0.15	0.16
200	0.03	0.30	0.33
500	0.07	0.62	0.69
1,000	0.13	1.12	1.25
2,500	0.33	1.87	2.20
5,000	0.66	3.12	3.78
7,500	0.98	0.94	1.92
10,000	1.31	0.50	1.81
20,000	2.62	0.25	2.87
30,000	3.93	0.19	4.12
40,000	5.24	0.12	5.36
50,000	6.55	0.09	6.64

Step 4. Plot total flux SF_T versus solids concentration C_i and locate the limiting solids flux SF_L. Also, estimate the underflow concentration C_u by the intersection of the total flux curve and the underflow velocity U_B (see the graph on the next page).

From the curve, a SF_L and C_u of 1.8 $\text{lb/ft}^2\cdot\text{h}$ and 15,000 mg/liter were determined, respectively, for the given operational conditions.

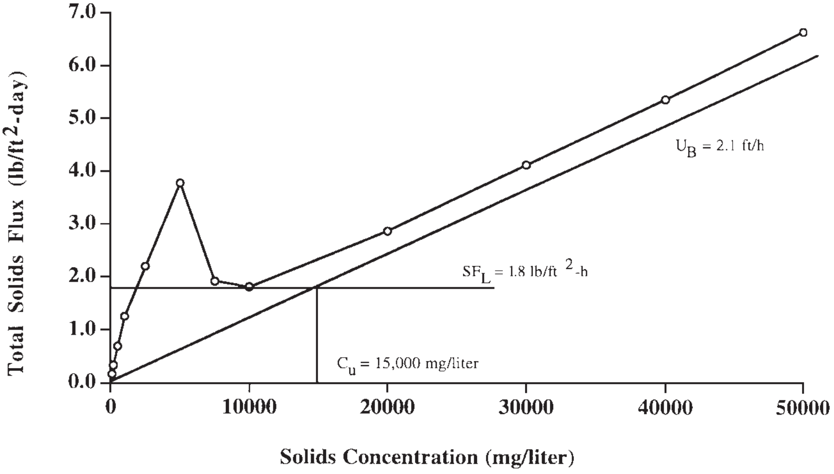
Step 5. Find the minimum required clarifier area to ensure that the solids loading rate will not result in an overloaded clarifier using Eq. (2.10).

$$\text{Area of clarifier (ft}^2\text{)} = \frac{Q_0 \cdot C_0 \cdot 8.34}{SF_L}$$

$$= \frac{1 \text{ MGD} \cdot 250 \text{ mg/liter} \cdot 8.34}{\dfrac{1.8 \text{ lb}}{\text{ft}^2 \cdot \text{h}} \cdot \dfrac{24 \text{ h}}{\text{day}}}$$

$$= 48.3 \text{ ft}^2$$

Since the required minimum area needed for ensuring proper solids transmission is significantly *less* than the clarifier area estimated using the overflow rate (i.e., 1250 ft^2), no accumulation of solids will occur in the system (i.e., the clarifier will not be overloaded with solids).



NOTE: To increase the solids content of the underflow solids concentration C_u, the operator should *decrease* the underflow velocity U_B by reducing the underflow pump rate Q_B.

An alternative graphic approach to using the total flux curve for estimating the limiting solids flux is the state point method [28,66]. Like the total flux approach, the state point method may be employed to estimate the underflow volumetric flow rate Q_u required to obtain a desired underflow sludge concentration C_u. One of the advantages of employing the state point method for estimating the minimum clarifier size is that only the batch (i.e., gravity) flux data are required.

In the state point method, the limiting solids flux SF_L is obtained by drawing a line tangent to the batch flux curve that passes through the desired underflow concentration C_u and intersects the y axis (Fig. 2.22). The point of tangency is called the *state point,* and the point of intersection with the y axis is the limiting solids flux SF_L. Moreover, by geometric comparison with the total flux curve, it can be shown that the slope of the tangent line is the negative value of the required underflow velocity (i.e., $-U_B$). The limiting solids flux can be used to estimate the minimum required area of the clarifier from which the underflow pumping rate Q_u can be estimated [Eq. (2.10)].

If an alternative underflow solids concentration C_u is desired, another line tangent to the batch flux curve should be drawn from the target underflow solids concentration to the y axis. Since the cross-sectional area is fixed, estimation of the new underflow solids concentration will require maintaining a different limiting solids flux SF_L and underflow pumping rate Q_u. Figure 2.23 illustrates the impact of changing the underflow velocity U_B on both the underflow

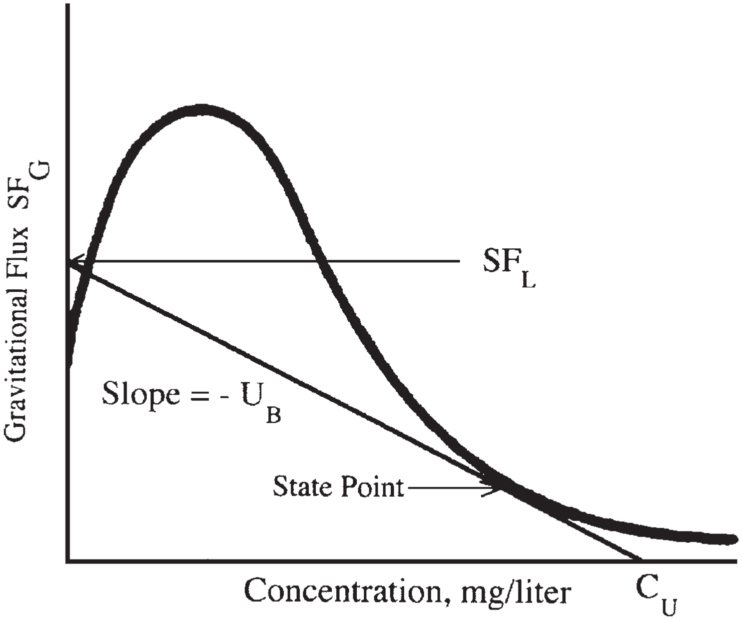


Figure 2.22 State point method for estimating primary clarifier area.

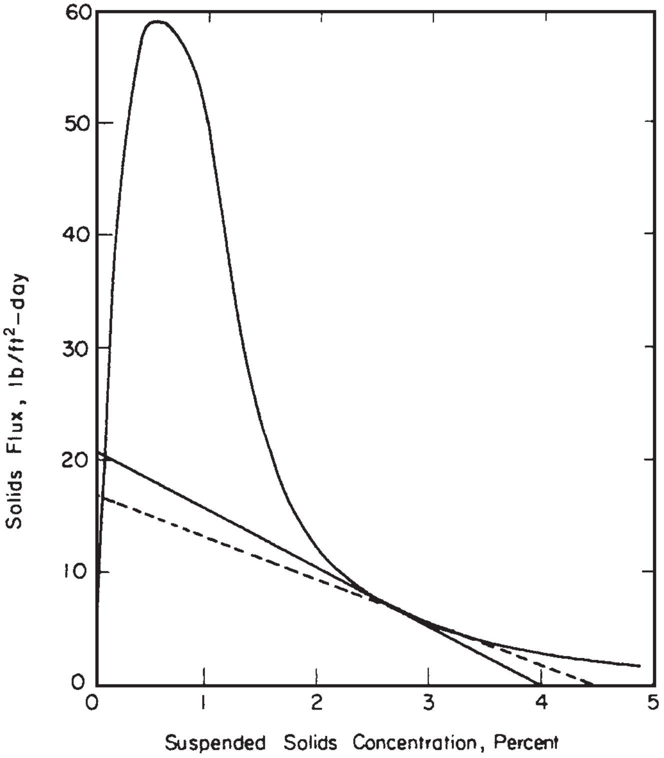


Figure 2.23 Effect of changing bulk velocity on underflow concentration C_u.

concentration C_u and the limiting solids flux SF_L. Example 2.4 illustrates the application of the point method for controlling clarifier performance.

Example 2.4 The Turner County Water Reclamation Facility has decided to install a new primary clarifier to improve its solids management program. If the maximum flow rate to the clarifier is estimated to be 800,000 gal/day and the suspended solids concentration is approximately 220 mg/liter, determine the following:

1. The underflow rate U_B to achieve a underflow concentration of 4 percent.
2. The minimum clarifier area to maintain the 4 percent underflow concentration.
3. The underflow pumping rate Q_u required to maintain the 4 percent underflow concentration.

Assume that the following solids settling data have been obtained from laboratory tests.

Solids concentration (mg/liter)	Initial settling velocity (ft/h)
100	24.00
200	24.00
500	20.00
1,000	18.00
2,500	12.00
5,000	10.00
7,500	2.00
10,000	0.80
20,000	0.20
30,000	0.10
40,000	0.05
50,000	0.03

solution

Step 1. Using Eq. (2.7), estimate the gravity flux:

$$SF_g = kC_iV_i$$

For a $C_i = 100$ mg/liter, SF_g is calculated as follows:

$$SF_g = \frac{1}{16{,}030} \cdot 100 \text{ mg/liter} \cdot 24 \text{ ft/h}$$

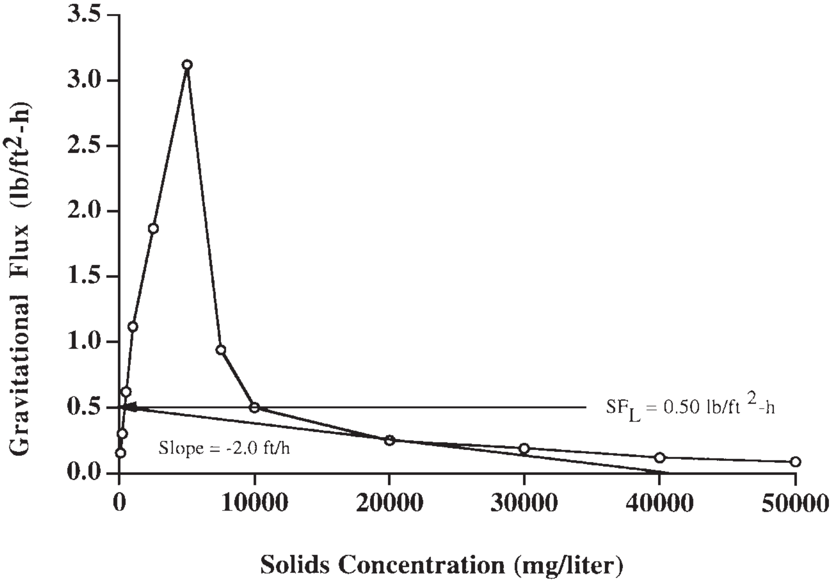
$$= \frac{0.15 \text{ lb}}{\text{ft}^2 \cdot \text{h}}$$

The gravity flux at each solids concentration may be estimated using the same approach and the results presented in tabular form.

Solids concentration (mg/liter)	Initial settling velocity (ft/h)	Gravity flux (lb/ft^2·h)
100	24.00	0.15
200	24.00	0.30
500	20.00	0.62
1,000	18.00	1.12
2,500	12.00	1.87
5,000	10.00	3.12
7,500	2.00	0.94
10,000	0.80	0.50
20,000	0.20	0.25
30,000	0.10	0.19
40,000	0.05	0.12
50,000	0.03	0.09

Step 2. Plot gravitational flux SF_g versus solids concentration. Draw a tangent line to the batch flux curve starting from an underflow concentration C_u of 4 percent (i.e., 40,000 mg/liter). Where the tangent line intersects the y axis is the limiting solids flux SF_L. From the following figure, the limiting solids flux is estimated to be 0.50 lb/ft^2·h.

Step 3. Estimate the underflow velocity U_B by taking the negative value of the slope. In this case, the tangent slope is -2.0 ft/h. Therefore, to achieve an underflow concentration of 4 percent, the underflow velocity must be maintained at 2.0 ft/h.



Step 4. Find the minimum required primary clarifier area to maintain the 4 percent underflow concentration using Eq. (2.10).

$$\text{Area of clarifier (ft}^2) = \frac{Q_0 \cdot C_0 \cdot 8.34}{SF_L}$$

$$= \frac{0.8 \text{ MGD} \cdot 220 \text{ mg/liter} \cdot 8.34}{\dfrac{0.5 \text{ lb}}{\text{ft}^2 \cdot \text{h}} \cdot \dfrac{24 \text{ h}}{\text{day}}} = 122.3 \text{ ft}^2$$

Step 5. Find the pumping rate Q_u necessary to maintain a 4 percent solids content in the underflow by multiplying the minimum required clarifier area times the underflow velocity U_B.

$$Q_u \text{ (gal/min)} = \text{area} \cdot U_B$$

$$= 122.3 \text{ ft}^2 \cdot \frac{2.0 \text{ ft}}{\text{h}} \cdot \frac{7.48 \text{ gal}}{\text{ft}^3} \cdot \frac{\text{h}}{60 \text{ min}}$$

$$= \frac{31 \text{ gal}}{\text{min}}$$

Together with the estimated area and underflow pumping rate, drive torque requirements, total tank depth, and maximum hydraulic loading must be evaluated to complete the clarifier design. Typical ranges for these design parameters may be found in the following references [22,29].

2.4.1.4 Compression settling.

The volume required for solids accumulating in the compression-settling region also can be determined by column tests. The rate of consolidation in this region has been found to be proportional to the difference in the depth at some time and the depth to which the solids will settle 24 hours later. This behavior has been described empirically by Eq. (2.11).

$$H_t - H_{24} = (H_2 - H_{24})\, e^{-i\,(t\,-\,t_2)} \tag{2.11}$$

where H_t = sludge height at time t
H_{24} = sludge height after 24 hours of settling
H_2 = sludge height at time t_2
i = constant for a given suspension

It has been reported that stirring compacts the solids in the compression region by breaking up the floc and permitting water to escape. Rakes often are used in clarifiers to break up solid aggregates, producing better compaction [22,29].

2.4.2 Primary clarification tank design

Primary clarification normally is viewed as a preliminary treatment step to secondary wastewater treatment, although in a limited number of cases primary clarification also may provide the principal degree of wastewater treatment [28,59,60]. Properly operating primary clarifiers should remove approximately 50 to 70 percent of the suspended solids together with 25 to 40 percent of the BOD_5 from the influent municipal wastewater flow.

In general, primary clarifiers are designed to provide from 0.5 to 2.5 hours of hydraulic retention time based on the average wastewater flow rate [29]. In addition to hydraulic retention time, the surface-loading (or overflow) rate expressed in units of gallons per square foot per day (or $m^3/m^2 \cdot$day), is used in design calculations. The choice of a suitable surface-loading rate will depend on the characteristics of the suspended solids to be removed. Typical values of primary clarifier design parameters are provided in Table 2.18.

The effect of the overflow rate and hydraulic retention time on suspended solids removal varies widely depending on the characteristics of the suspended solids. However, in all circumstances, the overflow rates must be established at levels that are adequate to ensure satisfactory performance of the clarifier at peak flow rates [62]. Once the overflow rate is established, the area of the clarifier can be computed using Eq.

TABLE 2.18 Primary Clarifier Design Data*

Parameter	Value: Range	Value: Typical
Primary Clarification Followed by Secondary Treatment		
Hydraulic retention time, hours	1.5–2.5	2.0
Surface-loading rate, $gal/ft^2 \cdot day$		
Average flow	800–1200	
Peak hourly flow	2000–3000	2,500
Weir loading rate, gal/ft·day	10,000–40,000	20,000
Primary Clarification with Waste-Activated Sludge Return		
Hydraulic retention time, hours	1.5–2.5	2.0
Surface-loading rate, $gal/ft^2 \cdot day$		
Average flow	600–800	
Peak hourly flow	1200–1700	1,500
Weir loading rate, gal/ft·day	10,000–40,000	20,000

Note: $gal/ft^2 \cdot day \cdot 0.0407 = m^3/m^2 \cdot day$.

*Adapted from refs. [28,29,62].

(2.3). Similarly, once the area and hydraulic retention time are determined, the depth of the tank can be estimated using standard geometry. It should be noted that weir loading rates have little effect on the effectiveness of clarifiers to remove suspended solids. The placement of the weirs and the design of the tanks have more of an impact on clarifier performance. Readers interested in weir design and placement on primary clarifiers are referred to the following references [22,62].

2.4.2.1 Scour velocity. *Scour velocity* refers to the fluid velocities within the settling zone of the clarifier that are sufficient to resuspend settled solids. Scour velocity is a critical design parameter in primary clarifiers. Velocities in the clarifier should be kept as low as possible so that settled solids are not resuspended, resulting in poor effluent quality. The critical horizontal velocity at which scour will begin may be estimated using Eq. (2.12).

$$V_H = \left[\frac{8k\,(s-1)\,gd}{f} \right]^{1/2} \tag{2.12}$$

where V_H = horizontal velocity that will just produce scour
k = constant (function of settled solids—0.06 for flocculent material, 0.04 for sand)
s = specific gravity of settled particles
g = acceleration due to gravity
d = diameter of particles
f = Darcy-Weisbach friction factor

By maintaining the horizontal velocities in the clarifier to values significantly less than those estimated by Eq. (2.12), resuspension of settled particles is minimized.

2.4.2.2 Primary clarifier tank dimensions and operation. Most primary clarification tanks are either rectangular or circular is shape and are cleaned mechanically [62]. The selection of a particular design depends on the given application, local site conditions, and the experience and judgment of the designer [22,62]. Typical data on both rectangular and circular primary clarification tanks are provided in Table 2.19.

Rectangular tanks normally will use chain-and-flight or traveling-bridge collectors to remove settled solids. In the chain-and-flight collectors, settled solids are scraped continuously into sludge hoppers or troughs by the mechanical system (Fig. 2.24). In the bridge-type collector, a single longitudinal collector scrapes sludge into a central hopper. The scrapper travels up and down the tank on rubber wheels or on rails supported by the side walls. Solids are pumped out of the bottom of the clarifier into collection troughs (Fig. 2.25).

TABLE 2.19 Design Parameters for Circular and Rectangular Primary Clarification Tanks*

Tank type	Range Range	Range Typical
Circular		
Depth, ft	10–15	12
Diameter, ft	10–200	40–150
Bottom slope, in/ft	0.75–2.0	1.0
Flight travel speed, rev/min	0.02–0.05	0.03
Rectangular		
Depth, ft	10–15	12
Length, ft	50–300	80–130
Width, ft	10–80	16–32
Flight travel speed, ft/min	2–4	3

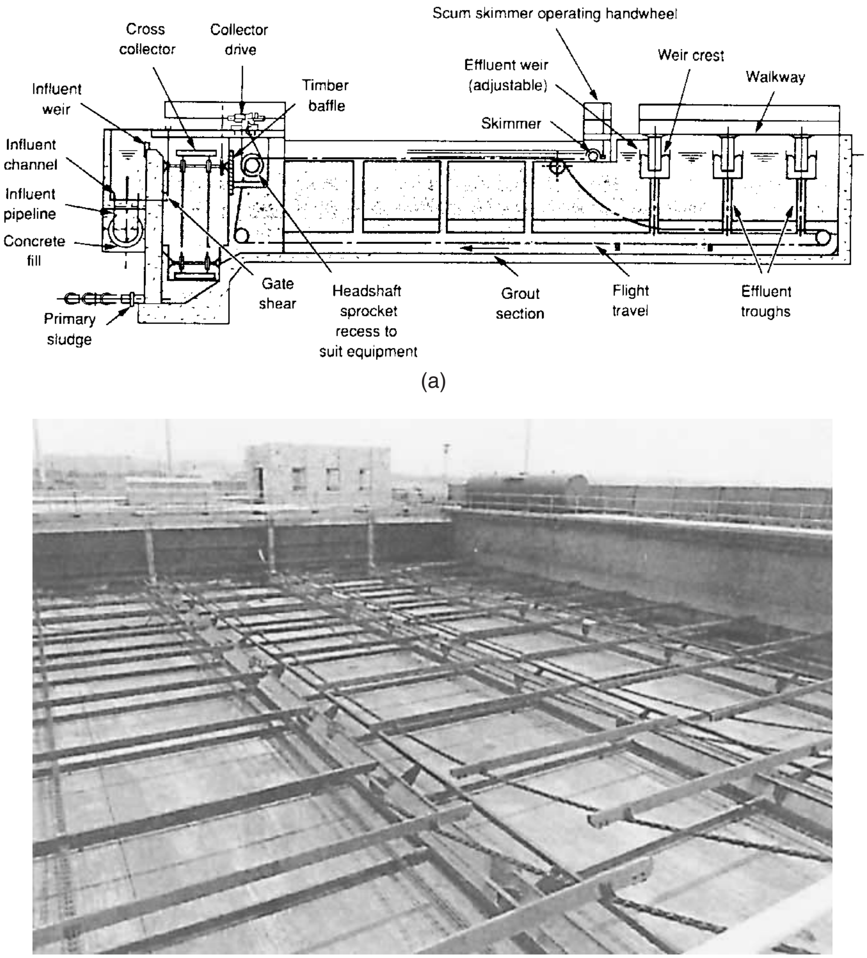
*Adapted from refs. [29,62].

Because of the potential resuspension of settled solids, the design of the inlet flow distribution system is critical in rectangular clarification tanks. Options for inlet design in rectangular clarification tanks include (1) inlet channels with inlet weirs, (2) inlet channels with submerged ports or orifices, and (3) inlet channels with wide gates and slotted baffles [22]. In all cases, inlets must be designed to minimize both the vertical and horizontal flow velocities.

Rectangular clarification tanks have several advantages over circular clarification tanks, including the fact that multiple rectangular tanks take up less land area than multiple circular tanks. Moreover, rectangular tanks lend themselves to common-wall construction that reduces overall construction costs [22,62]. Finally, rectangular tanks are preferred when clarification tank roofs or covers are required. Readers interested in additional design details on rectangular clarification tanks are referred to the following references [22,62].

In circular tanks, the flow of influent wastewater is radial. To achieve a radial flow pattern, the wastewater may be introduced in the center of the tank or around the periphery (Fig. 2.26).

Multiple circular tanks normally are arranged in groups of two or four. The flow is divided among tanks by a flow-split structure normally located between the tanks. In the center-feed design, the wastewater is carried to the center of the tank in a pipe suspended from the bridge or encased in concrete beneath the tank floor [28,60]. At the center of the tank, the wastewater enters a circular well designed to distribute the flow equally in all directions. The center well has a diameter that is normally between 15 and 20 percent of the total tank diameter and has a depth from 3 to 8 ft (see Fig. 2.26*a*).



(a)

(b)

Figure 2.24 (*a*) Schematic diagram of a chain-and-flight rectangular clarifier tank. (*b*) Photograph of a chain-and-flight sludge-removal system.

The settled solids are removed to a central hopper or draw-off by scrappers or rakes that revolve slowly around the tank bottom.

In the peripheral-feed design, the influent wastewater is discharged into an annular space formed between a suspended circular baffle and the tank wall (see Fig. 2.26*b*). The wastewater flows around the tank and underneath the baffle. The clarified effluent is discharged over weirs located on both sides of the weir trough.

The principal advantages of circular tanks over the rectangular tank design are that they cost less to build for greater tank depths and cost

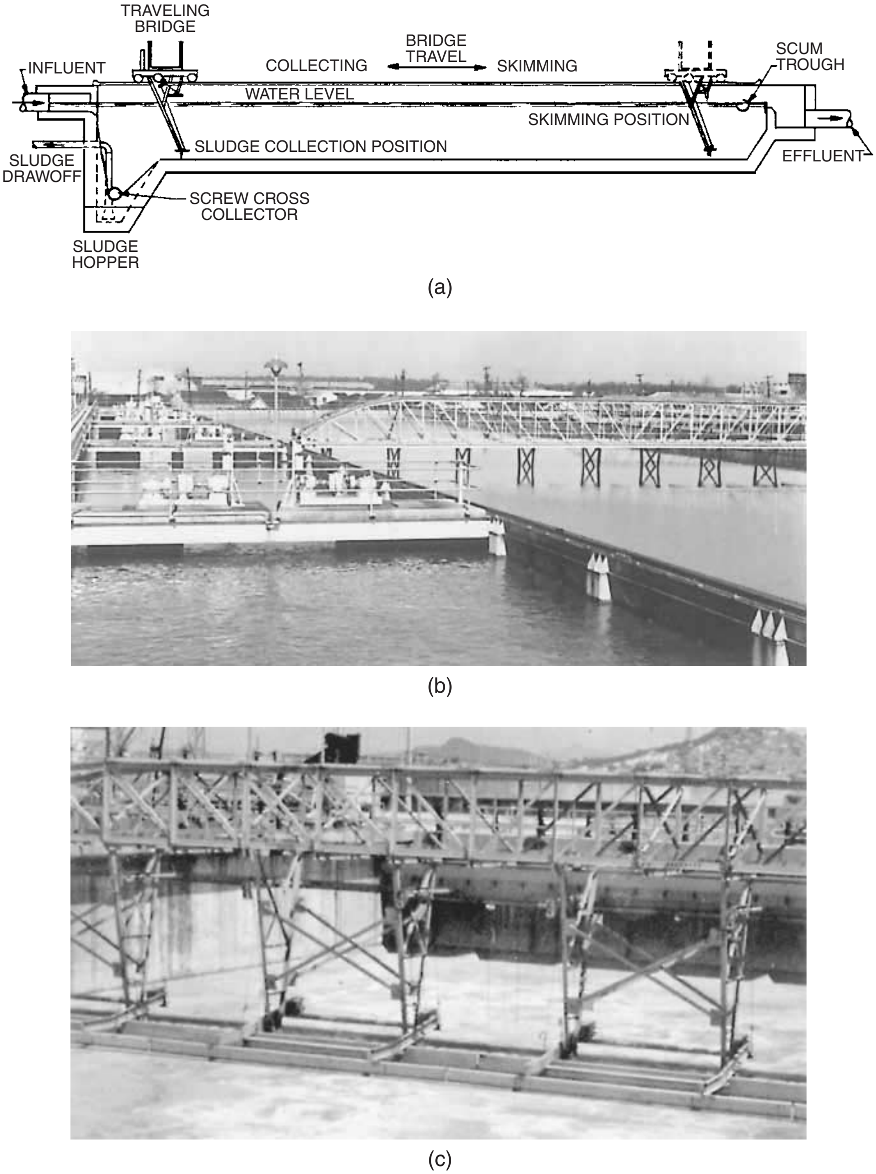


Figure 2.25 (*a*) Schematic diagram of a rectangular clarifier tank traveling-bridge sludge-collection system. (*b*) Photograph of a traveling-bridge sludge-collection system. (*Courtesy of Dorr-Oliver.*) (*c*) Photograph of an empty traveling-bridge sludge-collection system. (*Courtesy of Dorr-Oliver.*)

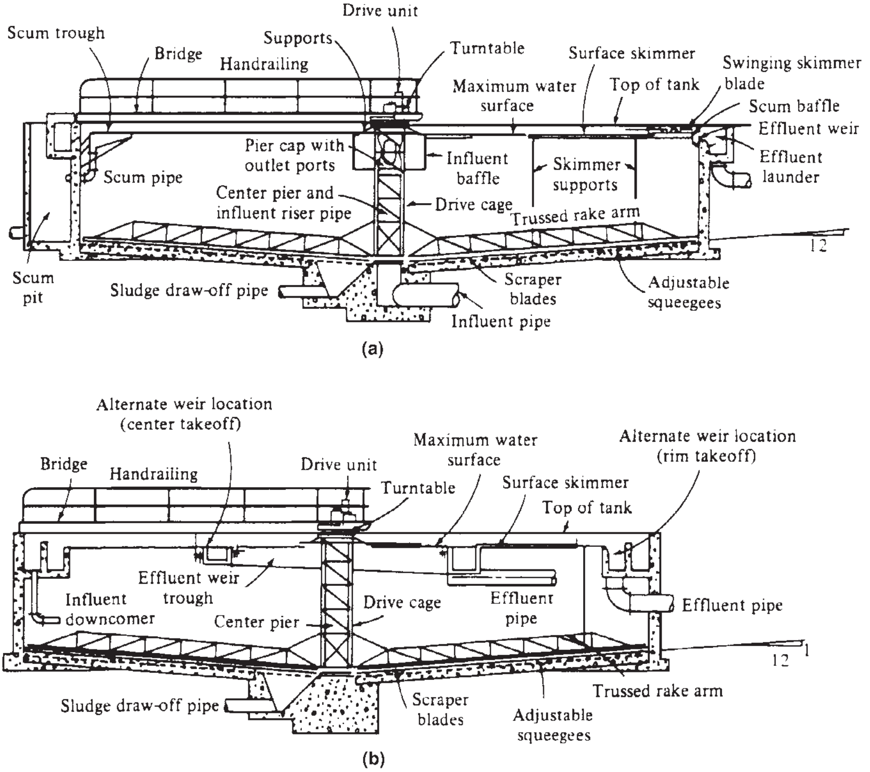


Figure 2.26 (*a*) Schematic diagram of a center-feed circular clarification tank. (*b*) Schematic diagram of a peripheral-feed circular clarification tank.

less to maintain [29]. For example, the rotary sludge-removal systems in circular tanks have significantly lower operation and maintenance costs than the chain-and-flight sludge collectors used in rectangular tanks [62].

2.4.2.3 Inclined-plate clarifiers. In addition to the traditional circular or rectangular clarification tank design, some primary clarifiers are equipped with modules consisting of small-diameter tubes or closely position plates inclined at an angle. The principle underlying the use of inclined-plate or tube settlers is the fact that the efficiency of particle removal was found to be independent of tank depth when discrete-particle settling behavior is predominant [Eq. (2.3)]. By minimizing the distance over which particles must travel to settle through the use of closely packed tubes or plates, the fraction of particles removed increases. Plate and tube settlers have developed as an alternative to shallow sedimentation basins and, in many cases, are used in conjunction with traditional primary clarifiers (Fig. 2.27).

In the inclined settler system, the flow within the basin passes upward from the bottom of the plate or tube modules and exits from above. The solids that settle out onto the plates or within the tubes move downward in a countercurrent direction to the wastewater flow.

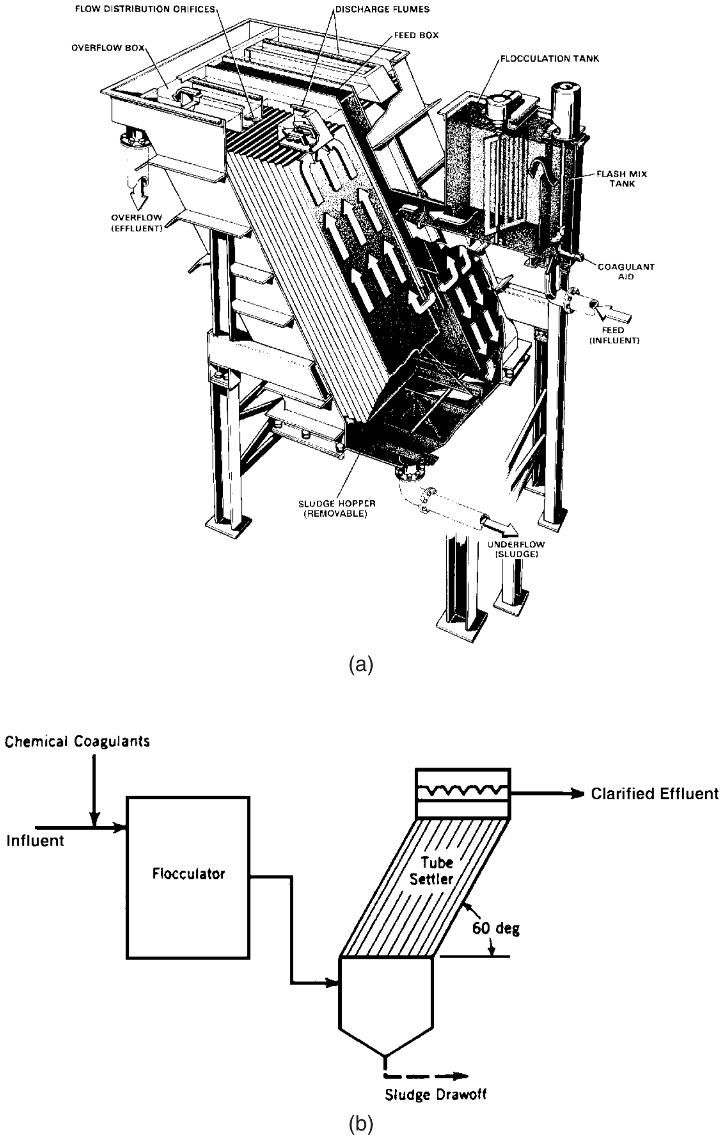


Figure 2.27 (*a*) Schematic diagram of an inclined-plate clarifier. (*Courtesy of Parkson Corporation.*) (*b*) Schematic diagram of flocculation treatment followed by a tube-settler clarifier. (*Adapted by permission from ref. [66].*)

To be self-cleansing, plate and tube settlers normally are set at an angle of between 45 and 60 degrees from the horizontal plane. If the plates or tubes are inclined at angles less than 45 degrees, solids will tend to accumulate within the modules, whereas if the angle is increased above 60 degrees, the efficiency of solids removal decreases. Care must be taken in establishing the inclination angle because one of the major concerns in the use of these systems is the accumulation of solids, which can lead anaerobic conditions and odor complaints [66]. The shape, angle of inclination, and plate or tube length will vary in plate or tube settlers according to the particular application. Readers interested in further details on inclined plate or tube clarifiers are referred to the following references [22,66].

2.4.3 Chemical precipitation

Properly designed and operated primary clarifiers will remove approximately 50 to 70 percent of the suspended solids without chemical treatment. If chemical treatment is used in conjunction with primary sedimentation, 80 to 90 percent of the total suspended solids can be removed [28,66]. The most common chemicals used in chemical precipitation are listed in Table 2.20.

In addition to the chemicals listed in Table 2.20, ferrous sulfate occasionally is used as well as organic polymers (e.g., polyelectrolytes). Polymers or polyelectrolytes are high-molecular-weight organic compounds that contain ionic or other functional groups [66]. Polymers are classified as anionic, cationic, or nonionic based on the overall electric

TABLE 2.20 Chemicals Used in Primary Treatment*

Ferric chloride†

$$\overset{162.1}{FeCl_3} + 3H_2O \rightarrow \overset{106.9}{Fe(OH)_2(s)} + 3H^+ + 3Cl^-$$

Lime

$$\overset{56 \text{ as CaO}}{Ca(OH)_2} + Ca(HCO_3)_2 \rightarrow \overset{2 \times 100}{2CaCO_3(s)} + 2H_2O$$

Ferric chloride and lime

$$\overset{2 \times 162}{2FeCl_3} + \overset{3 \times 56 \text{ as CaO}}{3Ca(OH)_2} \rightarrow 3CaCl_2 + \overset{2 \times 106.9}{2Fe(OH)_3(s)}$$

Alum

$$\overset{666.7}{Al_2(SO_4)_3 \cdot 18H_2O} + \overset{3 \times 100 \text{ as } CaCO_3}{3Ca(HCO_3)_2} \rightarrow 3CaSO_4 + \overset{2 \times 78}{2Al(OH)_3(s)} + 6CO_2 + 18H_2O$$

*Adapted from ref. [28].

†Numbers above chemical species are the molecular weights.

charge. Because of the large number of monomers available commercially and the tailoring of the polymer chemical behavior through varying the charge density and/or the number or type of ionizable groups, a large number of polymers currently are being marketed for wastewater chemical treatment. The principal advantage of using polymers rather that inorganic chemicals is more effective particle removal with lower chemical dose. However, polymers may be considerably more expensive that inorganic chemicals [60].

Regardless of the type of compound used, most precipitation chemicals are added to the raw wastewater in a flash-mixing tank followed by a coagulation/flocculation tank. It is in the coagulation/flocculation tank that the chemical/particle floc grows to an appropriate size that can be removed effectively by gravity. Figure 2.28 presents both a schematic diagram and a photograph of a primary clarifier system designed to achieve both flocculation and sedimentation.

2.4.3.1 Mixing. Mixing operations at the wastewater treatment plant can be classified as *continuous rapid* (i.e., 30 seconds or less) or *continuous.* The principal objective of continuous rapid mixing is to completely mix or blend one substance with another. This type of mixing is characteristic of the mixing of precipitation chemicals with influent wastewater prior to transfer to a coagulation/flocculation tank. The principal objective of continuous mixing is to maintain the contents of a tank in a completely mixed state. This type of mixing is characteristic of a flocculation tank in which particle aggregation is desired.

All approaches to evaluating mixing include estimation of a velocity gradient (i.e., shear intensity) and the hydraulic retention time. Fundamentally, the principal difference between continuous rapid and continuous mixing is the intensity of the velocity gradient and the hydraulic retention time. Typical velocity gradients and hydraulic detention times used in wastewater processes are summarized in Table 2.21.

Continuous rapid mixing can be accomplished by several methods, including (1) mechanical mixers, (2) static mixers, (3) pumping, (4) pipeline turbulence, (5) Venturi flumes, and (6) hydraulic jumps [60]. Some typical rapid mixing systems are shown in Fig. 2.29.

Continuous mixing normally is accomplished by the following systems: (1) mechanical mixing, (2) static mixers, (3) pumping, and (4) pneumatic systems [28].

In all mixing systems, turbulence is responsible for either the blending of substances or the maintaining of solids in suspension. In mechanical mixing systems, turbulence is induced through the input

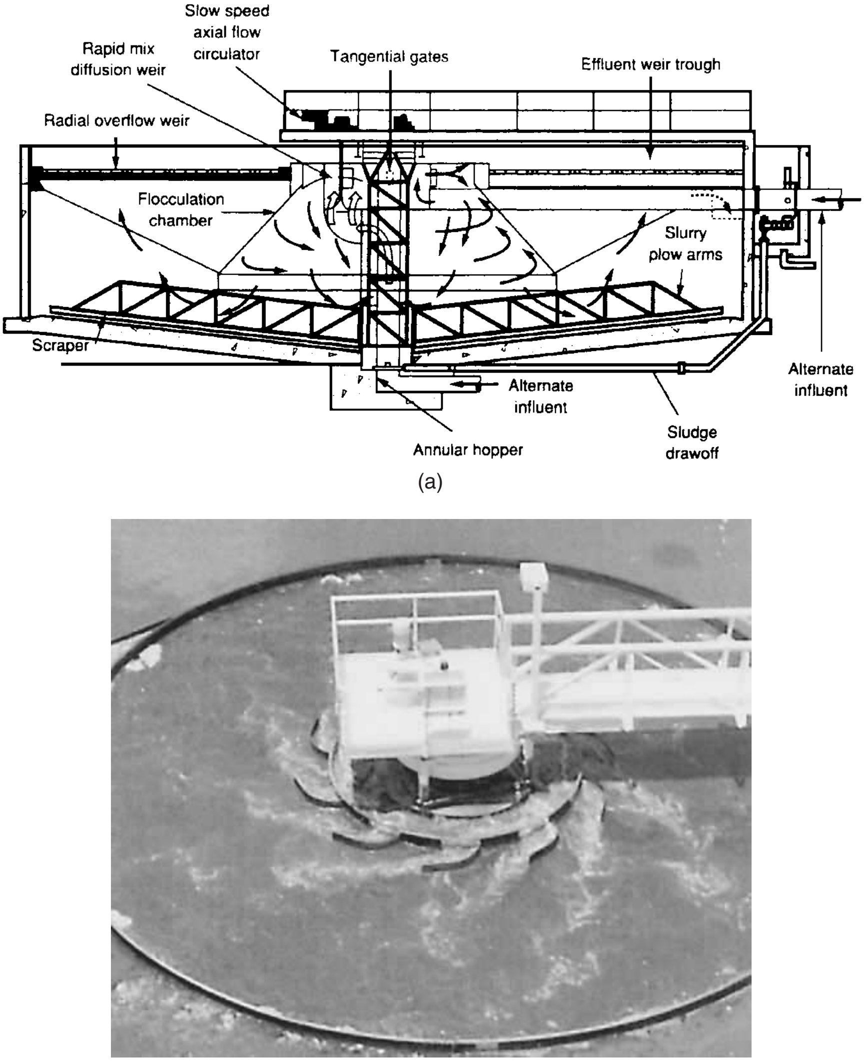


Figure 2.28 (*a*) Schematic diagram of primary sedimentation of wastewater designed for induced flocculation. (*b*) Primary clarifier equipped with an energy-dissipating inlet for enhanced flocculation. (*Courtesy of WesTech Engineering, Inc.*)

of energy by means of rotating impellers, turbines, or paddles. In most cases, the power input per unit volume of liquid can be used as an approximate measure of mixing effectiveness [22]. Equations (2.13) and (2.14) may be used in the general design and operation of mechanical mixing systems:

TABLE 2.21 Velocity Gradients (*G*) and Hydraulic Retention Times (*HRT*) Used in Rapid Mixing and Continuous Mixing Systems*

	Range of values	
	HRT	G (s^{-1})†
Rapid mixing systems (e.g., chemical mixing tanks)	5–20 s	250–1500
Continuous mixing (e.g., flocculation tanks)	10–30 min	20–80

*Adapted from refs. [28,66].
†Velocity gradient.

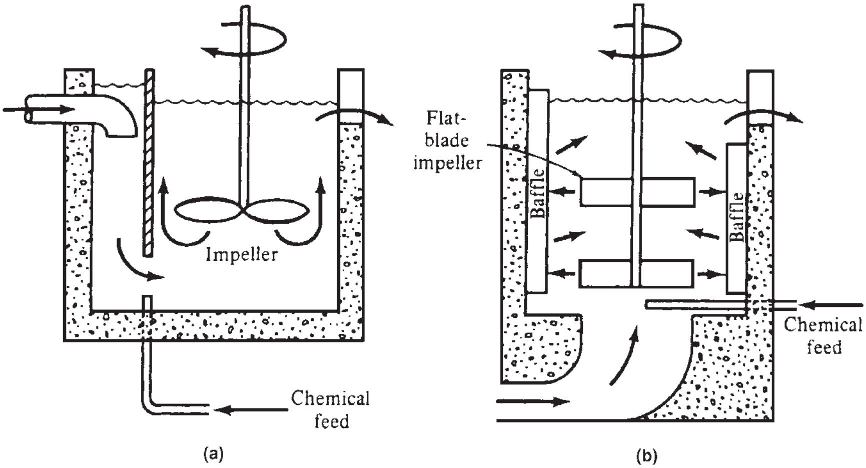


Figure 2.29 (*a*) Schematic diagram of a rapid impeller mixer. (*b*) Schematic diagram of a rapid paddle mixer.

$$G = \left(\frac{P}{\mu V}\right)^{1/2} \tag{2.13}$$

where G = mean velocity gradient, s^{-1}
P = power requirement, ft · lb/s (W)
μ = dynamic viscosity, lb · s/ft^2 (N · s/m^2)
V = tank volume, ft^3 (m^3)

$$Gt_d = \frac{V}{Q}\left(\frac{P}{\mu V}\right)^{1/2}$$

$$= \frac{1}{Q}\left(\frac{PV}{\mu}\right)^{1/2} \tag{2.14}$$

where G = mean velocity gradient, s^{-1}
t_d = hydraulic retention time, s

Q = flow rate, ft^3/s (m^3/s)
P = power requirement, ft · lb/s (W)
μ = dynamic viscosity, lb · s/ft^2 (N · s/m^2)
V = tank volume, ft^3 (m^3)

For propeller and turbine mixers, the empirical relationships given by Eqs. (2.15) and (2.16) have been used successfully in estimating the power requirements under turbulent or quiescent (i.e., laminar) conditions [22,28].

$$\text{Turbulent conditions: Power} = k\rho n^3 D^5 \tag{2.15}$$

$$\text{Laminar conditions: Power} = k\mu n^2 D^3 \tag{2.16}$$

where P = power requirement, ft · lb/s (W)
μ = dynamic viscosity, lb · s/ft^2 (N · s/m^2)
ρ = fluid density, slug/ft^3 (kg/m^3)
D = diameter of impeller, ft (m)
n = revolutions per second, rev/s
k = constant (function of mixing equipment; see Table 2.22)

Equation (2.15) is applicable for mixing situations where the flow is turbulent or the Reynolds number is 10,000 or more. Equation (2.16) applies to mixing applications where the flow is laminar or where the Reynolds number is 10 or less. The Reynolds number may be estimated using Eq. (2.17). For flow conditions in which the Reynolds number is between 10 and 10,000, the power requirements for propeller and turbine systems may be estimated using procedures described in the following references [1,22].

$$\text{Reynolds no. (Re)} = \frac{D^2 n \rho}{\mu} \tag{2.17}$$

where D = diameter of impeller, ft (m)
n = revolutions per minute, rev/min

TABLE 2.22 Values of *k* for Mixing Equipment*

Impeller	Turbulent	Laminar
Propeller, square pitch, three blades	0.32	41.0
Propeller, pitch of two, three blades	1.00	43.5
Turbine, six flat blades	6.30	71.0
Turbine, six curved blades	4.80	70.0
Fan turbine, six blades	1.65	70.0
Flat paddle, six blades	1.70	36.5

*Adapted from ref. [28]

ρ = liquid density, slug/ft^3 (kg/m^3)
μ = dynamic viscosity, lb · s/ft^2 (N · s/m^2)

Mixers with small impellers operating at high speeds are most suitable for dispersing chemicals in wastewater, whereas mixing systems with large impellers operating at low speeds are best suited for maintaining solids in suspension (e.g., flocculation tanks). For readers interested in the design and use of static and pneumatic mixing systems, the following references are recommended [1,22].

2.4.3.2 Coagulation/flocculation. In many gravity sedimentation operations, chemicals must be added to enhance suspended solids removal. The chemicals are added to coalesce or "coagulate" the suspended solids particles into larger particles. The term *coagulation* is used to define the overall process of suspended solids aggregation [66]. Since most suspended solids particles in wastewater are negatively charged, they will repel each other when brought into close proximity. Fortunately, in addition to these strong repulsive forces, there exist weak attractive forces, known as *van der Waals forces,* that are primarily responsible for particle aggregation [60,66]. Since the van der Waals forces decrease rapidly with increasing distance between particles, the purpose of the coagulant chemical is to reduce the magnitude of the repulsive forces between particles (i.e., particle destabilization) so that they can be brought into close proximity of one another, permitting the van der Waals forces to promote particle aggregation.

Different chemical coagulants bring about particle destabilization through different means, including (1) charge neutralization, (2) enmeshment in a precipitate, (3) compression of diffuse layer, and (4) adsorption to permit interparticle bridging [26]. Knowledge of the various destabilization mechanisms is important in selection of the proper type and dosage of chemicals. Readers interested in the physical chemistry of particle destabilization are referred to the following references [26,66].

Flocculation is defined as the physical agitation of chemically treated water to induce particle coagulation. In this process, the destabilized suspended solids collide with one another, forming aggregates called *flocs* that will settle in the sedimentation basin. Flocculation normally is preceded by a rapid mixing tank in which the influent wastewater and coagulant chemical(s) are thoroughly mixed. The wastewater-coagulant mixture is then transferred to a flocculation process tank.

Various methods are used to accomplish flocculation, including the use of turbine mixing tanks, baffled tanks, and paddle flocculation tanks. The most common type of flocculator system is the paddle flocculator, which consists of a shaft with protruding steel arms on which are mounted wooden or metal blades (Fig. 2.30). The shaft of the paddle flocculator moves slowly (60–100 rev/min), causing a gentle agitation and the production of a floc that will readily settle in the sedimentation basin.

The extent of flocculation is directly proportional to the hydraulic retention time and the velocity gradient established by the stirring action. For municipal wastewater, hydraulic detention times of 15 minutes or less with velocity gradients of 40 to 60 feet per second per foot (fps/ft) have been found to be satisfactory [29]. For paddle flocculators, the mean velocity gradient G can be estimated using Eq. (2.18), whereas the power input to achieve a given velocity gradient can be estimated using Eq. (2.19) [or the more general relationship given by Eq. (2.13)].

$$G = \left(\frac{C_D A \rho \nu^3}{2V\mu} \right)^{1/2} \tag{2.18}$$

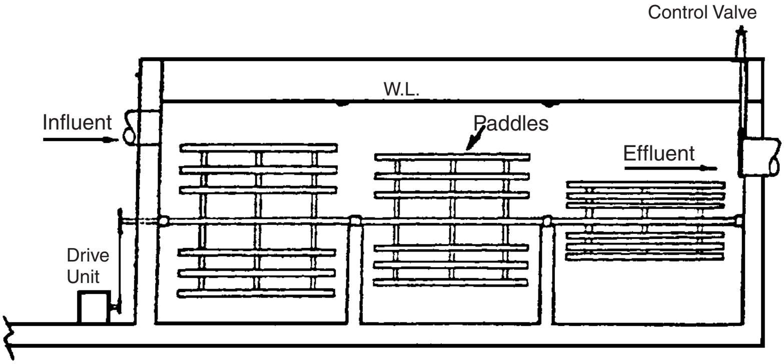
where G = velocity gradient, fps/ft
C_D = drag coefficient on paddles
A = cross-sectional area of paddles, ft^2 (m^2)
ρ = density of water, slug/ft^3 (kg/m^3)
ν = relative velocity between paddles and water (*ca.* 0.6 to 0.75 of paddle tip speed)
V = volume of flocculating basin, ft^3 (m^3)
μ = dynamic viscosity of water, lb · s/ft^2 (N · s/m^2)

$$\text{Power} = \frac{C_D A \rho \nu^3}{2} \tag{2.19}$$

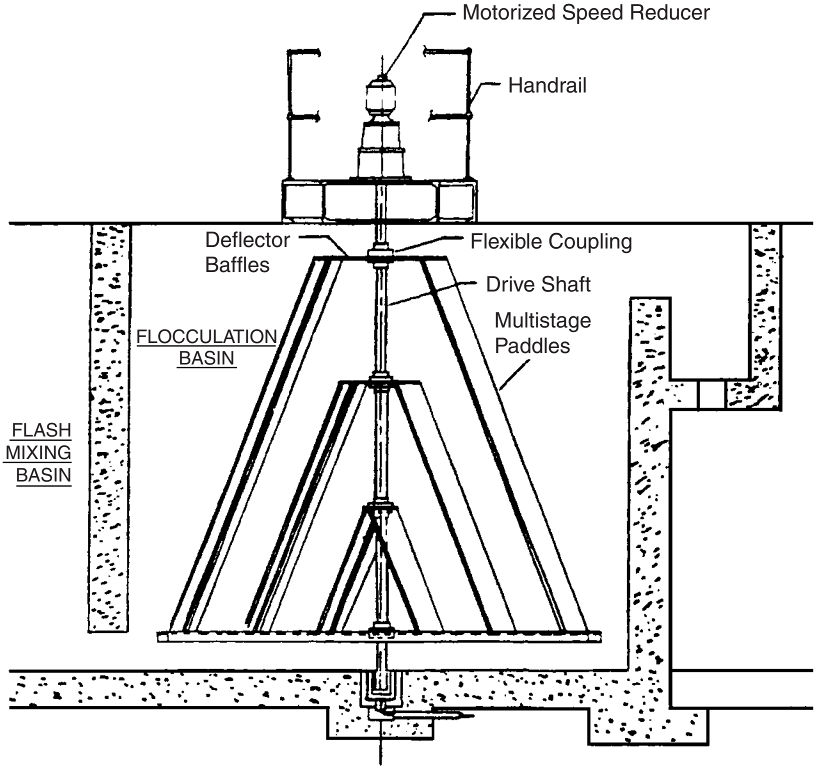
where C_D = drag coefficient on paddles
A = cross-sectional area of paddles, ft^2 (m^2)
ρ = density of water, slug/ft^3 (kg/m^3)
ν = relative velocity between paddles and water (*ca.* 0.6 to 0.75 of paddle tip speed)
μ = dynamic viscosity of water, lb · s/ft^2 (N · s/m^2)

Examples 2.5 and 2.6 illustrate the use of the mixing equations to estimate the power requirements for various wastewater treatment applications.

Example 2.5 Douglas County Water Conservancy District is installing a rapid mixing device to add a coagulant polymer to its influent wastewater. If the mixing equipment consists of a turbine mixer with six curved blades (3-ft diameter), estimate the power requirements if the blades are rotating



(a)



(b)

Figure 2.30 (*a*) Schematic diagram of a horizontal paddle type of flocculation tank. (*b*) Schematic diagram of a vertical paddle type of flocculation tank.

at a rate of 40 rev/min (0.67 rev/s). Assume that the average influent wastewater temperature is 70°F (21°C) and that the following data apply:

Density [at 70°F (21°C)]: -1.936 slugs/ft^3
Dynamic viscosity [at 70°F (21°C)]: $-2.05 \cdot 10^{-5}$ lb · s/ft^2

solution

Step 1. Estimate the Reynolds number under the present mixing conditions:

$$\mathrm{Re} = \frac{D^2 n \rho}{\mu}$$

$$= \frac{(3\ \mathrm{ft})^2\,(0.67\ \mathrm{rev/s})\,(1.936\ \mathrm{slug/ft^3})}{2.05 \cdot 10^{-5}\mathrm{lb} \cdot \mathrm{s/ft^2}}$$

$$= 56{,}947 \text{ (turbulent conditions)}$$

Since the mixing conditions are turbulent, Eq. (2.15) should be employed to estimate power requirements.

Step 2. Using Eq. (2.15) and $k = 70.0$ (turbine mixer with six curved blades), estimate the power requirement:

$$\text{Power} = k \rho n^3 D^5$$

$$= (70.0)\,(1.936\ \mathrm{slug/ft^3})\,(0.67\ \mathrm{rev/s})^3\,(3\ \mathrm{ft})^5$$

$$= \frac{9905\ \mathrm{ft} \cdot \mathrm{lb}}{\mathrm{s}} \quad (13.4\ \mathrm{kW})$$

NOTE: (550 ft · lb)/s=0.746 kW.

Example 2.6 The Douglas County Water Conservancy District (Example 2.5) has opted to add a coagulation/flocculation tank to its primary wastewater treatment system. If the tank will be a paddle-type flocculation system, estimate the power requirement and paddle area assuming that the following data apply:

1. Coagulation/flocculation tank volume: 500,000 gal
2. Desired velocity gradient: 60 fps/ft
3. Coefficient of drag on paddles C_D: 2.0
4. Paddle tip speed: 1.5 ft/s
5. Relative velocity of paddles: 0.7 times paddle tip speed
6. Water temperature: 70°F (21°C)
7. Density: 1.936 slugs/ft^3
8. Dynamic viscosity: $2.05 \cdot 10^{-5}$ lb · s/ft^2

solution

Step 1. Determine the power requirement by rearranging Eq. (2.13):

$$G = \left(\frac{P}{\mu V}\right)^{1/2}$$

or

$$\begin{aligned} P &= G^2 \mu V \\ &= (60\ \mathrm{s}^{-1})^2\,(2.05 \cdot 10^{-5}\ \mathrm{lb} \cdot \mathrm{s/ft}^2)\left(500{,}000\ \mathrm{gal} \cdot \frac{\mathrm{ft}^3}{7.48\ \mathrm{gal}}\right) \\ &= \frac{4933\ \mathrm{ft} \cdot \mathrm{lb}}{\mathrm{s}} \qquad (6.7\ \mathrm{kW}) \end{aligned}$$

Step 2. Use Eq. (2.19) to estimate the required paddle area:

$$\text{Power} = \frac{C_D A \rho \nu^3}{2}$$

or

$$\begin{aligned} \text{Area} &= \frac{2 \cdot \text{power}}{C_D \rho \nu^3} \\ &= \frac{2 \cdot \left(\dfrac{4933\ \mathrm{ft} \cdot \mathrm{lb}}{\mathrm{s}}\right)}{2.0\ (1.936\ \mathrm{slug/ft}^3)\ (0.70 \cdot 1.5\ \mathrm{ft/s})^3} \\ &= 2201\ \mathrm{ft}^2 \qquad (204.5\ \mathrm{m}^2) \end{aligned}$$

NOTE: $(550\ \mathrm{ft} \cdot \mathrm{lb})/\mathrm{s} = 0.746\ \mathrm{kW}$.

To properly employ the coagulation/flocculation process for enhanced suspended solids removal, the amount of coagulant chemical to add must be estimated together with the design of the mixing equipment. Example 2.7 illustrates the approach for estimating the amount of chemicals required in the coagulation/flocculation process together with the amount of chemical solids produced during primary treatment using the stoichiometric relationships from Table 2.20.

Example 2.7 The Poole County Wastewater Reclamation Facility is presently treating 4 million gallons per day (4 MGD) of domestic wastewater. Ferric chloride and lime are added in a flash mixer with the influent domestic wastewater to increase the efficiency of primary sedimentation. If 80 lb of ferric chloride is added per million gallon of wastewater treated, determine (a) the mass of chemical solids produced and (b) the number of pounds of lime required (as CaO).

solution

Step 1. Assuming that lime and ferric chloride are added in the appropriate stoichiometric amounts (see Table 2.20), develop the ratio of pounds of $Fe(OH)_3(s)$ generated per MGD treated.

$$\frac{80\ \text{lb FeCl}_3}{\text{MGD}} \cdot \frac{2 \cdot 106.9\ \text{lb Fe(OH)}_3(s)}{2 \cdot 162\ \text{lb FeCl}_3} = 52.8\ \frac{\text{lb Fe(OH)}_3(s)}{\text{MGD}}$$

For a flow rate of 4 MGD, the total mass of chemical solids produced per day would be

$$\frac{4\ \text{MG}}{\text{day}} \cdot \frac{52.8\ \text{lb Fe(OH)}_3}{\text{MG}} = \frac{211.2\ \text{lb Fe(OH)}_3}{\text{day}}$$

Step 2. From Table 2.20, use the ratio of pounds of lime required (as CaO) per pound of $Fe(OH)_3(s)$ produced to determine the total amount of lime required per day.

$$\frac{3 \cdot 56\ \text{lb CaO required}}{2 \cdot 106.9\ \text{lb Fe(OH)}_3\ (s)\ \text{produced}} \cdot \frac{211.2\ \text{lb Fe(OH)}_3\ (s)}{\text{day}} = \frac{165.9\ \text{lb CaO}}{\text{day}}$$

For situations in which a high degree of suspended solids removal is required to meet permit requirements, employing advanced flocculation and settling processes may be necessary. Several package systems are available commercially that employ combinations of inorganic coagulant, polymer, and microsand to enhance both suspended solids flocculation and settling. A typical advanced primary treatment system is illustrated in Fig. 2.31.

2.4.4 Sludge production from primary treatment

The management of sludge resulting from primary treatment represents a significant challenge to wastewater treatment operations. The

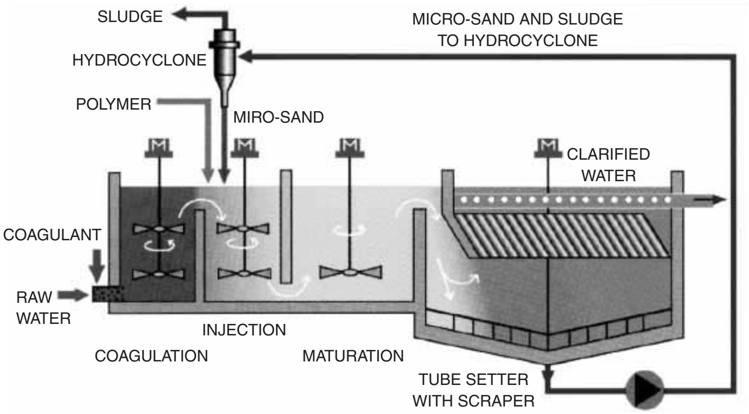


Figure 2.31 Advanced suspended solids and settling system—ACTIFLO. (*Courtesy of I. Kruger Inc.*)

daily volume of sludge generated in primary treatment can reach as much as 0.5 percent of the wastewater flow when lime is used as a chemical coagulant [32,66]. The volume of sludge in all solid-liquid separation processes (e.g., sedimentation, flotation etc.) may be estimated using Eq. (2.20):

$$\text{Volume (ft}^3) = \frac{W_s}{\rho_w S_{BS} P_s} \tag{2.20}$$

where W_s = weight of dry solids removed in treatment process (lb dry solids)

ρ_w = density of water (62.4 $\text{lb}_\text{m}/\text{ft}^3$)

S_{BS} = specific gravity of sludge (or biosolids)

P_s = percentage solids expressed as a decimal

To use Eq. (2.20), the weight of dry sludge (or biosolids) and its moisture content must be estimated. Estimation of the amount of dry solids removed requires some information regarding process efficiency. Once the influent suspended solids and treatment efficiency are known, the amount dry solids removed (dry weight) may be estimated using Eq. (2.21):

$$\text{Dry solids (lb/day)} = SS_\text{in} \cdot \%\text{ removal} \cdot \text{flow rate (MGD)} \cdot \frac{8.34\text{ lb}}{\text{MG} \cdot (\text{mg/liter})} \tag{2.21}$$

where SS_in = suspended solids in the influent wastewater (mg/liter)

% removal = removal efficiency (fraction)

MGD = million gallons per day

8.34 = conversion factor $\left[\dfrac{8.34\text{ lb}}{\text{MG} \cdot (\text{mg/liter})}\right]$

MG = million gallons

Examples 2.8 and 2.9 illustrate the approach used to estimate the volume of sludge that must be handled from a primary treatment process given the influent concentration of suspended solids and the efficiency of solids removal.

Example 2.8 Estimate the daily volume of primary sludge generated per million gallons from a typical medium-strength wastewater if the suspended solids removal efficiency in the primary clarifier is 60 percent. Assume that the specific gravity of the primary sludge is 1.03 and that it contains 94 percent moisture.

solution

Step 1. Estimate the suspended solids concentration. From Table 2.2, a medium-strength wastewater contains approximately 220 mg/liter suspended solids.

Step 2. Determine the mass of dry solids removed per million gallons of wastewater assuming 60 percent removal efficiency using Eq. (2.21):

$$\text{lb dry solids generated per day} = \frac{220\text{ mg}}{\text{liter}} \cdot (0.60) \cdot 1\text{ MGD} \cdot \frac{8.34\text{ lb}}{\text{MG} \cdot (\text{mg/liter})} = \frac{1100\text{ lb}}{\text{day}}$$

Step 3. Determine the volume of sludge per million gallons of wastewater treated using Eq. (2.20):

$$\text{Daily volume of biosolids produced (ft}^3\text{/day)} = \frac{1100\text{ lb/day}}{(1.03)\,(0.06)\,(62.4\text{ lb/ft}^3)} = \frac{285.2\text{ ft}^3}{\text{day}}$$

Example 2.9 Estimate the mass and volume of sludge produced in primary treatment with and without the use of ferric chloride for enhanced removal of suspended solids. Assume that 70 percent of the suspended solids is removed in the primary clarifier without chemical addition and that with the addition of ferric chloride the removal efficiency increases to 90 percent. Also assume that the following data apply:

1. Wastewater flow rate: 5.0 MGD
2. Influent suspended solids concentration: 250 mg/liter
3. Ferric chloride added: 60 lb/million gallons of wastewater treated
4. Sludge specific gravity (with $FeCl_3$): 1.05
5. Sludge specific gravity (without $FeCl_3$): 1.03
6. Moisture content (with $FeCl_3$): 92.5 percent
7. Moisture content (without $FeCl_3$): 94.0 percent

solution

Step 1. Compute the mass of suspended solids removed without and with ferric chloride using Eq. (2.21).

Mass of solids removed without $FeCl_3$ addition

$$\text{Dry solids (lb/day)} = SS_{\text{in}} \cdot \%\text{ removal} \cdot \text{flow rate (MGD)} \cdot \frac{8.34\text{ lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= 250 \text{ mg/liter} \cdot 0.70 \cdot 5 \text{ MGD} \cdot \frac{8.34 \text{ lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= \frac{7297.5 \text{ lb}}{\text{day}}$$

Mass of solids removed with $FeCl_3$ addition

$$\text{Dry solids (lb/day)} = SS_{\text{in}} \cdot \% \text{ removal} \cdot \text{flow rate (MGD)} \cdot \frac{8.34 \text{ lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= 250 \text{ mg/liter} \cdot 0.90 \cdot 5 \text{ MGD} \cdot \frac{8.34 \text{ lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= \frac{9382.5 \text{ lb}}{\text{day}}$$

Step 2. Determine the mass of ferric hydroxide [$Fe(OH)_3$] produced from the addition of 60 lb $FeCl_3$ per million gallons of wastewater processed using Table 2.20:

$$\text{Mass of } Fe(OH)_3(s) \text{ formed (lb/day)} = \frac{60 \text{ lb } FeCl_3}{\text{MG}} \cdot \frac{5 \text{ MG}}{\text{day}} \cdot \frac{106.9 \text{ lb } Fe(OH)_3(s)}{162.1 \text{ lb } FeCl_3}$$

$$= \frac{197.8 \text{ lb } Fe(OH)_3(s)}{\text{day}}$$

Step 3. Determine the total dry mass of sludge including chemical precipitation produced per day when $FeCl_3$ is used in primary treatment:

$$\text{Biosolids produced (lb/day)} = \frac{9382.5 \text{ lb}}{\text{day}} + \frac{197.8 \text{ lb}}{\text{day}}$$

$$= \frac{9580.3 \text{ lb}}{\text{day}}$$

Step 4. Determine the total volume of sludge produced per day when $FeCl_3$ is *not* used in primary treatment using Eq. (2.20):

$$\text{Volume produced per day} = \frac{W_s}{\rho_w S_{BS} P_s} = \frac{7297.5 \text{ lb/day}}{\frac{62.4 \text{ lb}}{\text{ft}^3} \cdot 1.03 \cdot 0.06}$$

$$= \frac{1892.3 \text{ ft}^3}{\text{day}}$$

Step 5. Determine the total volume of sludge produced per day when $FeCl_3$ is used in primary treatment using Eq. (2.20):

$$\text{Volume produced per day} = \frac{W_s}{\rho_w S_{BS} P_s} = \frac{9580.3\ \text{lb/day}}{\dfrac{62.4\ \text{lb}}{\text{ft}^3} \cdot 1.05 \cdot 0.075}$$

$$= \frac{1949.6\ \text{ft}^3}{\text{day}}$$

2.4.5 Screening

In addition to gravity sedimentation and flotation, the use of fine screens is another primary wastewater treatment option resulting in the generation of sludge. The three most common types of screens are (1) inclined screens, (2) rotary-drum screens, and (3) rotary-disk screens (Fig. 2.32). Typical design values for the various screens are provided in Table 2.23.

When screens are used as an alternative to sedimentation (or flotation), the secondary wastewater treatment system must be sized properly to handle the solids and organic loading. Readers interested in additional details on the design of screening devices for primary wastewater treatment are referred to the following references [22,28,60].

2.5 Secondary Wastewater Treatment

Secondary treatment refers to the biological processing of wastewater. The objectives of secondary treatment are to coagulate and remove the nonsettleable colloidal solids and to stabilize the influent organic matter. The biological coagulation of nonsettleable solids and stabilization of organic matter are accomplished by a variety of microorganisms but principally by aerobic and facultative heterotrophic bacteria [12]. Microorganisms use the influent organic matter in the synthesis of new microbial cells as well as for the generation of maintenance energy. In secondary wastewater treatment systems, aerobic microorganisms mediate the biochemical reactions described by Eqs. (2.22) and (2.23).

Oxidation and synthesis

$$\underset{\text{organic matter}}{\text{COHNS}} + O_2 + \text{nutrients} \xrightarrow[\text{microorganisms}]{} \underset{\text{new microbial cells}}{C_5H_7O_2N} + CO_2 + NH_3 + \text{other products} \tag{2.22}$$

Endogenous respiration

$$\underset{\text{mol. wt. 113}}{C_5H_7O_2N} + \underset{\text{mol. wt. 160}}{5O_2} \xrightarrow[\text{microorganisms}]{} 5CO_2 + 2H_2O + NH_3 + \text{energy} \tag{2.23}$$

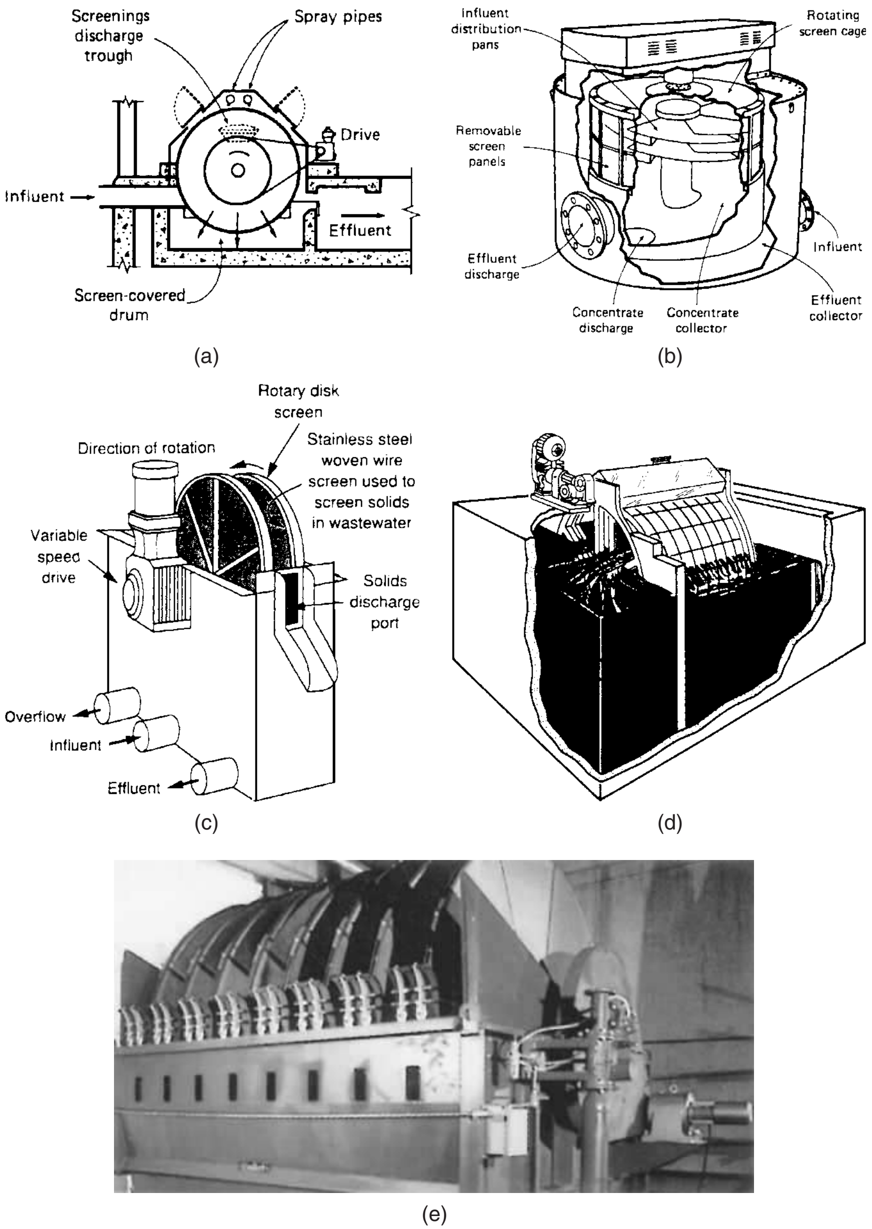


Figure 2.32 Schematic diagrams of screening devices: (*a*) rotary-drum screen; (*b*) centrifugal screen; (*c*) rotary-disk screen; (*d*) microscreen. (*e*) Rotary-disk screen. (*Courtesy of WesTech Engineering, Inc.*)

TABLE 2.23 Design Values for Screening Devices*

	Type of screen		
Parameter	Inclined	Rotary drum	Rotary disk
Size classification	Medium	Medium	Fine
Size range, in	0.01–0.06	0.01–0.06	0.001–0.01
Screen material	Stainless steel wedge wire	Stainless steel wedge wire	Stainless steel woven wire
Hydraulic capacity, gal/ft^2·min	15–60	0.12–1.0	0.1–1.0
Suspended solids removal (%)	15–30	15–30	40–50

*Adapted from ref. [28].

Over the past 90 years, many different secondary (or biological) systems have been designed and operated for the treatment of municipal wastewater. The most commonly used secondary treatment processes include (1) activated sludge, (2) trickling filters, (3) rotating biological contactors, (4) aerated lagoons, and (5) stabilization ponds. Although all five systems are effective in treating wastewater, it is only the first three biological systems (known as *mechanical secondary systems*) that are characterized by the continuous production of secondary sludge that must be managed on a daily basis. Aerated lagoons and stabilization ponds (passive systems) also generate sludge. However, the sludge generated in these systems normally settles within the treatment basin and is treated and disposed on an infrequent basis [12,28].

2.5.1 Activated-sludge process

The activated-sludge process is an aerobic suspended-growth wastewater treatment system in which microorganisms use the organic content of the wastewater (i.e., biochemical oxygen demand, BOD) as an energy source and for the synthesis of new cell material. Activated-sludge systems have been used for secondary wastewater treatment for over 75 years and have grown to become the most widely used secondary wastewater treatment option [12,28,60]. Although there are many variations of the activated-sludge system, the factors common to them all include the following: (1) a flocculent slurry of aerobic microorganisms maintained in an aeration basin is responsible for removing soluble organic matter from influent wastewater, (2) aerobic conditions are maintained through the use of a gas diffuser or mechanical agitation system, (3) flocculent microorganisms (i.e., secondary sludge) that leave the aeration basin are removed from the discharged wastewater through a secondary clarification (or sedimentation) system, (4) a portion of the concentrated underflow from the secondary clarification system is recycled to the aeration basin to maintain a suitable microbial population, and (5) a small mass of sludge must be removed periodically (or wasted) from the activated sludge system to maintain process control.

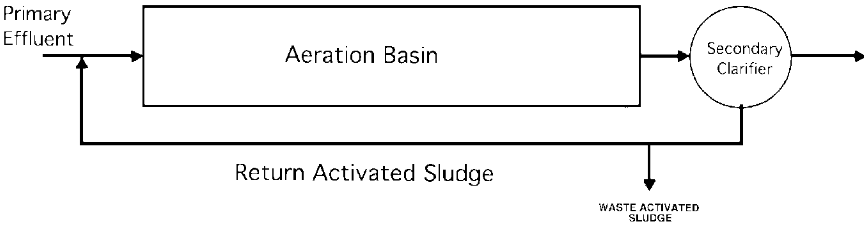
While the soluble organic matter contained in the influent wastewater is removed by microbial assimilation, particulate organic matter is physically entrapped in the flocculent sludge. During wastewater treatment, a portion of the particulate organic matter may be solubilized by exocellular microbial enzymes and rendered available for bacterial assimilation. The mixture of microorganisms and particulate matter in the aeration basin is called the *mixed-liquor suspended solids* (MLSS) or *activated sludge*.

2.5.1.1 Activated-sludge process configuration. The various process configurations available for activated-sludge systems present numerous design options for the design engineer. Normally, a particular design is chosen based on an economic assessment, but other factors (e.g., wastewater treatment objectives) are taken into account as well. The conventional activated-sludge system configuration employs a plug flow reactor design in which long rectangular aeration basins are used to contain the active microbial populations. The influent wastewater and concentrated underflow recycle are introduced at one end of the basin, and the clarified effluent is removed from the other end (Fig. 2.33).

To achieve adequate oxygen transfer and mixing, the basins are equipped with mechanical aerators or gas diffusers. Thus the name *aeration basin* normally is used to describe the tank in which microbial removal of organic matter is occurring. A typical design of a conventional activated-sludge system is the classic oxidation ditch, which is still used in many communities today (Fig. 2.34).

Because of the limited mixing achieved in a plug flow design, it was soon observed that for some wastewater treatment applications, portions of the aeration basin were becoming oxygen-limited, resulting in a loss of process efficiency [12]. By monitoring both the BOD and dissolved oxygen concentrations along the length of the basin, it was recognized that more oxygen was required near the influent end of the basin, where the organic loading is higher, than near the effluent end, where the oxygen demand is low [12,28].

The first major variation of the activated-sludge system was the tapered aeration system, in which the design objective was to match the rate of oxygen supply with the oxygen demand at various locations along the aeration basin length (Fig. 2.35). This approach to aeration basin design has become the standard practice for activated-sludge systems employing the plug flow design concept [12,28,60].



(a)

(b)

Figure 2.33 (*a*) Schematic diagram of a conventional plug flow activated-sludge system. (*b*) Plug flow activated-sludge system.

Figure 2.34 An oxidation ditch activated-sludge system.

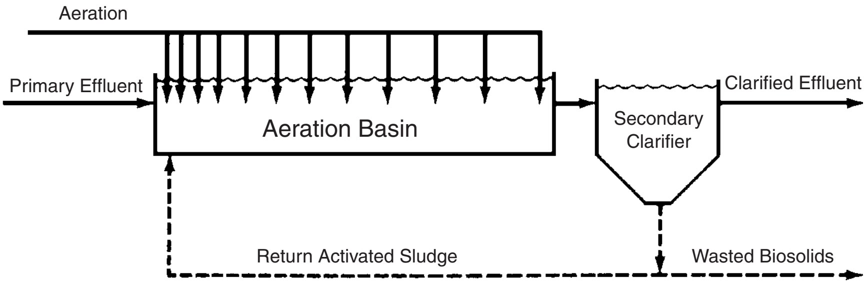


Figure 2.35 Schematic diagram of a tapered activated-sludge system.

By employing aeration and hydraulic flow computer models, the positioning of surface aerators is optimized to both transfer the appropriate amount of oxygen at the required rate and to maintain a suitable hydraulic retention time for effective wastewater treatment [12]. The result is a cost-effective aeration basin design that employs the minimum power requirements necessary for achieving the proper oxygen transfer and mixing requirements.

Another activated-sludge design modification developed to more effectively match oxygen supply with oxygen demand was the activated step-aeration process [12]. In this system, more efficient use of the aeration capacity of the plug flow design is obtained by splitting the influent wastewater flow into several streams that are fed into the aeration basin at different points. This design modification provides a more even distribution of the influent oxygen demand, thus eliminating oxygen-limited conditions in the aeration basin without the need to reposition aerators (Fig. 2.36).

In addition to matching oxygen supply with microbial oxygen demand, it was recognized that the size of the aeration basin could be decreased (thereby reducing both the activated-sludge system construction and operating costs) if the rate of organic removal could be enhanced [28]. This recognition led to several additional modifications

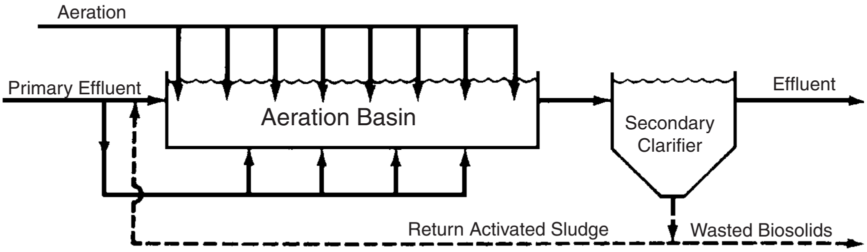


Figure 2.36 Step-aeration activated-sludge design.

to the activated-sludge process. The first such modification was the contact stabilization process, whose design objective was to maximize the microbial activity of the recycled sludge (Fig. 2.37).

To enhance microbial activity of the recycled sludge, the return activated sludge (RAS) is pumped to a stabilization tank where it is vigorously aerated prior to mixing with the influent wastewater in the contact tank. The benefit of aerating the recycled sludge is that through oxygen stimulation, the microbial cells are rendered capable of using the influent organic matter at significantly enhanced rates in the contact tank [12]. The result of the increased microbial activity is that the required wastewater detention time in the contact tank is much less (*ca.* 1–2 hours) than in a conventional activated sludge aeration basin (*ca.* 6 hours). Consequently, even though the contact stabilization system uses two basins, the total basin volume is significantly less than that required by conventional activated-sludge systems [12,60].

Although the conventional plug flow activated-sludge system design and its various process modifications were found to be adequate in treating most domestic wastewater to the required permit levels, industrial discharges of concentrated organic or toxic wastes into the municipal sewage collection system occasionally caused unstable conditions at many wastewater treatment facilities [28,60]. To overcome these operational difficulties, many activated-sludge systems today employ a completely mixed activated-sludge system (Fig. 2.38). By maintaining completely mixed conditions in the aeration basin, the microbial populations are maintained in a relatively constant growth condition that better enables them to treat any shock loads of organic and/or toxic compounds.

The classic definition of complete mixing requires that the influent wastewater be dispersed instantaneously throughout the aeration basin and that the concentrations of the biochemical oxygen demand (BOD), microbial cells, oxygen, nutrients, etc. everywhere in the aeration basin be identical. Although the former requirement cannot be met practically by completely mixed activated-sludge systems, the latter requirement normally is achievable when the power input to the aeration basin from a mechanical aerator is at least 0.5 hp per 1000 ft^3 of basin volume (i.e., 13.8 kW per 1000 m^3) or greater [12,28]. Alternatively, for diffused-air aeration systems, an airflow rate to the aeration basin of 20 standard cubic feet per minute (scfm) per 1000 ft^3 of basin volume (20 m^3/1000 m^3) or greater is normally sufficient to achieve complete mixing [12,28].

Although increasing the power or air input will improve the mixing characteristics of the aeration basin, care must be taken not to use excessive mixing that could adversely affect the overall activated-sludge system performance. As an upper limit, the power input should not

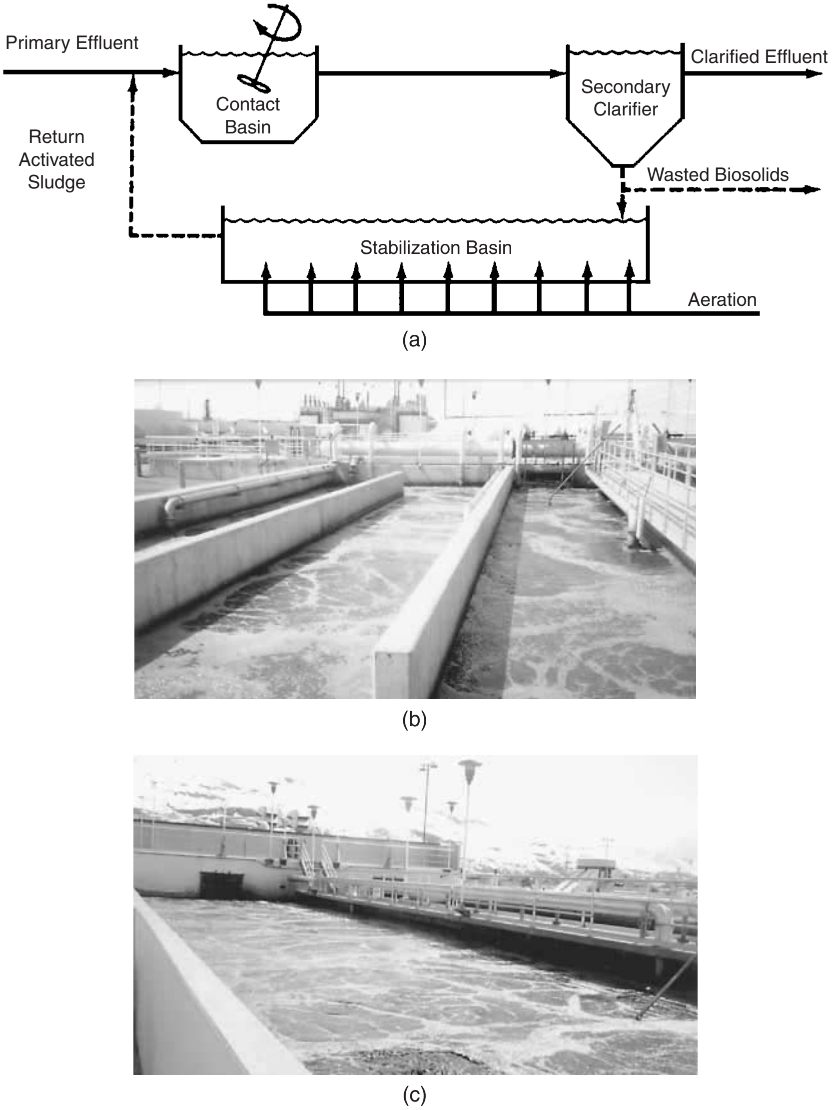


Figure 2.37 (*a*) Schematic diagram of a contact stabilization activated-sludge process. (*b*) Plug flow contact basin. (*c*) Stabilization basin.

exceed 2.25 hp per 1000 ft^3 of aeration basin volume (60 kW/1000 m^3) or 90 scfm per 1000 ft^3 of aeration basin volume in order to avoid shearing of the microbial flocs [12]. The loss of flocculent behavior by the microbial cells would have detrimental effects on the performance of the secondary clarification system. When the aeration basin volume and the operating range of mixed-liquor suspended solids concentrations are

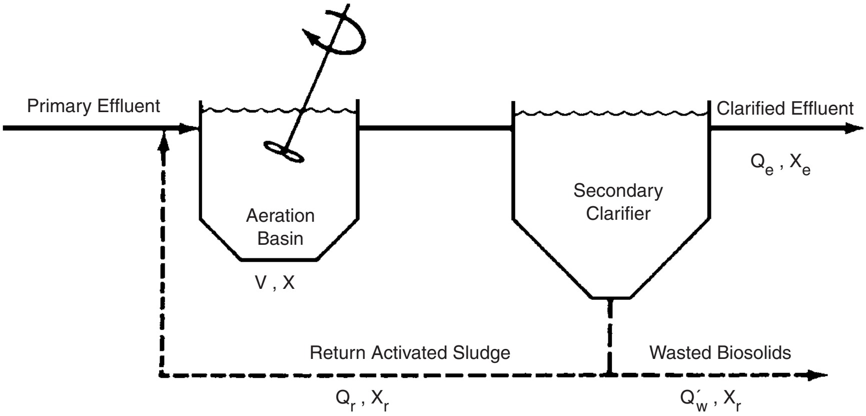


Figure 2.38 Schematic diagram of a completely mixed activated-sludge process.

(a)

(b)

Figure 2.39 Completely mixed activated-sludge process: (*a*) system in operation; (*b*) system drained for repair.

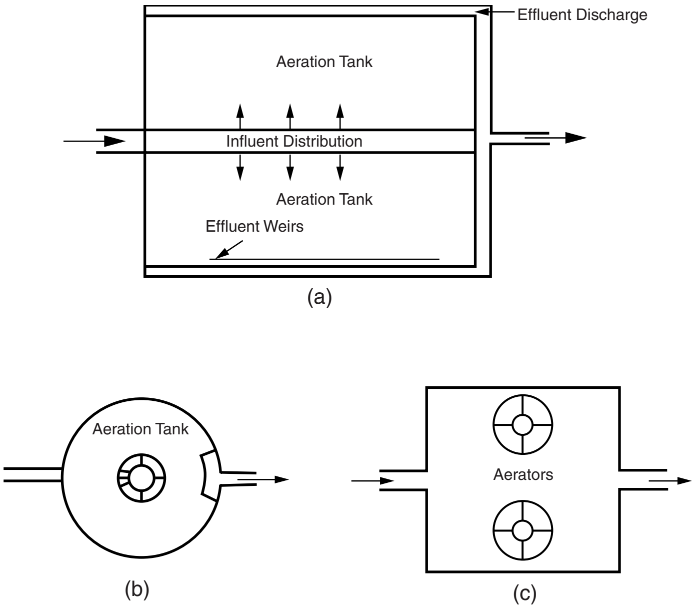


Figure 2.40 (*a*) Rectangular aeration basin. (*b*) Circular aeration basin. (*c*) Square aeration basin. (*Adapted by permission from ref. [12].*)

determined, the power input for oxygen transfer should be evaluated against the power input required to achieve complete mixing.

There are currently no well-established guidelines concerning the most suitable aeration basin configuration for attaining complete mixing. Generally, the aeration basins for completely mixed activated-sludge systems are of three types: square, round, or rectangular (Fig. 2.40). Square and round basins tend to be used more with mechanical aeration systems, whereas rectangular basins are used for both mechanical and diffused-air systems [12]. Basin depths are generally 12 to 15 ft, with widths (or diameters for circular aeration basins) of 25 to 50 ft [12,28].

For wastewater treatment plants in the capacity of 0.5 to 10 MGD, at least two basins should be provided for operational flexibility. In the range of 10 to 50 MGD, at least four basins often are provided, whereas for over 50 MGD, six basins or more should be installed. Individual basins should have inlet and outlet gates or valves so that they may be removed from service for inspection and repair.

For efficient use of aeration basins, a method of splitting or controlling the flow rate to each of the individual basins should be employed. Methods commonly used are splitter boxes equipped with weirs or control valves or aeration basin influent control gates. Readers interested

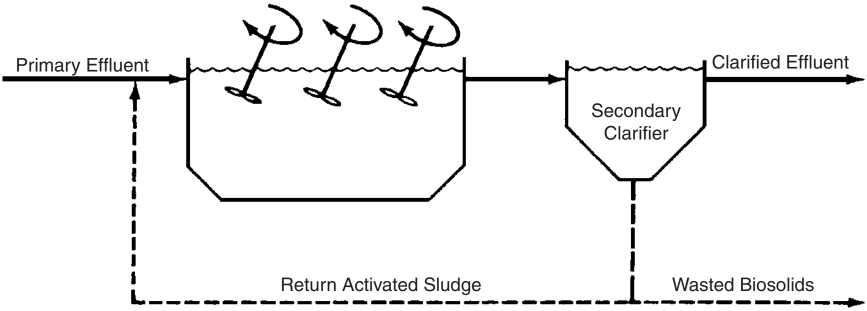


Figure 2.41 Schematic diagram of an extended aeration activated-sludge system.

in further design details on activated-sludge aeration basins for completely mixed systems are referred to the following references [12,24,28,63].

The generation and disposal of significant amounts of sludge from activated-sludge systems have always represented an operational challenge for municipal wastewater treatment plants employing mechanical systems. To minimize the generation of sludge, an additional design modification to the conventional activated-sludge system known as *extended aeration* was developed (Fig. 2.41).

In the extended aeration system, the design objective is to maintain a long hydraulic retention time (typically 24 hours or more) within the aeration basin together with a large concentration of mixed-liquor suspended solids (typically 4000 to 9000 mg/liter). Theoretically, under these conditions, the microbial cells are maintained essentially within the endogenous growth phase [28]. From a practical standpoint, this condition results in reducing the rate of sludge production in the aeration basin to the rate at which suspended solids are removed in the clarified effluent from the secondary clarification system. In such a process, the theoretical sludge wastage rate would be reduced to zero. However, data from the field application of this activated-sludge design have indicated that the concentration of suspended solids in the final clarified effluent is often above the permissible discharge limit [12,60].

It is now common practice to waste a small amount of sludge periodically from extended aeration systems to control discharge wastewater effluent quality [12]. Because of the long hydraulic retention times, together with the large mixed-liquor suspended solids required for proper operation, the extended aeration design is normally limited to small communities or package plants [28].

Finally, it has been recognized that, in general, by increasing the oxygen transfer rate to the aeration basin, the rate of microbial removal of organic matter can be increased [12]. Enhanced rates of organic matter removal would result in a smaller required aeration basin volume and/or permit a greater throughput of wastewater. However, the use of air as the oxygen source meant that to increase the oxygen transfer rate, the aeration tank would need to receive greater agitation. The practical limitation to this approach was that enhanced agitation (i.e., mixing) eventually would lead to shearing of the microbial floc particles and poor sludge settling behavior in the secondary clarification system. This dilemma led to the design of the pure oxygen activated-sludge system (Fig. 2.42). By employing pure oxygen rather

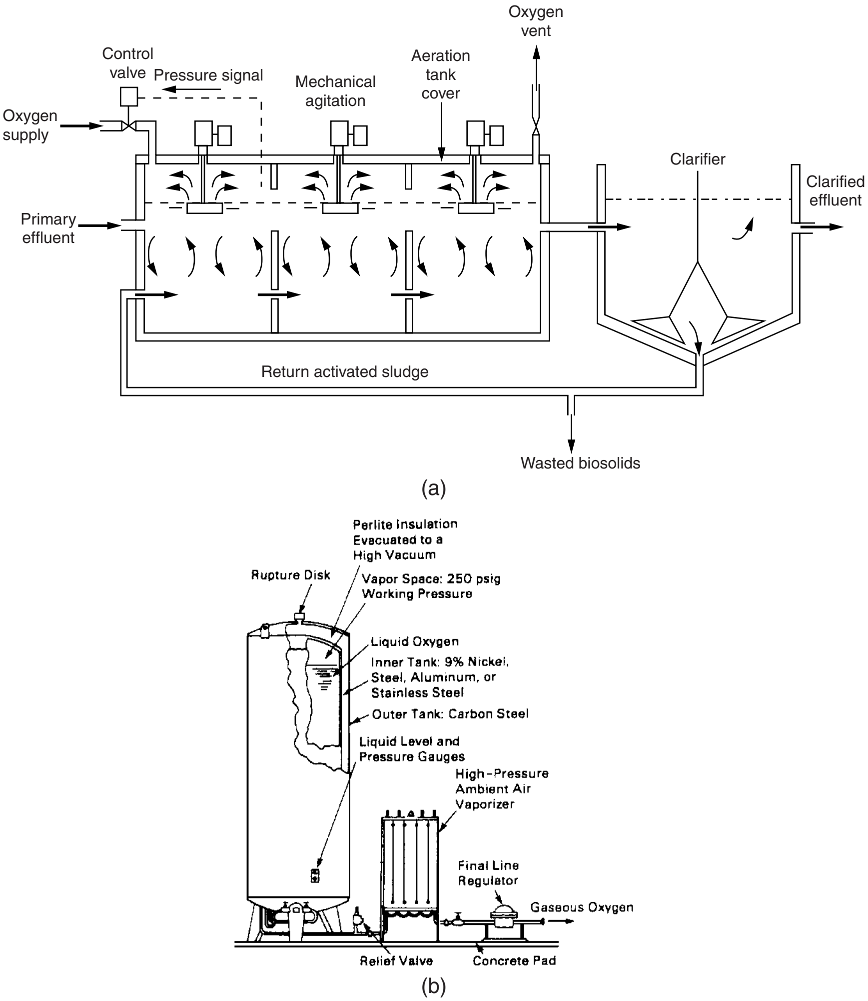


Figure 2.42 Schematic diagram of (*a*) pure oxygen activated-sludge system; (*b*) typical liquid oxygen storage system.

than air as the oxygen source, the oxygen transfer rate could be increased by a factor of five without the requirement for increased mixing [12]. The most important limitation to the application of the pure oxygen activated-sludge system was the development of a system that would use oxygen cost-effectively.

The options presently available for obtaining a pure oxygen gas stream consist of either purchasing (and storing) liquid oxygen at the wastewater treatment plant (which is gasified when needed) or manufacturing a pure oxygen gas stream directly from ambient air by using one of the standard industrial oxygen-generating processes (e.g., pressure swing adsorption or the cryogenic oxygen generation systems) [12]. Both options add significant costs to the overall wastewater treatment system operations. Moreover, if the pure oxygen stream were added directly to an uncovered aeration basin, the amount of oxygen escaping to the atmosphere would be considerable, resulting in additional economic concerns.

Pure oxygen systems typically use covered tanks that are operated in a stepwise fashion (see Fig. 2.42). Pure oxygen is introduced into the first stage, and the off-gases (unused oxygen and carbon dioxide) are passed on to stage two. This scheme is continued through the various stages until practically all the oxygen is consumed within the activated-sludge system. Because of the large oxygen transfer rates, pure oxygen activated-sludge systems typically can operate at hydraulic retention times of 2 hours or less for many wastewater treatment applications [12,28].

In addition to the activated-sludge process configuration, other factors that affect sludge production and management must be considered in the overall system operation. These factors include (1) loading criteria, (2) sludge production, (3) return activated sludge (RAS) rates, (4) oxygen requirements, (5) mixing requirements, (6) nutrients, (7) control of filamentous microbes, and (8) rising sludge. Each of these factors is discussed in the following sections.

2.5.1.2 Loading criteria. *Loading criteria* refer to the methods employed to control the level of microbial activity (and thereby wastewater treatment efficiency) in the activated-sludge aeration basin. The two most commonly used loading criteria for activated-sludge process control are: (1) the food-to-microorganism ratio F/M and (2) the mean cell residence time θ_c. The *food-to-microorganisms ratio* is defined as the mass of organic matter transferred daily to the aeration basin divided by the mass of microorganisms within the aeration tank (measured as volatile suspended solids, VSS). This ratio is described quantitatively by Eq. (2.24):

$$\frac{F}{M} = \frac{S_0}{\theta X} \tag{2.24}$$

where F/M = food-to-microorganism ratio $\left(\dfrac{\text{mass of BOD discharged to aeration basin/day)}}{\text{mass of cells in aeration basin}}\right)$, day^{-1}

S_0 = influent $\mathrm{BOD_L}$, mg/liter

θ = hydraulic retention time of aeration basin, V/Q

V = volume of aeration tank, MG (million gallons) or cubic meters

Q = influent wastewater flow rate, MGD (million gallons per day) or m^3/day

X = concentration of microorganisms in aeration basin (measured as VSS)

VSS = volatile suspended solids, mg/liter

$\mathrm{BOD_L}$ = ultimate biochemical oxygen demand, mg/liter

$\mathrm{BOD_5}$ = 5-day biochemical oxygen demand, mg/liter ($\mathrm{BOD_5} \cong \mathrm{BOD_L} \cdot 0.68$)

0.68 = conversion factor

Typical values for the F/M ratio in a properly operating activated-sludge wastewater treatment system will vary from 0.05 to 1.0 [28]. The choice of the operating F/M ratio will depend on the activated-sludge process configuration and operating conditions.

Another term that is related to the F/M ratio and is often used to describe the removal of organic matter in the aeration basin is the *specific substrate utilization rate*. The specific substrate utilization rate U is defined as the mass of BOD removed daily from the aeration basin per unit mass of microorganisms within the aeration basin. The relationship between the specific substrate utilization rate U and the F/M ratio is described by Eq. (2.25):

$$U = \frac{(F/M)\,E}{100} \tag{2.25}$$

where U = specific substrate utilization rate $\left(\dfrac{\text{mg BOD removed}}{\text{mg VSS} \cdot \text{day}}\right)$

F/M = food to microorganism ratio, day^{-1}

E = BOD removal efficiency, $\left(\dfrac{S_0 - S}{S_0}\right) \cdot 100$

S_0 = influent $\mathrm{BOD_L}$ concentration, mg/liter

S = effluent $\mathrm{BOD_L}$ concentration, mg/liter

$\mathrm{BOD_L}$ = ultimate biochemical oxygen demand, mg/liter

BOD_5 = 5-day biochemical oxygen demand, mg/liter ($BOD_5 \cong BOD_L \cdot 0.68$)
0.68 = conversion factor

To apply the F/M ratio (or U) appropriately in estimating the mass of sludge generated within an activated-sludge system, the relationship between substrate removed and sludge produced must be developed. It has been recognized generally that the ratio of the mass of sludge generated per mass of substrate removed is constant for a given substrate [12]. Therefore, the relationship between substrate removal and sludge generation may be described quantitatively by the maximum yield coefficient. The *maximum yield coefficient* Y is defined as the mass of sludge generated per mass of organic matter (measured, in this case, as ultimate BOD) removed. Multiplication of the maximum yield coefficient Y by the specific substrate utilization rate U and average sludge concentration X will provide an estimate of the sludge produced within the aeration basin [Eq. (2.26)].

$$\text{Sludge generation rate}\left(\frac{\text{mass}}{\text{unit volume}\cdot\text{day}}\right) = YUX = \frac{Y\,(F/M)\,E}{100}\,X \qquad (2.26)$$

where Y = maximum yield coefficient (mass of VSS produced/mass of BOD removed), lb/lb, mg/mg (kg/kg)
X = sludge concentration in aeration basin, mg/liter (measured as the mixed liquor volatile suspended solids concentration)

Although Eq. (2.26) is useful in estimating the maximum mass of sludge generated within the aeration basin as a function of organic matter removed, the actual mass produced will be less than that estimated using this equation. The reason for the lower sludge generation rate is that there is a continuous loss of sludge within the activated-sludge system due to endogenous respiration. *Endogenous respiration* is a comprehensive term used to quantitatively describe the cumulative loss of sludge due to (1) microbial use of stored energetic compounds (e.g., carbohydrates, proteins, etc.), (2) microbial cell death, and (3) predation [12]. The loss of sludge due to endogenous respiration is assumed to be proportional to the concentration of sludge within the aeration basin X [60]. Equation (2.26) may be modified to reflect this loss of sludge, which results in the development of an expression for estimating the net sludge production rate [Eq. (2.27)]. It should be noted that it is the net mass of sludge produced within the activated-sludge system that must be removed daily to maintain steady-state conditions (i.e., no sludge accumulation).

$$\text{Net rate of sludge production}\left(\frac{\text{mass}}{\text{unit volume}\cdot\text{day}}\right)$$

$$= YUX - k_dX = Y\,\frac{(F/M)\,E}{100}\,X - k_dX \qquad (2.27)$$

where k_d = endogenous decay coefficient, day^{-1}. (NOTE: All other terms are as previously defined.)

The other loading criterion used for activated-sludge process control is the mean cell residence time θ_c. The *mean cell residence time* (MCRT) is defined as the mass of microorganisms within the aeration tank divided by the rate at which microorganisms are removed from the system [Eq. (2.28)]. Note that the parameters used in Eq. (2.28) were defined in Fig. 2.38.

$$\theta_c = \frac{\text{mass of microorganisms within aeration tank}}{\text{microorganisms removed from system}}$$

$$= \frac{VX}{Q'_w\,X_r + Q_eX_e} \qquad (2.28)$$

where θ_c = mean cell residence time, days
V = aeration tank volume, MG (m^3)
X = concentration of microorganisms in aeration tank, mg MLVSS/liter
Q'_w = volumetric flow rate of waste sludge, MGD (m^3/day)
X_r = concentration of microorganisms in waste sludge, mg VSS/liter
Q_e = treated effluent flow rate from secondary clarifier, MGD (m^3/day)
X_e = concentration of microorganisms in treated effluent, mg VSS/liter

The mean cell residence time θ_c is an estimate of the average time microbial cells remain in the activated-sludge system. Dividing the sludge concentration in the aeration basin X by the mean cell residence time provides an estimate of the removal rate of sludge. Since the sludge removal rate is equivalent to the sludge growth rate at steady-state conditions, by maintaining a constant mean cell residence time, the rate of microbial growth within the aeration basin can be controlled. In other words, choosing a θ_c fixes both the rate at which microbial cells (i.e., sludge) are generated within the aeration basin and the rate at which microbial solids must be removed.

To ensure no net accumulation of sludge (i.e., solids) within the activated-sludge system (i.e., steady-state conditions), sludge removal must be balanced by the rate at which sludge is generated within the system. This requirement leads to Eq. (2.29), which illustrates the fundamental relationship between the two loading rate criteria:

$$\text{Solids removed}\left(\frac{\text{mass of VSS}}{\text{unit volume}\cdot\text{day}}\right) = \text{net solids produced}\ \frac{\text{mass of VSS}}{\text{unit volume}\cdot\text{day}}$$

$$\frac{X}{\theta_c} = Y\left(\frac{F}{M}\right)\left(\frac{E}{100}\right)X - k_d X \tag{2.29}$$

$$\frac{1}{\theta_c} = Y\left(\frac{F}{M}\right)\left(\frac{E}{100}\right) - k_d$$

where Y = maximum yield coefficient (mass of VSS produced/mass of BOD removed), lb/lb (kg/kg)

k_d = endogenous decay coefficient, day^{-1}

NOTE: All other terms are as previously defined.

Typical values for the activated-sludge kinetic coefficients used in Eq. (2.29) are given in Table 2.24. Although the values of the kinetic parameters given in Table 2.24 may be employed for preliminary design and economic analyses, the kinetic coefficients used in the actual full-scale activated-sludge process design should be determined from the results of laboratory treatability studies using site-specific wastewater samples [12].

Although the F/M ratio is still used for process control at some wastewater treatment facilities, the simplicity of routinely measuring the mean cell residence time θ_c has led many plants to adopt it as the basis of their process control [12]. Therefore, the remainder of the activated-sludge system design and operation discussion will highlight the use of θ_c as the principal process-control parameter.

TABLE 2.24 Typical Kinetic Coefficients for the Activated-Sludge Process*

Coefficient	Unit	Range	Typical value
Y	lb (kg) VSS/lb (kg) BOD	0.4–0.8	0.6
k_d	day^{-1}	0.02–0.08	0.06

*Adapted from refs. [12,28].

2.5.1.3 Sludge production. For proper design of sludge-management systems, it is critical that the design engineer be able to estimate the quantity of sludge produced per day. The quantity of secondary sludge produced on a daily basis may be estimated from the anticipated organic matter removal efficiency and the observed microbial yield [Eq. (2.30)]:

$$P_x = Y_{\text{obs}} Q\,(S_0 - S)\left(\frac{8.34\text{ lb}}{\text{MG}\cdot\text{mg/liter}}\right) \qquad \text{(US units)}$$

$$P_x = Y_{\text{obs}} Q\,(S_0 - S)\left(\frac{\text{kg}}{10^3\text{ g}}\right) \qquad \text{(SI units)} \tag{2.30}$$

where P_x = sludge production rate (lb/day, kg/day) , dry solids
Y_{obs} = observed yield {[lb (kg) solids generated as VSS]/[lb (kg) of BOD_L removed]}
Q = flow rate (MGD, or m^3/day)
S_0 = influent BOD_L (mg/liter or g/m^3) to secondary treatment system
S = effluent BOD_L (mg/liter or g/m^3) from secondary treatment system
BOD_L = ultimate biochemical oxygen demand, mg/liter
BOD_5 = 5-day biochemical oxygen demand, mg/liter ($\text{BOD}_5 = \text{BOD}_\text{L} \cdot 0.68$)
0.68 = conversion factor

It should be noted that the observed microbial yield coefficient Y_{obs} is a function of the maximum microbial yield coefficient Y, the endogenous decay coefficient k_d, and the mean cell residence time θ_c [12,28]. This relationship is described by Eq. (2.31):

$$Y_{\text{obs}} = \frac{Y}{1 + k_d\theta_c} \tag{2.31}$$

where Y_{obs} = observed yield {[lb (kg) solids generated as VSS]/[lb (kg) of BOD_L removed]}
Y = maximum yield {[maximum mass of biosolids generated lb (kg) as VSS]/[lb (kg) of BOD_L removed]}
k_d = endogenous decay rate, day^{-1}
θ_c = mean cell residence time, days

From Eqs. (2.30) and (2.31), it is clear that by increasing the mean cell residence time of the activated-sludge system while keeping all other parameters constant, the mass of sludge produced from secondary treatment P_x decreases. However, from a practical standpoint, increasing the mean cell residence time results in the accumulation of sludge within the activated-sludge system. If θ_c is not managed properly, the sludge storage capacity of the activated sludge system eventually will be exceeded, and sludge will overflow from the secondary clarification system into the clarified effluent, reducing the overall system effectiveness. Therefore, effective activated-sludge system operation requires the periodic removal (i.e., wasting) of sludge from the system. Example 2.10 illustrates the basic approach for estimating the daily sludge production rate for an activated-sludge system.

Example 2.10 The Poole County Wastewater Treatment Facility is planning to build an oxidation ditch activated-sludge treatment facility to process 3 million gallons per day of domestic wastewater. What mass (dry basis) and volume of sludge would be produced at this facility assuming that the following data apply:

1. Influent BOD_5: 220 mg/liter
2. BOD_5 removal efficiency: 95 percent
3. Maximum yield coefficient Y: 0.65 lb/lb (kg/kg)
4. Mean cell residence time: 4 days
5. Endogenous decay coefficient k_d: 0.08 day^{-1}
6. Moisture content of settled sludge: 98 percent
7. Specific gravity of settled sludge: 1.02
8. Factor for converting BOD_5 to BOD_L: 0.68

solution

Step 1. Calculate the observed yield coefficient Y_{obs} from the maximum yield coefficient Y using Eq. (2.31):

$$Y_{\text{obs}} = \frac{Y}{1 + k_d\theta_c} = \frac{0.65}{1 + (0.08\ \text{day}^{-1})\ (4\ \text{days})} \cong 0.5$$

Step 2. Estimate the daily dry mass of sludge produced using Eq. (2.28). Note that the effluent BOD_5 concentration is $0.05 \cdot S_0$ (i.e., 95 percent BOD_5 removal efficiency) and that the BOD_5 concentration must be converted to ultimate BOD_L in order to apply the design equations.

$$P_x\left(\frac{\text{lb}}{\text{day}}\right) = \frac{Y_{\text{obs}}Q\ (S_0 - S)}{f}\left(\frac{8.34\ \text{lb}}{\text{MG}\cdot\text{mg/liter}}\right)$$

$$= \frac{0.5 \cdot 3\ \text{MGD}\ [220\ \text{mg/liter} - 0.05\ (220\ \text{mg/liter})]}{0.68} \cdot \left(\frac{8.34\ \text{lb}}{\text{MG}\cdot\text{mg/liter}}\right)$$

$$= 3845\ \text{lb/day}$$

Step 3. Estimate the daily volume of sludge to be managed at the wastewater treatment facility using Eq. (2.20):

$$\text{Volume produced per day}\left(\frac{\text{ft}^3}{\text{day}}\right)$$

$$= \frac{W_S}{\rho_W S_{BS} P_S} = \frac{3845 \text{ lb/day}}{(62.4 \text{ lb/ft}^3)\cdot 1.02 \cdot 0.02} = 3020.5 \text{ ft}^3\text{/day}$$

The excess sludge produced each day must be wasted to maintain the desired mean cell residence time (or equivalently, a desired *F/M* ratio). The most common practice is to waste sludge from the return activated sludge (RAS) line because the sludge is more concentrated at this location, and therefore, less pump capacity is needed. The wasted sludge normally is transferred to thickening tanks, for further processing [28].

An alternative approach is to waste sludge from the aeration basin where the concentration of sludge is more uniform (Fig. 2.43). Again, the waste mixed liquor suspended solids should be discharged to a thickening tank. However, in some cases the sludge discharged from the aeration basin is returned to the primary clarifier [12,32]. Although not uncommon, this procedure leads to difficulty in maintaining a constant mean cell residence time and therefore is not a recommended practice [12].

Although the required mass of sludge to be removed is fixed when θ_c is chosen, the actual volume of sludge that must be pumped to achieve process control depends on the location at which sludge wasting occurs. For example, if sludge wasting is from the RAS line (see Fig. 2.43*a*), the mean cell residence time θ_c is defined by Eq. (2.32):

$$\theta_c = \frac{VX}{Q'_w X_r + Q_e X_e} \tag{2.32}$$

where θ_c = mean cell residence time, days
V = aeration basin volume, MG (m^3)
X = concentration of microorganisms in aeration tank, mg VSS/liter
Q'_w = volumetric flow rate of waste sludge from recycle line, MGD (m^3/day)
X_r = concentration of microorganisms in recycle line, mg VSS/liter
Q_e = treated effluent flow rate from secondary clarifier, MGD (m^3/day)
X_e = concentration of microorganisms in clarified effluent, mg VSS/liter

If the mass of suspended solids in the clarified effluent (i.e., X_e) can be assumed to be negligible, the required volumetric wastage rate Q'_w may then be estimated using Eq. (2.33):

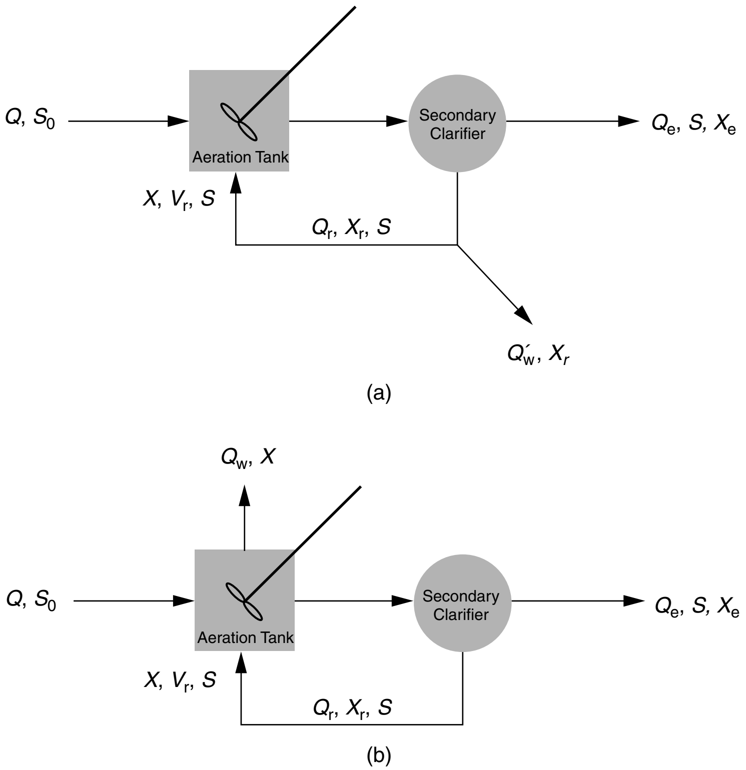


Figure 2.43 (*a*) Schematic diagram of sludge wasting from a RAS line. (*b*) Schematic diagram of sludge wasted from the aeration tank.

$$\theta_c = \frac{VX}{Q'_w X_r} \quad (\text{for } X_e \cong 0) \qquad \text{or} \qquad Q'_w = \frac{VX}{\theta_c X_r} \tag{2.33}$$

Conversely, if sludge wasting is from the aeration tank (see Fig. 2.43*b*), the mean cell residence time θ_c is defined by Eq. (2.34):

$$\theta_c = \frac{VX}{Q_w X + Q_e X_e} \tag{2.34}$$

where θ_c = mean cell residence time, days
V = aeration basin volume, MG (m^3)
X = concentration of microorganisms in aeration basin, mg VSS/liter
Q_w = volumetric flow rate of waste sludge from aeration basin, MGD (m^3/day)
Q_e = treated effluent flow rate from secondary clarifier, MGD (m^3/day)

X_e = concentration of microorganisms in clarified effluent, mg VSS/liter

Again, if the mass of suspended solids in the clarified effluent (i.e., X_e) can be assumed to be negligible, the volumetric wastage rate Q_w may then be estimated using Eq. (2.35):

$$\theta_c = \frac{VX}{Q_w X} \quad (\text{for } X_e \cong 0) \qquad \text{or} \qquad Q_w = \frac{V}{\theta_c} \tag{2.35}$$

It should be noted that if sludge is wasted from the aeration basin, knowledge of only the design mean cell residence time and aeration basin volume is necessary for process control. However, because of the relatively dilute mixed liquor suspended solids concentration, a significantly greater volume must be pumped to remove the necessary mass of sludge to maintain process control than if sludge were wasted from the recycle line. The difference in the required pumping capacities necessary for maintaining the design mean cell residence time when wasting sludge from different locations is illustrated in Example 2.11.

Example 2.11 The Kitale County Wastewater Treatment Plant is currently operating an oxidation ditch activated-sludge system at a mean cell residence time of 20 days. Given the following conditions, estimate the sludge pumping rate if sludge is (1) wasted from the secondary clarifier recycle line and (2) wasted from the aeration basin.

1. Aeration basin mixed liquor volatile suspended solids: 3200 mg/liter
2. Recycle flow mixed liquor volatile suspended solids: 12,400 mg/liter
3. Aeration basin volume: $1 \cdot 10^6$ gal
4. Concentration of solids from effluent of settling tank: 0.0 mg/liter

solution

Step 1. Estimate the sludge pumping rate if sludge is wasted from the recycle line using Eq. (2.33):

$$\theta_c = \frac{V_r X}{Q_w' X_r}$$

or

$$Q_w' = \frac{V_r X}{\theta_c X_r} = \frac{(1 \cdot 10^6 \text{ gal})\ (3200 \text{ mg/liter})}{(20 \text{ days})\ (12{,}400 \text{ mg/liter})}$$

$$= 12{,}903.2 \text{ gal/day } (9.0 \text{ gal/min})$$

Step 2. Estimate the sludge pumping rate if sludge is wasted directly from the aeration tank using Eq. (2.35):

$$Q_w = \frac{V_r}{\theta_c}$$

$$= 1 \cdot 10^6 \text{ gal/20 days} = 50{,}000 \text{ gal/day (34.7 gal/min)}$$

NOTE: To maintain the desired mean cell residence time, the sludge pumping rate from the aeration basin is approximately four times that required if sludge is wasted from the recycle line.

2.5.1.4 Return activated sludge (RAS). The objective of returning a portion of the concentrated activated sludge from the secondary clarifier underflow to the aeration basin is to maintain a sufficient concentration of microorganisms in the aeration basin for effective wastewater treatment as well as to control the sludge blanket depth in the secondary clarifier. Return activated sludge (RAS) pumping capacities of 50 to 100 percent of the influent wastewater flow rate are common in large plants, while for smaller plants (flow rates less than 3 MGD) RAS flow rates as large as 150 percent of the influent wastewater flow rate have been reported [12,28]. Several techniques are available for calculating the required RAS flow rate, including (1) settleability tests, (2) sludge blanket level control, and (3) secondary clarifier mass balance.

In settleability tests, a 1-liter sample of mixed liquor suspended solids from the aeration basin is allowed to settle for 30 minutes in a graduated cylinder or settleometer (Fig. 2.44). The RAS pumping rate is then estimated by multiplying the volume fraction occupied by the settleable solids by the volume flow rate of clarified effluent from the secondary clarification system. By setting the RAS pumping rate at this level, the depth of the sludge blanket in the secondary clarifier will remain constant. Example 2.12 illustrates the process of using a settleability test in establishing the RAS flow rate.

Example 2.12 To improve the effluent water quality from the secondary clarifier, a wastewater treatment plant operator desires to adjust the return activated sludge (RAS) flow rate for an oxidation ditch activated sludge system using the results from a 1-liter settleometer test. After a 1-liter sample of mixed liquor from the aeration tank is allowed to settle for 30 minutes, the volume occupied by the settled solids is approximately 250 ml. Given the volume of settled solids, estimate the RAS flow rate if the average plant flow rate is 3 million gal/day (MG/day) and the sludge volumetric wastage rate from the recycle line is maintained at 22,000 gal/day.

solution

Step 1. Estimate the percent volume occupied by the settled sludge:

$$\text{Percent volume (decimal)} = \frac{\text{sludge volume (ml)}}{\text{clarified water volume (ml)}}$$

$$= \frac{250 \text{ ml}}{750 \text{ ml}} = 0.33$$

Step 2. Estimate the desired RAS flow rate if the overall plant flow is 3 million gal/day.

$$\text{RAS flow rate (MG/day)} = \text{percent volume (decimal)} \cdot \text{plant flow rate (MG/day)}$$

$$= 0.33 \cdot 3 \text{ MG/day}$$

$$= 1.0 \text{ MG/day}$$

NOTE: The sludge volumetric pumping rate does not directly enter into the calculation for the RAS flow rate when using settlometer data.

Another settleability test occasionally used to estimate the RAS flow rate is the *sludge volume index* (SVI). The SVI is defined as the percentage volume occupied by the sludge in a 1-liter sample of mixed-liquor suspended solids after settling in a graduated cylinder for 30 minutes divided by the suspended solids concentration of the mixed liquor (expressed as a percentage P_w). Given this definition, the percentage of return activated sludge required to maintain a desired mixed-liquor suspended solids concentration in the aeration basin is given by Eq. (2.36). Care must be taken in extrapolating laboratory and/or pilot scale SVI data to establish the RAS flow rate for

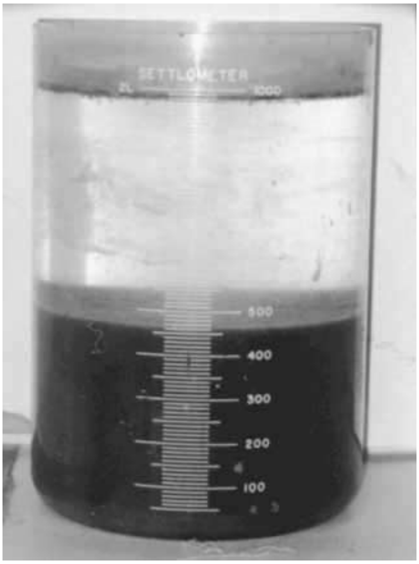


Figure 2.44 A settleometer used to estimate RAS flow rate.

full-scale systems because the settling characteristics of the mixed-liquor suspended solids will change depending on the influent wastewater characteristics [32,63].

$$\frac{Q_r}{Q} = \frac{1}{[100/(P_w \cdot \text{SVI})\,] - 1} \tag{2.36}$$

where Q_r = RAS flow rate, MGD (m^3/day)
Q = influent flow rate to activated sludge system, MGD (m^3/day)
P_w = suspended solids concentration in aeration tank (expressed as a percentage)
SVI = sludge volume index

In the sludge blanket control method for establishing the return activated sludge flow rate, the wastewater treatment operator must determine by experience both the optimal blanket depth (normally 2 to 4 ft) and the RAS flow rate to maintain that particular blanket depth. This approach to establishing the RAS flow rate requires considerable operator attention because of the variation in influent flow and sludge settling characteristics. Several methods are presently available for detecting the sludge blanket depth, including (1) core samplers, (2) air-lift pumps, (3) gravity-flow tubes, and (4) sludge-supernatant interface detectors [12].

Performing a mass balance around the secondary clarifier also may be used to estimate the required RAS flow rate (Q_r). It should be noted that Eq. (2.37) assumes steady-state conditions in which no accumulation of sludge in the secondary clarifier is assumed. A mass-balance procedure for estimating the RAS flow rate Q_r is illustrated schematically in Fig. 2.45 and described mathematically by Eq. (2.37).

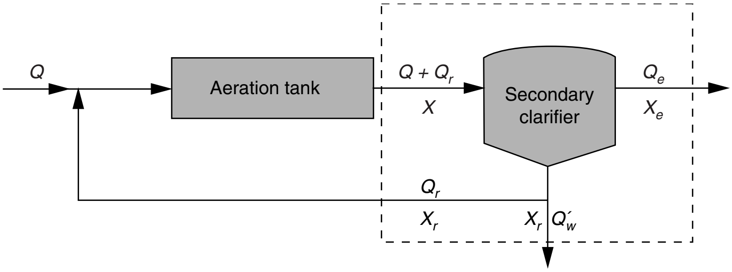


Figure 2.45 Mass balance of sludge around the secondary clarification system.

Solid accumulation within secondary clarifier

$$= \text{solids in} - \text{solids out}$$

$$0 = X\,(Q + Q_r)\,8.34 - X_r Q_r \cdot 8.34 - X_r Q'_w \cdot 8.34$$

or, solving for Q_r,

$$Q_r = \frac{XQ - X_r Q'_w}{X_r - X} \qquad (2.37)$$

where Q_r = RAS flow rate, MGD
X = mixed-liquor suspended solids concentration, mg/liter
X_r = RAS mixed-liquor suspended solids concentration, mg/liter
Q = influent flow, MGD
Q'_w = sludge wastage rate, MGD
8.34 = conversion factor [8.34 lb/MG · (mg/liter)]
MG = million gallons

Unfortunately, the RAS flow rate Q_r is sometimes estimated erroneously by performing a sludge mass balance around the aeration basin (Fig. 2.46). The results of this mass balance are presented in Eq. (2.38). It will be noted that Eqs. (2.37) and (2.38) produce the same RAS flow rate only when the sludge wastage rate Q'_w is set equal to zero (which is unrealistic).

Solids accumulation within the aeration tank

$$= \text{solids in} - \text{solids out}$$

$$0 = X_r Q_r \cdot 8.34 - X\,(Q + Q_r) \cdot 8.34$$

or, solving for Q_r,

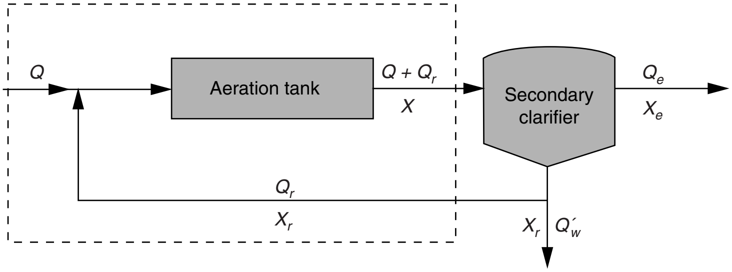


Figure 2.46 Mass balance of sludge around the aeration basin.

$$Q_r = Q \frac{X}{X_r - X} \tag{2.38}$$

NOTE: This procedure should *not* be used to estimate the RAS flow rate.

The limitation in using a mass balance around the aeration basin to establish the RAS flow rate stems from the fact that this approach typically neglects the sludge that is wasted from the activated-sludge system. Under steady-state conditions, the mass of sludge that is transferred from the aeration basin to the secondary clarification system must equal the sum of the sludge contained in the RAS flow Q_r and that which was wasted Q'_w. Example 2.13 illustrates the procedure for estimating the RAS flow rate.

Example 2.13 The Eldoret County Water Reclamation Facility is currently treating 1.5 million gal/day of municipal wastewater in a conventional plug flow activated-sludge system. If the mixed liquor volatile suspended solids (MLVSS) concentrations in the aeration basin and return recycle line are maintained at 2800 and 14,000 mg/liter, respectively, estimate the return activated-sludge (RAS) flow rate. Assume that the sludge wastage rate from the return recycle line is held constant at 42,000 gal/day.

solution Estimate the RAS flow rate using the solids mass balance [Eq. (2.37)]:

$$Q_r = \frac{XQ - X_r Q_w'}{X_r - X}$$

$$= \frac{(2800 \text{ mg/liter})\,(1.5 \cdot 10^6 \text{ gal/day}) - (14{,}000 \text{ mg/liter}) \cdot (42{,}000 \text{ gal/day})}{14{,}000 \text{ mg/liter} - 2800 \text{ mg/liter}}$$

$$= 322{,}500 \text{ gal/day}$$

Finally, Table 2.25 summarizes the design parameters for each of the major activated-sludge process configurations found at municipal wastewater treatment facilities. It is important to note that the ranges in parameter values provided in Table 2.25 are only recommendations. Actual activated-sludge system parameter values to be used in full-scale design should be obtained from results of laboratory and/or pilot-scale studies using actual wastewater from the municipal sewage collection system.

2.5.1.5 Oxygen requirements. Since the activated-sludge system is an aerobic wastewater treatment process, sufficient quantities of oxygen must be transferred to the aeration basin to microbially oxidize the influent BOD_L while continuously maintaining a residual dissolved

TABLE 2.25 Design Parameters for the Various Activated-Sludge Processes*

Process	Mean cell residence time θ_c (days)	*F*/*M* ratio (lb BOD_5 per lb MLVSS·day)	Volumetric loading rate (lb BOD_5 per 1000 ft^3·day)	Mixed-liquor suspended solids (mg/liter)	Hydraulic retention time *V*/*Q* (hours)	Recycle ratio Q_r/Q (percent)
Conventional plug flow	5–15	0.2–0.4	20–40	1500–3000	4–8	0.25–0.75
Oxidation ditch	10–30	0.05–0.3	5–30	1500–5000	8–36	0.75–1.50
Complete mix	5–15	0.20–0.6	50–120	2500–6500	3–5	0.25–1.0
Step feed	5–15	0.2–0.4	40–60	1500–3500	3–5	0.25–0.75
Contact stabilization	5–15	0.2–0.6	60–75	1000–3000† 4000–9000‡	0.5–1.0‡ 3.0–6.0§	0.5–1.5
Extended aeration	20–30	0.05–0.15	10–25	1500–5000	2–4	1.0–5.0
Pure oxygen	3–10	0.25–1.0	100–200	3000–8000	1–3	0.25–0.5

*Adapted from refs. [12,28].
†Mixed-liquor volatile suspended solids (mg/liter).
‡Contact basin.
§Stabilization basin.

oxygen level of at least 2 mg/liter. It should be noted that maintaining a dissolved oxygen level above 4.0 mg/liter does not improve the activated-sludge process significantly [12,28].

When the dissolved oxygen is reduced below 2 mg/liter, filamentous microbes (e.g., fungi) begin to predominate in the aeration basin [28]. The presence of excessive amounts of filamentous microbes results in poor settling characteristics of the mixed liquor within the secondary clarification system. The poor settling behavior of the mixed liquor will result in unacceptably high concentrations of sludge being discharged over the secondary clarifier weir.

The theoretical daily oxygen requirements for an activated-sludge system may be estimated from the ultimate biochemical oxygen demand of the influent waste stream and the mass of sludge generated each day. The mass of oxygen that must be supplied to the activated-sludge system is equal to the total amount of $\mathrm{BOD_L}$ that is removed from the system minus the oxygen equivalent of the microorganisms produced. The premise behind this theory is the fact that only that portion of the influent $\mathrm{BOD_L}$ which is used for energy by the microorganisms consumes oxygen. The portion of the influent BOD used in the synthesis of new cells does not exhibit an oxygen demand. Given this approach to estimating the microbial oxygen demand, the oxygen requirement for an activated-sludge system may be estimated using Eq. (2.39):

$$\begin{aligned} \frac{\text{lb O}_2}{\text{day}} &= Q\,(S_0 - S)\,8.34 - 1.42P_x \qquad \text{(U.S. units)} \\ \frac{\text{kg O}_2}{\text{day}} &= Q\,(S_0 - S)\,\frac{\text{kg}}{10^3\ \text{g}} - 1.42P_x \qquad \text{(SI units)} \end{aligned} \tag{2.39}$$

where Q = flow rate (MGD or $\mathrm{m^3/day}$)
S_0 = influent $\mathrm{BOD_L}$ (mg/liter or $\mathrm{g/m^3}$) to secondary treatment system
S = effluent $\mathrm{BOD_L}$ (mg/liter or $\mathrm{g/m^3}$) from secondary treatment system
1.42 = factor for converting VSS to $\mathrm{BOD_L}$ [1.42 lb (kg) $\mathrm{BOD_L}$/lb (kg) VSS]
P_x = sludge production rate—dry-mass basis (lb/day or kg/day)
8.34 = conversion factor [8.34 lb/MG · (mg/liter)]
MG = million gallons

When significant nitrification is occurring within the aeration basin, the total oxygen requirements should be computed as the sum of the oxygen requirements for the oxidation of organic matter plus the oxygen requirements for ammonia conversion to nitrate [Eq. (2.40)]:

$$\frac{\text{lb O}_2}{\text{day}} = Q\,(S_0 - S)\,8.34 - 1.42P_x + 4.57Q\,(N_0 - N)\,8.34 \quad \text{(U.S. units)} \tag{2.40}$$

$$\frac{\text{kg O}_2}{\text{day}} = Q\,(S_0 - S)\,\frac{\text{kg}}{10^3\ \text{g}} - 1.42P_x + 4.57Q\,(N_0 - N)\,\frac{\text{kg}}{10^3\ \text{g}} \quad \text{(SI units)}$$

where N_0 = influent TKN—total Kjeldahl nitrogen (mg/liter or g/m³)
N = effluent TKN—total Kjeldahl nitrogen (mg/liter or g/m³)
4.57 = factor for converting TKN to BOD_L [4.57 lb (kg) BOD_L/lb (kg) TKN]

NOTE: All other terms are as previously defined.

Example 2.14 illustrates the use of Eq. (2.39) in estimating the oxygen transfer and power requirements for an activated-sludge aeration system.

Example 2.14 The Siaya County Sewer Improvement District is currently treating 5 million gal/day of municipal wastewater in a conventional plug flow activated-sludge process equipped with a diffused-air aeration system. Given the following conditions, estimate the minimum pounds of oxygen that must be supplied by the aeration system each day to treat the organic loading. Assume that sludge wasting occurs from the recycle line and that the suspended solids concentration discharged from the secondary clarifier effluent is negligible.

1. Influent BOD_L: 210 mg/liter
2. Effluent BOD_L: 20 mg/liter
3. Aeration basin volume: $2{\cdot}10^6$ gal
4. Mean cell residence time: 15 days
5. MLVSS (aeration basin): 3500 mg/liter
6. MLVSS (sludge recycle): 12,800 mg/liter

solution

Step 1. Estimate the sludge wastage rate Q_w', using Eq. (2.33):

$$Q_w' = \frac{V_r X}{\theta_c X_r} = \frac{(2 \cdot 10^6\ \text{gal})\,(3500\ \text{mg/liter})}{(15\ \text{days})\,(12{,}800\ \text{mg/liter})}$$

$$= 36{,}458\ \text{gal/day}\ (0.036458 \cdot 10^6\ \text{gal/day})$$

Step 2. Estimate the pounds of volatile suspended solids generated per day:

$$P_x\ (\text{lb/day}) = Q_w{}'X_r \cdot \frac{8.34\ \text{lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= 0.036458\ \text{MG/day} \cdot 12{,}800\ \text{mg/liter} \cdot \frac{8.34\ \text{lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= 3892\ \text{lb VSS/day}$$

Step 3. Estimate the pounds of oxygen required using Eq. (2.39):

$$\text{lb O}_2\text{/day} = Q(S_o - S) \cdot \frac{8.34\ \text{lb}}{\text{MG} \cdot (\text{mg/liter})} - \frac{1.42\ \text{lb O}_2}{\text{lb VSS}}\ P_x$$

$$= 5 \cdot (210 - 20) \cdot \frac{8.34\ \text{lb}}{\text{MG} \cdot (\text{mg/liter})} - \frac{1.42\ \text{lb O}_2}{\text{lb VSS}} \cdot \frac{3892\ \text{lb VSS}}{\text{day}}$$

$$= 2396.4\ \text{lb O}_2\text{/day}$$

The two basic methods to achieve oxygen transfer in activated-sludge aeration basins include (1) the use of diffusers to introduce air or pure oxygen or (2) the use of mechanical agitation to promote dissolution of atmospheric oxygen. Regardless of the method employed to supply oxygen to the aeration basin, the oxygen transfer system also must achieve sufficient mixing as well.

Diffuser systems may be categorized as (1) porous or fine-pore diffusers, (2) nonporous diffusers, or (3) other diffusion devices (e.g., jet aerators, aspirating aerators, and U-tube aerators). Various types of diffused air devices are shown schematically in Fig. 2.47.

The efficiency of a diffuser to transfer oxygen to the wastewater depends on many factors, including (1) airflow rate, (2) depth of submersion of the diffuser, (3) aeration tank geometry, and (4) wastewater characteristics [60]. All diffuser systems normally are evaluated in tap water, and the results of the oxygen transfer tests are then adjusted to process operating conditions. Typically, manufacturers report the standard oxygen transfer efficiencies (SOTEs) for diffusers in clean water at a submergence depth of 15 ft (Table 2.26).

The energy required to overcome the hydrostatic pressure and the frictional losses associated with the transfer of air (or oxygen) into the aeration basin by diffuser systems must be supplied by blowers. Blowers act by compressing the inlet gas, resulting in a significant increase in outlet gas pressure. Two types of blowers are used commonly in diffuser operation: (1) rotary-lobe positive-displacement blowers and (2) centrifugal blowers (Fig. 2.48).

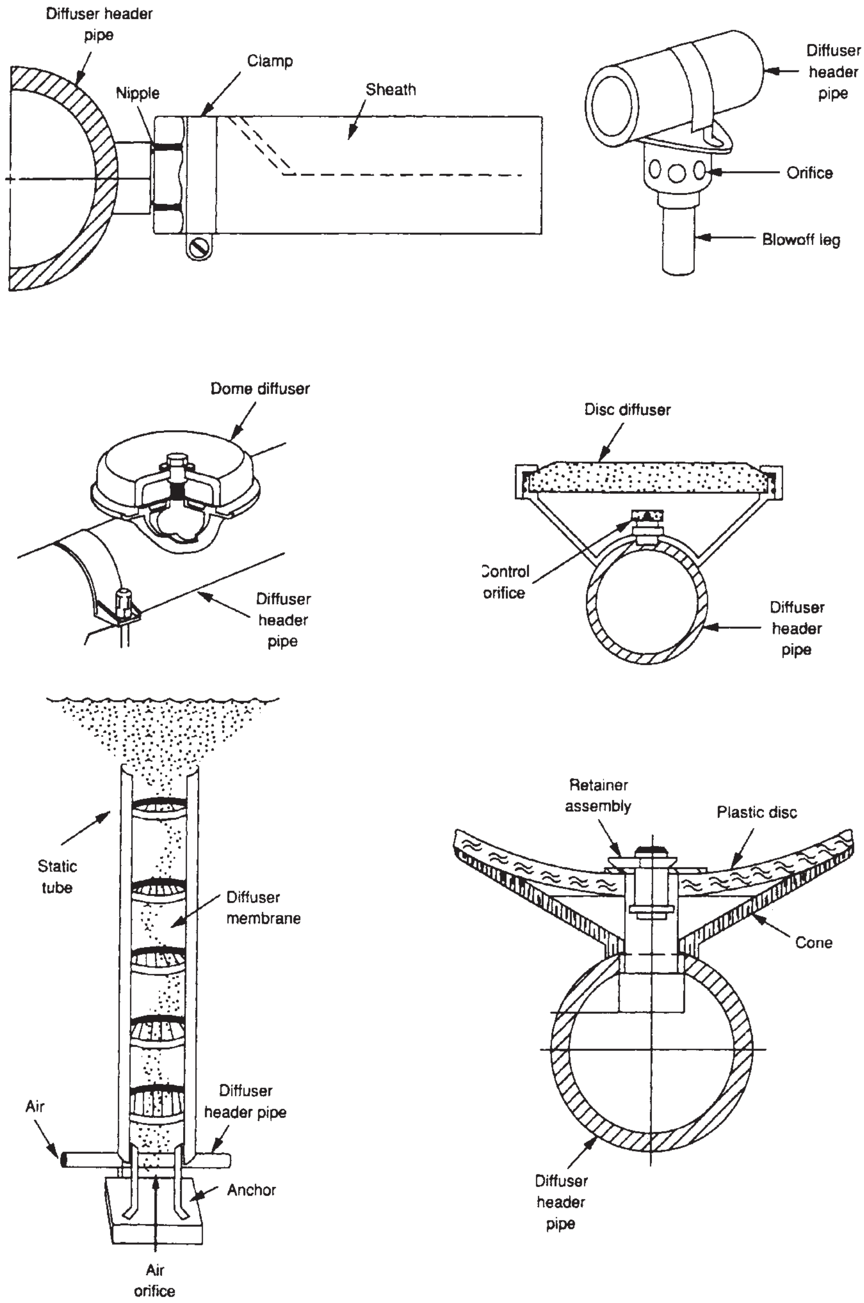


Figure 2.47 Schematic diagrams of diffusers used in the activated-sludge process.

TABLE 2.26 Clean Water Oxygen Transfer Efficiencies for Diffuser Systems*

Diffuser type	Airflow rate (ft^3/min per diffuser)	Standard oxygen transfer efficiency (%) at 15 ft submergence†
Ceramic disks—grid	0.4–3.4	25–40
Ceramic domes—grid	0.5–2.5	27–39
Ceramic plates—grid	2.0–5.0‡	26–33
Rigid porous plastic tubes		
Grid	2.4–4.0	28–32
Dual spiral roll	3.0–11.0	17–28
Single spiral roll	2.0–12.0	13–25
Nonrigid porous plastic tubes		
Grid	1.0–7.0	26–36
Single spiral roll	2.0–7.0	19–37
Perforated membrane tubes		
Grid	1.0–4.0	22–29
Quarter points	2.0–6.0	19–24
Single spiral roll	2.0–6.0	15–19
Jet aeration		
Side header	54.0–300	15–24
Nonporous diffuser		
Dual spiral roll	3.3–10.0	12–13
Midwidth	4.2–45.0	10–13
Single spiral roll	10.0–35.0	9–12

*Adapted from refs. [28,60].
†Standard conditions (68°F, 1 atm pressure, dissolved oxygen, 0.0 mg/liter).
‡Units are ft^3 of air/ft^2 of diffuser plate area per minute.

Centrifugal blowers are used commonly when a gas flow rate of at least 3000 ft^3/min (85 m^3/min) is required [1,22,28]. Discharge pressures from centrifugal blowers normally range from 7 to 9 lb/in^2 gauge (i.e., 48–62 kN/m^2). Centrifugal blowers have operating characteristics similar to centrifugal pumps in that their most efficient operating point is obtained by the intersection of the head-capacity curve and the system curve [1,22]. Centrifugal blowers are rated in terms of airflow rate delivered (e.g., cubic feet per minute) at a given pressure and motor speed [22]. For cases where higher discharge pressures are required and/or smaller discharge flow rates than those achievable with centrifugal blowers, rotary-lobe positive-displacement blowers are used.

All blowers normally meet one particular set of operating conditions at their maximum efficiency. However, since it is normally necessary to meet a wide range of airflows during activated-sludge operations, the capability to regulate blower airflow rate and/or outlet pressures must be an integral component of the aeration system [1,22,60].

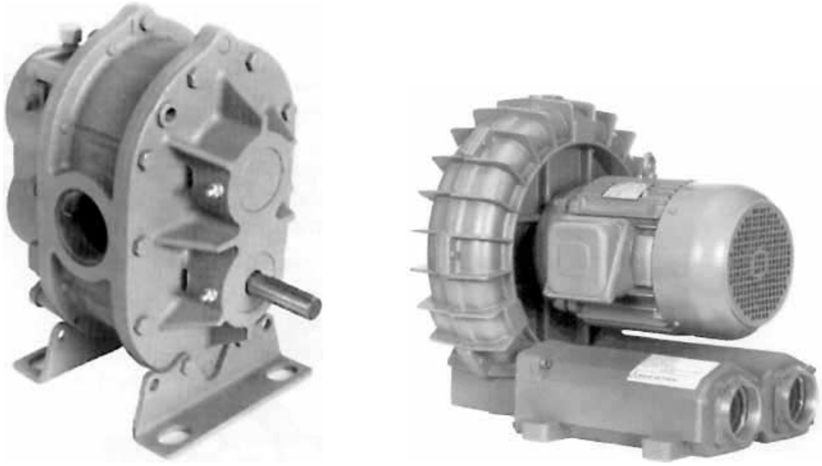

(a) (b)

Figure 2.48 (*a*) A rotary-lobe positive-displacement blower. (*Courtesy of Dresser Industries, Inc., Roots Division.*) (*b*) Centrifugal blower. (*Courtesy of AMETEK Rotron TMD.*)

Methods to achieve regulation of centrifugal blowers include inlet throttling and adjustable-discharge diffusers, whereas variable-speed drives are used to regulate positive-displacement blowers [60].

Blower manufacturers normally report the operating conditions of their systems at standard conditions, which are defined as 1 atm barometric pressure, an inlet temperature of 68°F (20°C), and a relative humidity of 36 percent [28]. At these conditions, standard air has a specific weight of 0.075 $\mathrm{lb_m/ft^3}$. However, to properly adjust the blower to meet aeration requirements at field conditions, it must be recognized that any variation in inlet temperature or barometric pressure from standard conditions will change the density of the outlet compressed gas. A change in gas density, in turn, will affect both the outlet gas pressure and blower power requirements. The power requirement for blower systems may be estimated by assuming adiabatic compression of the gas [Eq. (2.41)]. Example 2.15 illustrates the use of Eq. (2.41) in estimating the power requirement of a blower system.

$$\text{Power (hp)} = \frac{wRT_1}{550ne}\left[\left(\frac{P_2}{P_1}\right)^{0.283} - 1\right] \qquad \text{(U.S. units)}$$

$$\text{Power (kW)} = \frac{wRT_1}{29.7ne}\left[\left(\frac{P_2}{P_1}\right)^{0.283} - 1\right] \qquad \text{(SI units)} \tag{2.41}$$

where hp = horsepower
kW = kilowatt
w = weight of airflow, lb_m/s (kg/s)
R = gas constant (53.3 ft · $\text{lb}_f/\text{lb}_{\text{air}}$ · °R)
550 = factor for converting hp to ft · lb_f/s (550 ft · lb_f/s · hp)
29.7 = conversion to SI units
T_1 = absolute inlet temperature, Rankine (Kelvin)
P_1 = inlet pressure (absolute), psia (atm)
P_2 = outlet pressure (absolute), psia (atm)
e = compressor efficiency (0.7–0.9)
n = $[1-(1/\gamma)] = 0.283$ for air
γ = ratio of specific heat of gas at constant pressure to specific heat of gas at constant volume (C_p/C_v), for air—1.395

Example 2.15 For the Siaya County Sewer Improvement District (see Example 2.14), estimate the 40-hp minimum compressor horsepower necessary to meet the oxygen demand if the average compressor efficiency is 80 percent. Assume that the following conditions apply:

1. Compressor inlet air temperature: 70°F (530°R)
2. Compressor inlet air pressure: 14.7 psia
3. Compressor outlet air pressure: 140.0 psia
4. Weight fraction of oxygen in air: 0.232
5. n (constant for air): 0.283
6. Gas constant (R): 53.3 ft·$\text{lb}_f/\text{lb}_{\text{air}}$·°R

solution

Step 1. Estimate the airflow rate in pounds per second:

$$\text{Airflow rate (lb air/s)} = \text{oxygen flow rate (lb/day) (lb air/0.232) (day/86,400 s)}$$

$$= (2396.4 \text{ lb } O_2\text{/day}) \text{ (lb air/0.232) (day/86,400 s)}$$

$$= 1.196 \text{ lb } O_2\text{/s}$$

Step 2. Using Eq. (2.41), estimate the compressor horsepower needed to supply the oxygen demand.

$$\text{Power (hp)} = \frac{wRT_1}{550ne}\left[\left(\frac{P_2}{P_1}\right)^{0.283} - 1\right]$$

$$= \frac{1.196 \text{ lb } O_2\text{/s } (53.3)\ (530°\text{R})}{(550)\ (0.283)\ (0.8)}\left[\left(\frac{140}{14.7}\right)^{0.283} - 1\right]$$

$$= 24.2 \text{ hp}$$

> Since a minimum of 24.2 hp is required by the compressor to meet the wastewater's oxygen demand, the use of a 30- to 40-hp compressor (of 80 percent mechanical efficiency) should be recommended. Normally, two compressor units would be installed with one maintained in standby mode.

Mechanical aerators are divided into two groups: (1) aerators with a vertical axis and (2) aerators with a horizontal axis. Both groups are further divided into surface and submerged aerators [22,28]. In surface aerators, oxygen is entrained from the atmosphere, whereas in submerged aerators, oxygen is entrained from the atmosphere or from an air or pure oxygen flow introduced into the bottom of the aeration basin [28]. Typical examples of mechanical aerators/mixers used in activated-sludge systems are depicted in Fig. 2.49.

Mechanical aerators are rated in terms of their oxygen transfer rate, expressed as pounds of oxygen transferred per horsepower per hour (i.e., kilograms of oxygen per kilowatthour) at standard conditions

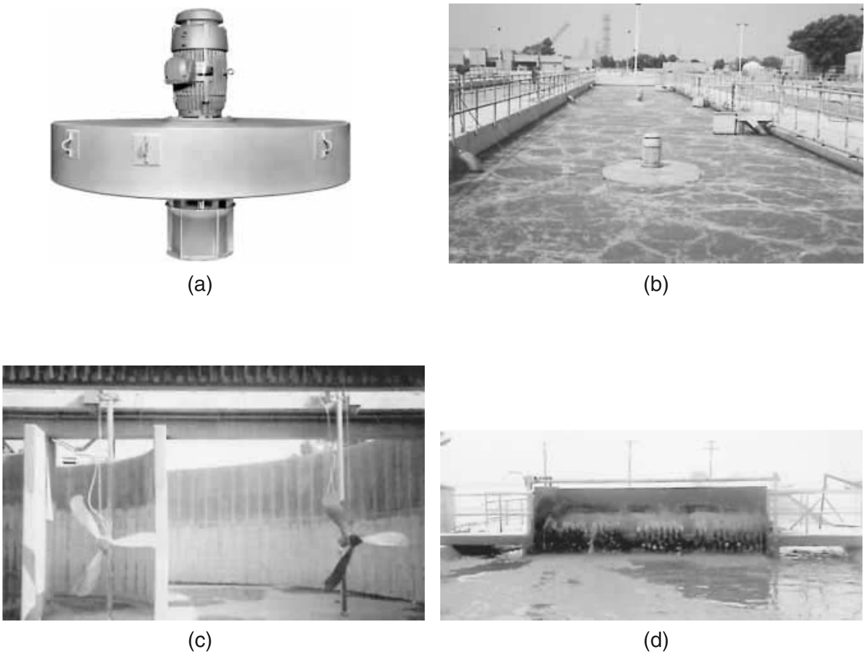


Figure 2.49 (*a*) Floating mechanical surface aerator. (*Courtesy of Aerators, Inc.*) (*b*) Application of floating mechanical surface aerator system in activated-sludge aeration basin. (*Courtesy of Aerators, Inc.*) (*c*) Submersible mixer for oxidation ditch activated-sludge systems. (*Courtesy of I. Kruger, Inc.*) (*d*) Mechanical brush aerators used in oxidation ditch activated-sludge systems.

TABLE 2.27 Oxygen Transfer Rates for Mechanical Aerators*

Aerator type	Transfer rate (lb O_2/hp · h) Standard†	Field‡
Surface, low speed	2.0–5.0	1.2–2.4
Surface, low speed with draft tube	2.0–4.6	1.2–2.1
Surface, high speed	2.0–3.6	1.2–2.0
Surface, down-draft turbine	2.0–4.0	1.0–2.0
Submerged, turbine with sparger	2.0–3.3	1.2–1.8
Submerged, impeller	2.0–4.0	1.2–1.8
Surface, brush and blade	1.5–3.6	0.8–1.8

*Adapted from refs. [28,60].
†Standard conditions (68°F; 1 atm pressure; dissolved oxygen, 0.0 mg/liter).
‡Wastewater conditions (57°F; 1 atm pressure; dissolved oxygen 2.0 mg/liter; altitude 500 ft; $\alpha = 0.85$; $\beta = 0.9$).

[defined as tap water at 68°F (20°C) with an initial dissolved oxygen concentration of 0.0 mg/liter]. Oxygen transfer data for various types of mechanical aerators are reported in Table 2.27.

For design purposes, the standard performance data provided by the mechanical aerator manufacturer must be adjusted to reflect the anticipated field conditions using Eq. (2.42):

$$N = N_0 \left(\frac{\beta C_{w_{\text{alt}}} - C_L}{C_{s_{20}}} \right) 1.024^{T-20} \alpha \tag{2.42}$$

where N = lb O_2/hp · h transferred at field conditions
N_0 = lb O_2/hp · h transferred at standard conditions
β = salinity-surface tension correction factor (usually 1)
$C_{w_{\text{alt}}}$ = oxygen saturation concentration for tap water at given temperature and altitude
$C_{s_{20}}$ = oxygen saturation concentration in tap water at 20°C, mg/liter (9.0 mg/liter at 1 atm pressure)
C_L = operation oxygen concentration, mg/liter (typically 2.0–4.0 mg/liter)
T = temperature, °C
α = oxygen transfer correction factor (typical values: influent wastewater—0.82; effluent wastewater—0.98)

Use of Eq. (2.42) requires that the oxygen saturation for tap water at a given altitude $C_{w_{\text{alt}}}$ be estimated. The value of this parameter may be obtained by multiplying the dissolved oxygen saturation concentration at the same temperature and zero altitude (i.e., sea level) by a dissolved oxygen altitude correction factor F_a. The dissolved oxygen

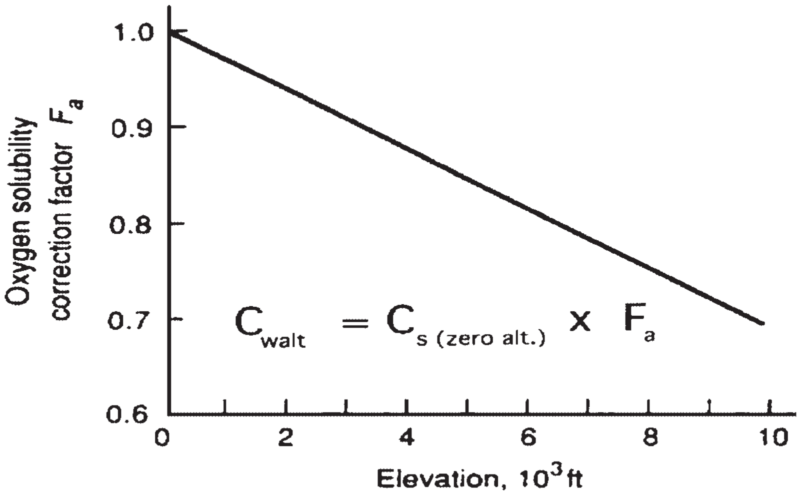


Figure 2.50 Variation of dissolved oxygen saturation as a function of altitude.

altitude correction factor may be estimated using Fig. 2.50. Example 2.16 illustrates the use of Eq. (2.42) in the design of a mechanical aerator system.

Example 2.16 The Naivasha County Water Reclamation Plant is currently treating 3.8 million gal/day of municipal wastewater in a completely mixed activated-sludge process. If the plant manager desires to maintain a dissolved oxygen concentration of at least 2.5 mg/liter in the mixed liquor, estimate the number of 50-hp surface aerators that must be installed in the aeration tank. Assume that the following conditions apply:

1. Average wastewater temperature: 15°C
2. Biochemical oxygen demand (influent): 220 mg/liter BOD_5
3. Biochemical oxygen demand (effluent): 15 mg/liter BOD_5
4. Volume of aeration basin: $2 \cdot 10^6$ gal
5. Mean cell residence time θ_c: 12 days
6. Mixed liquor volatile suspended solids (aeration basin): 3000 mg/liter
7. Volatile suspended solids concentration (sludge recycle): 13,000 mg/liter
8. Altitude of treatment plant above sea level: 4000 ft
9. Theoretical oxygen transfer rate of surface aerator (N_o): 1.7 lb O_2/hp·h (from manufacturer)
10. α: 0.86
11. β: 1.0
12. Conversion of BOD_5 to BOD_L (f): 0.68

solution

Step 1. Estimate the daily waste activated-sludge production rate Q_w':

$$Q_w' = \frac{V_r X}{\theta_c X_r} = \frac{(2 \cdot 10^6 \text{ gal})\,(3000 \text{ mg/liter})}{(12 \text{ days})\,(13{,}000 \text{ mg/liter})}$$

$$= 38{,}462 \text{ gal/day } (0.0385 \text{ MG/day})$$

Step 2. Estimate the mass of waste activated-sludge generated daily P_x:

$$P_x \text{ (lb VSS/day)} = Q_w' \text{ (MG/day)} \cdot X_r \cdot \frac{8.34 \text{ lb}}{(\text{mg/liter})}$$

$$= (0.0385 \text{ MG/day})\,(13{,}000 \text{ mg/liter})\,\frac{8.34 \text{ lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= 4170 \text{ lb VSS/day}$$

Step 3. Estimate the minimum pounds of oxygen needed to treat the organic loading using Eq. (2.39). Note that BOD_5 must be converted to BOD_L in the calculation.

$$\text{lb O}_2\text{/day} = Q\,\frac{S_o - S}{f} \cdot \frac{8.34 \text{ lb}}{\text{MG} \cdot (\text{mg/liter})} - \frac{1.42 \text{ lb O}_2}{\text{lb VSS}}\,P_x$$

$$= 3.8 \text{ MG/day}\,\frac{210 - 20}{0.68}\,\frac{8.34 \text{ lb}}{\text{MG} \cdot (\text{mg/liter})} - \frac{1.42 \text{ lb O}_2}{\text{lb VSS}}\,\frac{4170 \text{ lb VSS}}{\text{day}}$$

$$= \frac{3632.8 \text{ lb O}_2}{\text{day}}$$

Step 4. Estimate the aerator oxygen transfer rate N at field conditions using Eq. (2.42). Note that at 15°C, oxygen concentration in tap water is 10.02 mg/liter, while the saturation concentration for oxygen in water at 20°C is 9.0 mg/liter. From Fig. 2.50, the altitude correction factor for oxygen saturation in water at 4000 ft is 0.88; therefore, C_{w_alt} is estimated to be 8.82 mg/liter (i.e., 10.02 mg/liter·0.88).

$$N = N_o \left(\frac{\beta C_{w_\text{alt}} - C_L}{C_{s_{20}}} \right) 1.024^{T-20} \cdot \alpha$$

$$= \frac{1.7 \text{ lb O}_2}{\text{hp} \cdot \text{h}} \left[\frac{1 \cdot (8.82 \text{ mg/liter}) - 2.5 \text{ mg/liter}}{9.0 \text{ mg/liter}} \right] 1.024^{15\text{-}20} \cdot 0.86$$

$$= 0.92 \text{ lb O}_2/(\text{hp} \cdot \text{h})$$

Step 5. Estimate the overall power requirement to meet the oxygen demand:

$$\text{Power requirement (hp)} = \frac{\text{total oxygen demand (lb O}_2\text{/h)}}{\text{aerator oxygen transfer rate [lb O}_2\text{/ (hp} \cdot \text{h)]}}$$

$$= \frac{(3362.8 \text{ lb O}_2\text{/day) (day/24 h)}}{0.92 \text{ lb O}_2\text{/ (hp} \cdot \text{h)}}$$

$$= 164.5 \text{ hp}$$

Therefore, to meet the wastewater's oxygen demand, a minimum of four 50-hp aerators must be operating continuously in the activated-sludge system. In practice, at least five aerators should be installed with one maintained in standby mode.

2.5.1.6 Mixing requirements. In activated-sludge systems, adequate oxygen transfer and mixing must be achieved by the aeration system. Because maintenance of an adequate mean cell residence time establishes both the organic matter (i.e., BOD_L) removal and microbial growth rates in the aeration basin, the activated-sludge oxygen demand is fixed for a given θ_c. The oxygen demand, in turn, establishes the power requirement given the oxygen transfer efficiency of the aeration equipment (see Example 2.14).

In contrast to meeting the activated-sludge oxygen demand, to achieve adequate activated-sludge mixing requires a certain amount of power input per unit volume of aeration basin. At high mixed-liquor suspended solids concentrations (i.e., above 5000 mg/liter) in the aeration basin, the power needed for oxygen transfer is likely to be larger than that required for mixing. As the mixed-liquor suspended solids concentration is reduced, the power required for mixing will increase relative to the power required for oxygen transfer.

For most municipal wastewaters, when the mixed-liquor suspended solids concentration in the aeration basin is reduced to approximately 1500 mg/liter, the power for mixing is significantly larger than that required for oxygen transfer, so further reductions in the mixed-liquor suspended solids concentrations are not economically justifiable [29,66]. Conversely, as the mixed-liquor suspended solids concentration increases, the power per unit volume for oxygen transfer increases, thereby increasing the turbulence in the reactor. As the power input to the aeration basin is increased to meet the required oxygen demand, a power input level will be reached at which the fluid turbulence will damage the microbial floc, making clarification difficult. When air is used as the source of oxygen, this limitation on turbulence is generally reached at a mixed-liquor suspended solids concentration of approximately 6000 mg/liter [12].

Therefore, the upper limit on the mixed-liquor suspended solids concentration in the aeration basin is imposed by the need to maintain proper operation of the secondary clarification system. Activated-sludge systems employed for treating municipal wastewater normally will operate at mixed-liquor suspended solids concentrations in the range of 2500 to 5000 mg/liter.

To maintain proper aeration and mixing conditions without shearing the microbial flocs, most aeration basins will have mixing power inputs in the range of 0.75 to 1.5 hp per 1000 ft^3 of basin volume for mechanical aerator systems or 20 to 30 standard cubic feet per minute (scfm) of airflow per 1000 ft^3 of basin volume for diffused-air systems [12,28]. The actual power input or airflow rate employed will depend on the type of mechanical aerator or diffuser system used, basin size, and basin shape. Readers interested in additional information on aeration basin design are referred to the following references [12,22,24,60].

2.5.1.7 Nutrient requirements. For an activated-sludge system to operate properly, nutrients must be available in adequate amounts for cell synthesis. Once a θ_c is chosen for activated-sludge process control, the rate of cell synthesis is fixed. The measured or predicted value of the observed microbial yield coefficient Y_{obs} may then be used to estimate the nutrient requirements.

This approach to estimating the nutrient requirement is based on the average elemental composition of microbial cells (i.e., $C_5H_7O_2N$), in which nitrogen represents 12.4 percent of the cells' dry weight. Therefore, multiplication of the observed microbial yield coefficient by 0.124 gives the mass of nitrogen incorporated into the mass of cells per unit mass of BOD_L removed. Using this relationship, the minimum nitrogen requirement may then be estimated using Eq. (2.43):

$$N\left(\frac{\text{lb of nitrogen required}}{\text{lb of BOD removed}}\right) = \frac{0.124\ \text{lb nitrogen required}}{\text{lb VSS produced}}\,Y_{obs}$$

$$= \frac{0.124\,Y}{1 + k_d\theta_c} \qquad (2.43)$$

where Y_{obs} = observed yield {[lb (kg) biosolids generated as VSS]/[lb (kg) of BOD_L removed]}

Y = maximum yield {[maximum mass of biosolids generated, lb (kg), as VSS]/[lb (kg) of BOD_L removed]}

0.124 = factor for estimating nitrogen requirement [(0.124 lb nitrogen required]/[lb VSS produced)]

k_d = endogenous decay rate, day^{-1}

θ_c = mean cell residence time, days

The minimum phosphorus requirement for proper activated-sludge operation is approximately one-fifth the nitrogen requirement [12,28]. It should be noted that nutrients should be available in slight excess of the theoretical minimum to ensure proper activated-sludge operation.

Domestic wastewater normally will have sufficient nutrient levels for stable activated-sludge operation. However, if a slug organic load is discharged by an industrial source, the nutrient levels in the incoming wastewater should be evaluated, and supplemental nutrients should be added, if necessary. Since the required amount of nutrients will depend on the net mass of sludge produced, the design mean cell residence time will affect the nutrient requirements. Therefore, by controlling the mean cell residence time, the plant operator can change the system requirement for nutrients. Example 2.17 illustrates the use of Eq. (2.43) in estimating the minimum quantity of nutrients necessary for proper activated-sludge operation.

Example 2.17 The Otieno City Water Reclamation Plant is currently treating 2.0 million gal/day of municipal wastewater in an oxidation ditch activated-sludge system. An operator notices that the mixed-liquor suspended solids are settling poorly in the secondary clarifier and suspects that the cause is a nutrient deficiency resulting in bulking sludge. For the following operational conditions, estimate the minimum nitrogen loading to ensure that the activated-sludge process is not nutrient limited.

1. Biochemical oxygen demand (influent): 225 mg/liter BOD_5
2. Biochemical oxygen demand (effluent): 20 mg/liter BOD_5
3. Volume of aeration basin: $2 \cdot 10^6$ gal
4. Mean cell residence time θ_c: 15 days
5. Y: maximum yield coefficient, 0.4 lb VSS/lb BOD_L
6. k_d: decay coefficient 0.02 day^{-1}
7. Conversion of BOD_5 to BOD_L: 0.68

solution

Step 1. Estimate the pounds of BOD_L removed per day.

$$\text{lb BOD}_L/\text{day} = Q\,\frac{S_o - S}{f} \cdot \frac{8.34\ \text{lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= 2\ \text{MG/day}\ \frac{225 - 20}{0.68}\ \frac{8.34\ \text{lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= 5028.5\ \text{lb/day}$$

Step 2. Estimate the pounds of nitrogen needed per pound of $\mathrm{BOD_L}$ removed using Eq. (2.43):

$$\frac{\text{lb of nitrogen required}}{\text{lb of BOD removed}} = \frac{0.124Y}{1 + k_d\theta_c}$$

$$= \frac{0.124\,(0.4)}{1 + (0.02)\,(15)} = 0.038 \text{ lb N/lb } \mathrm{BOD_L} \text{ removed}$$

Step 3. Estimate the minimum daily nitrogen demand to ensure that the activated-sludge system is not nitrogen-limited.

$$\text{lb N/day} = \mathrm{BOD_L} \text{ removal rate (lb } \mathrm{BOD_L} \text{ removed/day)}$$

$$\cdot \text{ specific nitrogen demand (lb N/lb } \mathrm{BOD_L} \text{ removed)}$$

$$= 5028.5 \text{ lb } \mathrm{BOD_L} \text{ removed/day} \cdot 0.038 \text{ lb N/lb } \mathrm{BOD_L} \text{ removed}$$

$$= 191.9 \text{ lb N/day}$$

NOTE: In actual practice, the nitrogen loading to the plant should be at least twice the minimum to ensure that the microbial cells are not nutrient-limited.

2.5.1.8 Control of filamentous microbes. The growth of filamentous microorganisms is the most common operational problem encountered in the activated-sludge process. A proliferation of filamentous microbes in the aeration basin results in poorly settling sludge in the secondary clarification system. A phenomenon commonly termed in practice as *bulking sludge*. Operational causes of bulking sludge include low dissolved oxygen and insufficient nutrient levels [12,28]. It should be noted that the formation of pin floc also will result in poor settling in the secondary clarification system. Although it produces similar results, the causes of pin-floc formation are different from those causing bulking sludge [28,60]. The control of pin-floc formation is discussed in Sec. 2.5.1.10.

If there is limited dissolved oxygen in the aeration basin resulting in filamentous growth, the situation normally can be corrected by operating the aeration system at higher capacity. Under all anticipated loading conditions, the aeration system should have adequate capacity to maintain at least 2 mg/liter of dissolved oxygen in the aeration basin. If this level of dissolved oxygen cannot be maintained, additional aeration capacity must be installed.

The operating characteristics of the secondary clarification system also may contribute to the formation of bulking sludge. Bulking sludge is often reported to be a problem in center-feed circular secondary clarifier tanks. In this design, sludge normally is removed from the tank directly under the point where the mixed liquor enters the tank [28,63]. Where bulking sludge has occurred, examination of the sludge blanket typically will indicate that a large fraction of the settled sludge is retained in the tank for a much longer time than the desired 30 minutes. The long-term accumulation of sludge within the secondary clarifier results in the depletion of dissolved oxygen through endogenous respiration. The depletion of dissolved oxygen, in turn, results in filamentous microbial growth and poor settleability of sludge. If accumulation of sludge in the secondary clarification system is observed, then the secondary clarification design is at fault, and physical changes must be made to the system [63,66].

Nutrient limitations resulting in filamentous microbial growth can be identified by comparing the nutrient loading to the wastewater treatment plant to the organic (i.e., BOD_L) removal rate. The ratio of the nitrogen loading to the wastewater treatment plant relative to the BOD_L removal rate should be at least equal to the results obtained from Eq. (2.43). If the actual ratio is significantly less than that obtained from Eq. (2.43), there is a nutrient deficiency. The wastewater treatment operator can correct the problem either by adding supplemental nutrients to the aeration basin or by increasing the mean cell residence time θ_c.

If the former option is chosen to correct the nutrient deficiency, sufficient nutrients should be added to the system so that the ratio of BOD_L to nitrogen to phosphorus in the aeration basin is at least 100:5:1 (mass basis) [12,28]. It should be noted that in this approach, supplemental nutrients must be added as long as the nutrient deficiency exists. If the latter option is chosen to correct the nutrient deficiency, the plant operator simply reduces the mass of sludge wasted from the system. No nutrient supplements would be required in this case.

2.5.1.9 Rising sludge. Occasionally, sludge that has acceptable settling characteristics in laboratory tests will be observed to float to the surface of the secondary clarifier after a relatively short period of time. The cause of rising sludge is denitrification (i.e., conversion of nitrite and nitrate to nitrogen gas, N_2). As nitrogen gas is formed in the secondary clarifier, much of it is adsorbed to sludge particles, causing them to become buoyant. Rising sludge can be differentiated from bulking sludge or pin floc by noting the presence of small gas bubbles attached to the floating sludge.

Since denitrification will only occur when the dissolved oxygen concentration is approximately zero and the nitrate concentration is nonlimiting, certain operational conditions in the secondary clarifier are conducive to causing rising sludge problems [12,28]. For example, denitrification typically will occur in secondary clarification systems if the sludge is allowed to accumulate for long periods of time (more than several hours). Sludge accumulation not only results in depletion of dissolved oxygen but also results in the mineralization of organic nitrogen (i.e., conversion of organic nitrogen to ammonia) through endogenous respiration [28,60]. Rising sludge problems may be eliminated by the wastewater treatment operator by employing any of the following practices: (1) increasing the RAS flow rate, (2) decreasing the flow rate from the aeration basin to the problematic secondary clarifier, or (3) increasing the rate of sludge wasting [12,28].

2.5.1.10 Process control. The efficiency of the activated-sludge system to treat wastewater depends on the development of a flocculent mass of microorganisms that can be separated from the wastewater by gravity settling. Although the impact of the mean cell residence time θ_c on organic matter removal and sludge production is well understood, its influence on solids-liquid separation is not.

The microorganisms responsible for organic matter removal are generally colloidal in size when growing as individual cells, so if flocculation does not occur, their settleability in the secondary clarifier is limited. Several theories exist regarding the mechanism of bioflocculation, but in all cases, bioflocculation has been found to be absent below a threshold θ_c. One theory proposed that the presence of protozoa is essential to cause bioflocculation [12]. Since the protozoa do not grow as rapidly as bacteria, below a certain threshold θ_c, the protozoa concentration as well as the extent of bioflocculation decreases. Another theory on bioflocculation proposed that it is the generation of bacterial biopolymers that results in the bridging of microbial cells that is responsible for bioflocculation. This latter theory suggests that as the θ_c is reduced and cell growth is rapid, new surfaces are generated faster than the biopolymer can be produced, and little bridging of cells and bioflocculation can occur [12].

The practical impact of varying the θ_c on sludge settleability in the secondary clarifier is illustrated in Fig. 2.51. Figure 2.51 indicates that an increase in θ_c results in increasing the hindered settling velocity of the floc. At θ_c values of less than 2 days, there is dispersed microbial growth that prevents an accurate determination of the sludge settling velocity. Therefore, on the basis of secondary clarifier operation, a minimum θ_c of at least 3 days should be maintained within the activated-sludge system.

As the θ_c value is increased beyond 15 days, there is a deterioration of the settling characteristics of the sludge due to the formation of small floc particles, called *pin floc*. The mechanism for pin-floc formation is unclear, but it has been proposed that it is the result of

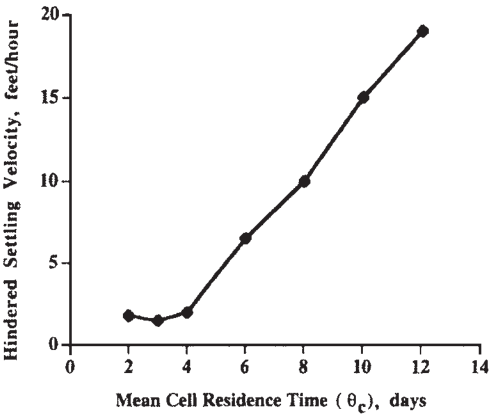


Figure 2.51 Impact of θ_c on hindered settling in the secondary clarifier. (*Adapted by permission from ref. [12].*)

excess biopolymer production [12,28]. Therefore, a θ_c value of 15 days is normally the upper limit used in activated-sludge operation when treating municipal wastewater. It should be noted that in some industrial operations, a θ_c value of greater than 15 days may be required to treat concentrated or toxic organic wastes. This requirement, however, must be evaluated on a case-by-case basis.

Because of its inherent flexibility, the activated-sludge system operation can be adapted to treat almost any type of organic waste. However, in all cases, to achieve a consistent effluent quality, a relatively constant θ_c value must be maintained even when the influent organic and/or hydraulic loading conditions change [12,28]. The significance of operating the activated-sludge system at a constant mean cell residence time is related to its impact on both the organic removal efficiency in the aeration basin and the performance of the secondary clarification system.

Variations in the long-term wastewater loading normally result from growth of the municipality being served by the wastewater treatment plant or of seasonal adjustments to loads (e.g., resort location). During long-term changes in plant loading, corrective action must be taken by the plant operator in the form of adjustments in the sludge wasting and RAS flow rates to ensure that the θ_c remains relatively constant. For example, a significant increase in organic loading to the wastewater treatment plant will result in the generation of more sludge within the aeration basin. To maintain a constant θ_c, more sludge may have to be wasted. At the same time, however, the RAS flow rate would be increased to ensure adequate organic matter removal in the aeration basin and to minimize sludge accumulation within the secondary clarification system.

Short-term loading changes are due primarily to the normal diurnal pattern in wastewater flow and strength (including shock loads) [12,60]. The most effective approach to reducing the negative effects of short-term changes is to dampen their magnitude by employing flow equalization.

2.5.1.11 Secondary clarification systems. The objective of the activated-sludge secondary clarification system is to separate sludge from the mixed liquor. It should be noted that at some wastewater treatment facilities, flotation rather than sedimentation is used to facilitate solids-liquid separation [28,66]. Although flotation is rarely used at municipal wastewater treatment facilities for secondary clarification, it is commonly employed at industrial pretreatment plants. The design approach employed for flotation systems is described in Chap. 3 under sludge-thickening processes (see Sec. 3.1.2).

For design of secondary clarification systems, the mechanism used to describe the settling of sludge is the hindered (or zone) settling model (see Sec. 2.4.1.3). Using this approach, the area required for the proper thickening of the mixed-liquor suspended solids depends on the limiting solids flux that can be transmitted through the clarifier. Since the limiting solids flux varies with both the characteristics of the sludge and the underflow pumping rate, gravity settling tests should be conducted to develop a batch flux and/or total flux curve (see Sec. 2.4.1.3). From the results of settling tests, the required underflow rate for the secondary clarifier Q_u to achieve a desired underflow sludge concentration X_r may be determined. It should be noted that since the underflow rate Q_u is the sum of both the sludge wastage rate Q'_w and the RAS flow rate Q_r, by establishing the RAS flow rate the wastewater treatment operator can effectively control both the underflow sludge concentration and the sludge blanket depth [28].

In designing a secondary clarification system, it is critical that the depth of the secondary clarifier be adequate to (1) ensure maintenance of an adequate sludge blanket and (2) provide temporary storage of sludge in excess of transmitting capacity. Typical design parameters for secondary clarifiers are given in Table 2.28.

Although the hindered settling mechanism is the appropriate model to use in the design of secondary clarifiers, because of mixed-liquor density currents, additional design factors must be addressed by the design engineer. These design factors include (1) tank type, (2) settling characteristics of the sludge, (3) surface and solids loading rate, (4) side water depth, (5) flow distribution, (6) inlet design, (7) weir placement, and (8) scum removal. Readers interested in additional information on the design of secondary clarification systems are referred to the following references [28,66].

TABLE 2.28 Design Parameters for the Secondary Clarifier*

	Surface loading rate (gal/ft^2 · day)		Solids loading rate (lb/ft^2 · h)		
Type of treatment	Average	Peak	Average	Peak	Depth (ft)
Conventional and complete mix WAS	400–800	1000–1200	0.8–1.2	2.0	12–20
Pure oxygen WAS	400–800	1000–1200	1.0–1.4	2.0	12–20
Extended aeration WAS	200–400	600–800	0.2–1.0	1.4	12–20

*Adapted from refs. [28,60].

2.5.1.12 Membrane processes for solids separation. In membrane liquid-solids separation technology, solids are removed from wastewater using a semipermeable membrane that removes wastewater constituents based on particle size. Most membranes used in wastewater treatment are fabricated from either ceramic or specially designed polymeric materials. Although the use of membranes to separate contaminants from water is well established for industrial applications, the employment of membranes to separate sludge solids from municipal wastewater is relatively new.

For separation of sludge solids, either microfiltration or ultrafiltration may be used. Microfiltration is capable of separating particles with an average size of 0.6 μm or larger, whereas ultrafiltration can remove particles that are 0.01 μm or larger. Therefore, in membrane systems designed to achieve complete suspended solids removal, ultrafiltration typically is employed [23].

Unlike conventional filtration, in which liquid passes vertically through the filter, in membrane filtration, liquid-solid separation is achieved by passing the sludge slurry across the membrane in a cross-flow fashion. The water that passes through the membrane is defined as the *permeate* and flows at right angles (i.e., cross-flow) to the bulk fluid movement (Fig. 2.52). The concentrated-solids slurry remaining is defined as the *retentate* (or *concentrate*). The efficiency of the membrane liquid-solids separation system depends on a pressure differential maintained between the permeate and retentate sides of the membrane. Typical pressure differences in an ultrafiltration system are in the range of 5 to 100 lb/in^2 (35–690 kPa).

Although effective in removing solids, membranes are subject to fouling (i.e., plugging) and degradation that can limit their effectiveness. To minimize fouling, the influent solids slurry typically is passed at high velocities over the membrane. This operational procedure results in a reduced rate of fouling but at the expense of a lower solids-liquid separation efficiency. To improve the overall liquid-solids separation efficiency for the system, the solids slurry must be either

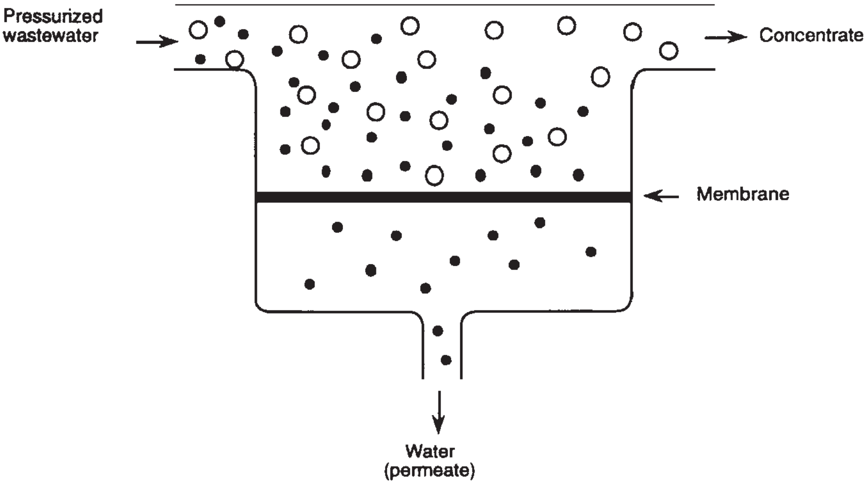
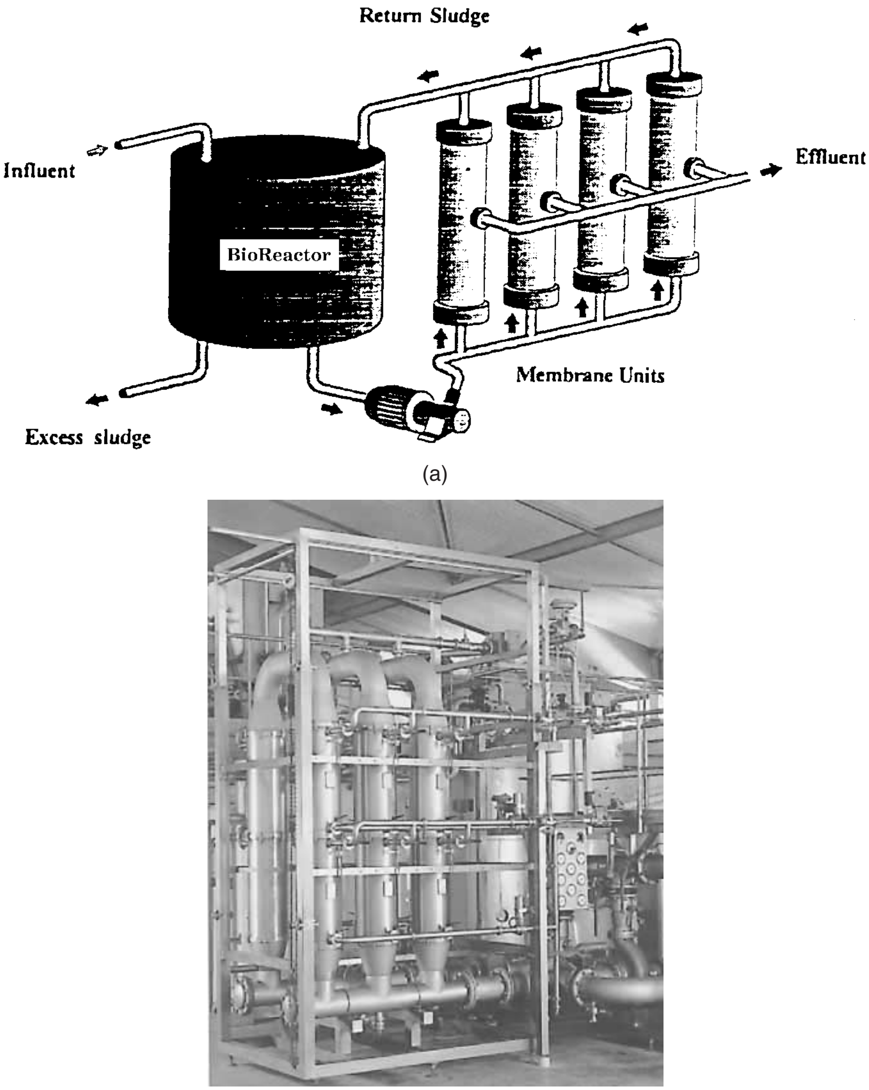


Figure 2.52 Schematic diagram of a membrane liquid-solids separation mechanism.

recycled through the membrane unit several times or conveyed through several membrane modules placed in series.

Recent advances in membrane technology have led to the commercialization of several secondary biological treatment processes that employ ultrafiltration membranes to achieve complete suspended solids removal. These membrane-based secondary wastewater treatment systems essentially have eliminated the need for secondary clarification (Fig. 2.53). The use of the membrane technology allows the aeration tank to operate at larger volatile suspended solids (VSS) concentrations (typically 10,000–20,000 mg/liter) and longer mean cell residence times (typical MCRT values range from 60 to 100 days) than standard activated-sludge systems. Sludge contained in the retentate is recycled back to the aeration tank to maintain the high biomass concentration and to ensure effective wastewater treatment. Excess sludge, which is wasted directly from the aeration tank, is conveyed to sludge-processing operations.

Benefits claimed for use of the membrane technology for liquid-solids separation over traditional activated-sludge secondary clarifiers include (1) zero suspended solids concentration in treated water, (2) increased reliability (e.g., no concern over bulking sludge), (3) long mean cell residence time in the aeration tank allows growth of nitrifying bacteria, (4) compact installation (i.e., smaller footprint than the standard activated-sludge system), and (5) flexible operation (i.e., modular design facilitates system expansion).



(a)

(b)

Figure 2.53 (*a*) Schematic diagram of a membrane-based secondary wastewater treatment system. (*Courtesy of Infilco Degremont, Inc.*) (*b*) A membrane filter system used in conjunction with a secondary wastewater treatment aeration tank. (*Courtesy of Infilco Degremont, Inc.*)

The principal disadvantages of membrane technology compared with the traditional activated-sludge system are the high capital and maintenance costs. Membrane systems are expensive to both purchase and operate. In addition to the high capital costs associated with the membrane separation system, effective liquid-solids separation requires maintaining high fluid pressures and flow velocities. Moreover, membranes must be cleaned routinely to prevent fouling. Finally, membranes inevitably wear out and must be replaced at regular intervals (typically 2–4 years depending on the application).

Although membrane liquid-solids separation technology is not presently cost competitive with traditional secondary treatment systems (e.g., activated sludge) for most municipal wastewater applications, in cases where there is a need for recycling of high-quality water (e.g., industrial applications) or where the municipal wastewater effluent discharge requirements are extremely stringent, use of membrane separation technology should be considered [22,23].

2.5.2 Trickling filters

A *trickling filter* is a fixed-film biological wastewater treatment system used primarily for the removal of soluble organic matter as well as the oxidation of ammonia (i.e., nitrification). In trickling filters, the applied wastewater percolates through a filter medium on which a microbial film (i.e., biofilm) is growing (Fig. 2.54). The aerobic microbial populations associated with the biofilm use the soluble organic matter as a source of energy and for the synthesis of new cells [12,28]. A principal advantage of fixed-film biological wastewater treatment systems over suspended-growth systems (e.g., activated sludge) is that their microbial populations are more resistant to shock loads of contaminants [28,60].

Two factors that affect the performance of a trickling filter are the hydraulic loading rate (expressed in units of gal/ft^2·day or m^3/m^2·day) and the organic loading rate (expressed in units of lb BOD$_5$/1000 ft^3·day or kg BOD$_5$/1000 m^3·day). As the microbial populations

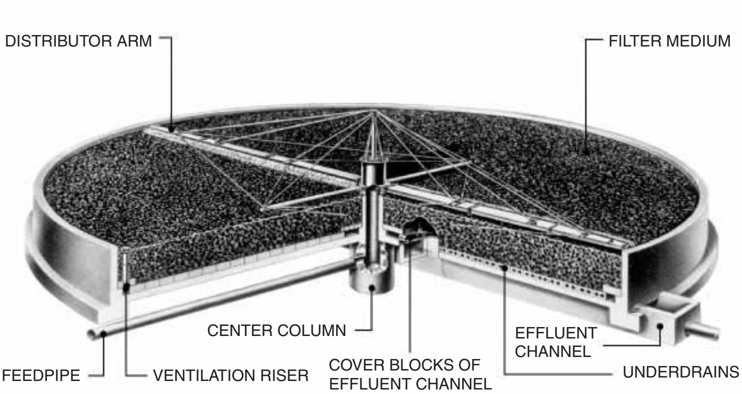


Figure 2.54 Schematic diagram of a trickling filter system. (*Courtesy of Dorr-Oliver.*)

continue to remove organic matter from the percolating wastewater, the thickness of the biofilm increases rapidly. Over time, the rate of both organic matter and oxygen transfer to the microbial cells attached to the medium surface decrease, resulting in the loss of the biofilm's ability to adhere to the medium surface. The weakened biofilm is then effectively removed from the medium by the shearing action of the percolating wastewater (a process called *sloughing*), and a new biofilm starts to grow [12].

The treated wastewater and any solids that may have become detached from the medium are collected in an underdrain and conveyed to a secondary clarification system. After they are separated from the treated wastewater in the secondary clarifier, the solids are transferred to downstream sludge-processing operations (e.g., thickening, conditioning, dewatering, etc.), while the wastewater is further treated (e.g., disinfection) prior to discharge. In some circumstances, a portion of the trickling filter wastewater effluent may be recycled to dilute the strength of the incoming wastewater and/or to maintain the attached biofilm in a moist condition [12].

Trickling filters are classified as low or standard rate, intermediate rate, high rate, super high rate, or roughing depending on their hydraulic and organic loading rates. A low-rate filter consists of a rectangular or circular vessel normally containing rock or slag medium.

Low-rate trickling filters maintain a constant hydraulic loading rate by employing suction-level control pumps or a dosing siphon rather than effluent recirculation [12,28]. Dosing tanks are small containers designed to minimize the time interval between wastewater application to the medium surface. Minimizing the dosing time interval is critical for maintaining treatment effectiveness because the biofilm activity will deteriorate if moisture becomes limiting [12]. Under normal operating conditions, autotrophic nitrifying bacteria typically will populate the lower portions of a low-rate trickling filter, resulting in the production of a highly nitrified effluent [28].

In intermediate- and high-rate filters, the dilution effect of recirculating effluent wastewater allows these systems to treat higher organic loadings than the low-rate system (see Table 2.29). Wastewater flow to the intermediate-rate filter is usually continuous, although intermittent wetting of the filter medium is permissible (Fig. 2.55). High-rate trickling filters normally employ a plastic medium and are designed to handle organic and hydraulic loadings substantially higher than either the low- or intermediate-rate filters (Fig. 2.56). High-rate trickling filters usually are circular, and flow is continuous [60]. In addition to improving organic matter removal, the increased rate of sloughing caused by wastewater effluent recirculation in high-rate trickling filters reduces the potential of surface ponding.

TABLE 2.29 Design Parameters for Trickling Filters*

Item	Low-rate	Intermediate-rate	High-rate	Super-high-rate	Roughing	Two-stage
Filter medium	Rock, slag	Rock, slag	Rock	Plastic	Plastic, redwood	Rock, plastic
Hydraulic loading (gal/ft^2·min)	0.02–0.06	0.06–0.16	0.16–0.64	0.2–1.2	0.8–3.2	0.16–0.64
BOD_5 loading (lb/10^3 ft^3·d)	5–25	15–30	30–60	30–100	100–500	60–120
Depth (ft)	6–8	6–8	3–6	10–40	15–40	6–8
Recirculation ratio (Q_R/Q)	0	0–1	1–2	1–2	1–4	0 .5–2
Filter flies	Many	Some	Few	Few or none	Few or none	Few or none
Sloughing	Intermittent	Intermittent	Continuous	Continuous	Continuous	Continuous
BOD removal efficiency (%)	80–90	50–70	65–85	65–80	40–65	85–95
Effluent quality	Nitrified	Partially nitrified	Little nitrification	Little nitrification	No nitrification	Nitrified

NOTE: gal/ft^2·min·58.674 = m^3/m^2·day; lb/10^3 ft^3·day·0.016 = kg/m^3·day.

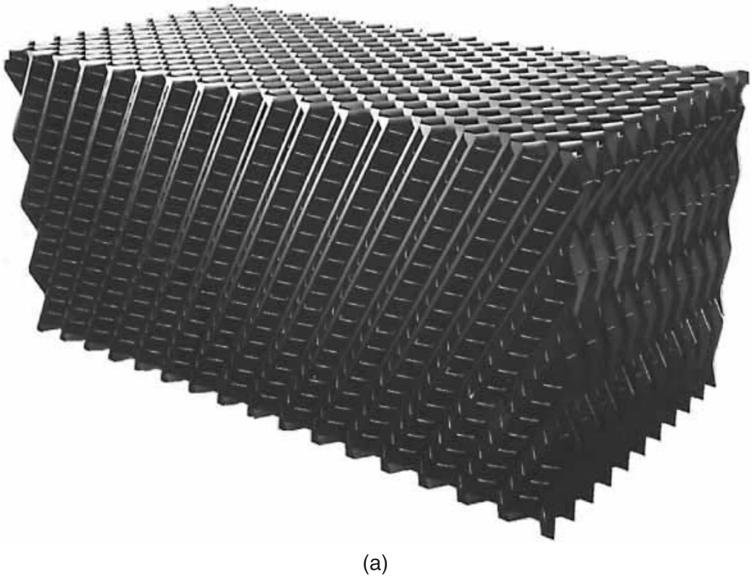
*Adapted from refs. [12,28].

Figure 2.55 An intermediate-rate trickling filter.

Super-high-rate trickling filters are loaded at high hydraulic and organic loading rates (Table 2.29). The principal difference between super-high-rate and high-rate trickling filters is the increased hydraulic loading and medium depth that characterize the super-high-rate systems. The greater filter depths are possible because the super-high-rate filter normally employs a lighter plastic medium rather than a rock or slag medium [12]. Finally, roughing filters are high-rate trickling filters that treat an organic load of more than 100 lb BOD_5/1000 $ft^3 \cdot day$ (1.6 $kg/m^3 \cdot day$) and hydraulic loadings as high as 3.2 $gal/ft^2 \cdot min$ (187 $m^3/m^2 \cdot day$) [12]. Roughing filters typically are used as a pretreatment step for reducing the organic load to the principal secondary treatment system [30].

In addition to classifying trickling filter systems based on organic and/or hydraulic loading rate, trickling filters may be operated in series to achieve specific water-quality goals. Two or more trickling filters connected in series are classified as a *multistage filter* and often are used to treat wastewater with organic loadings in excess of what could be handled by a single trickling filter [12,30]. Multistage systems also are used when nitrification of wastewater effluent is required. In this operation, the first-stage filter and intermediate clarification system reduce the carbonaceous BOD_5, whereas nitrification (i.e., conversion of ammonia to nitrite and nitrate) occurs in the second or subsequent stages (Fig. 2.57).

The range of hydraulic and organic loadings normally encountered in practice, together with other operational characteristics for the various trickling filter classifications, is provided in Table 2.29. In general, the lower hydraulic loading limits reported in Table 2.29 are the minimum



(a)

(b)

Figure 2.56 A MARPAK corrugated PVC medium used in trickling filters. (*Courtesy of Marley Cooling Tower.*) (*a*) Vertical-flow plastic medium module. (*b*) Plastic medium installed in a trickling filter.

wastewater application rates required to completely wet the medium surface, whereas the upper hydraulic loading limits reflect the maximum rate at which wastewater can be transmitted through the tortuous voids of the filter without resulting in surface ponding [28,30].

Because of its impact on biofilm growth, the organic loading rate will dictate the minimum hydraulic loading rate necessary to ensure wastewater percolation through the trickling filter bed. For example,

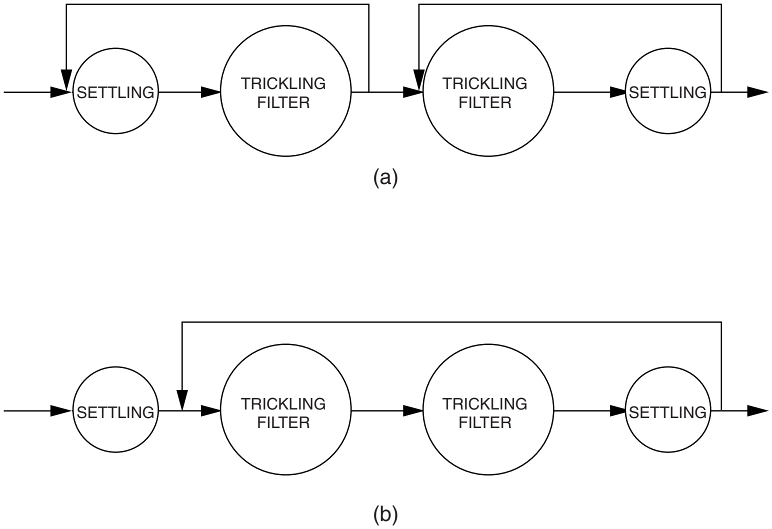


Figure 2.57 (*a*) Filters in series with clarified effluent recycle from individual filters. (*b*) Filters in series with clarified effluent from entire process chain.

to properly manage biofilm growth, high organic loadings must be accompanied by a high hydraulic loading rate. If the hydraulic loading rate is insufficient to induce an adequate rate of sloughing, excessive biofilm thickness will clog the pores of the trickling filter, causing surface ponding and eventual system failure [28,30].

The major design components of all trickling filters include (1) the filter medium, (2) the wastewater distribution system, and (3) an underdrain system. In addition to these components, the design engineer must weigh the advantages and costs of incorporating effluent recirculation in trickling filter operation. Each of these components, as well as the operational advantages and concerns regarding effluent wastewater recirculation, are described briefly in the following sections.

2.5.2.1 Filter medium. The filter medium provides the solid surface on which the microorganisms responsible for wastewater treatment adhere and grow. The ideal filter medium should have the following characteristics:

1. Large surface area for microbial growth
2. Large void space for airflow and removal of sloughed biological solids
3. Biologically and chemically inert
4. Mechanically stable

TABLE 2.30 Physical Properties of Trickling Filter Media*

Medium	Nominal size (in)	Bulk density (lb/ft^3)	Specific area (ft^2/ft^3)	Void space (%)
River rock				
Small	1–2.5	78–90	17–21	40–50
Large	4–5	50–62	12–50	50–60
Slag				
Small	2–3	56–75	17–21	40–50
Large	3–5	50–62	14–18	50–60
Plastic				
Conventional†	24·24·48	2–6	25– 30	94–97
High surface area†	24·24·48	2–6	30–60	94–97
Random pack	1–3.5	3–6	30–104	93–95
Redwood†	48·48·20	9–11	12–15	70–80

*Adapted from ref. [28].
†Module size.

Although a large number of materials have been evaluated, the most successful trickling filter media have been gravel or limestone river rock, slag, plastic, and redwood [28,30]. The physical characteristics of commonly used trickling filter media are reported in Table 2.30.

When locally available, rock medium has the economic advantage of low cost. The most suitable rock media are gravel and crushed limestone rock graded to a uniform size so that 95 percent is within 3 to 4 inches in diameter [30]. The tank or vessel constructed for a rock medium trickling filter must be made of reinforced concrete or other material capable of structurally maintaining the medium in place. The enclosure also must have sufficient strength to allow occasional flooding of the medium (which is sometimes required for nuisance control). Because of the medium weight, the depth of rock filters is usually limited to a range of 5 to 10 ft (1.5–3.0 m) [12,30].

Plastic trickling filter medium comes in two types: (1) random (or loose) packing and (2) modular (or stacked). The random-packing medium will have a range of specific surface areas of between 30 and 104 ft^2/ft^3 (98 and 340 m^2/m^3) with void ratios of 93 to 95 percent, whereas modular plastic medium will have a range of specific surface areas that varies from 25 to 60 ft^2/ft^3 (81–195 m^2/m^3) with void ratios of 94 to 97 percent [28,30]. Like rock medium filters, the vessel or tank constructed for a trickling filter filled with random-packing plastic medium must be supported vertically, although its low bulk density requires less structural support than rock medium. Modular plastic medium is self-supporting, and therefore, enclosures serve primarily to minimize splashing and heat loss and to improve the aesthetics of the system. Filters as deep as 40 ft (12 m) have been constructed using modular plastic medium [7,60].

2.5.2.2 Wastewater distribution system. The purpose of the wastewater distribution system is to provide a uniform hydraulic load of wastewater over the filter medium. Two types of wastewater distributors are used: rotary and fixed-nozzle systems [12,30].

The rotary distributor has become a standard for rock medium trickling filters because it is reliable and easy to maintain. A rotary distributor consists of two or more hollow arms (up to 200 ft in length) mounted on a pivot that allows revolution along a horizontal plane (Fig. 2.58).

The distributor arm contains a set of nozzles through which the wastewater is discharged onto the filter medium. Nozzles are spaced unevenly along the distributor arm so that a greater flow per unit length is achieved near the periphery of the trickling filter bed than at the center [12,60]. The driving force required to move the distributor assembly may be supplied by the momentum of the wastewater discharging from the nozzles or from the torque generated by an electric motor [28,60].

Fixed-nozzle distribution systems typically are employed in trickling filters filled with plastic medium. Fixed-nozzle distribution systems consist of a series of spray nozzles positioned at various locations that permit complete coverage of the cross-sectional area of the trickling filter bed by discharged wastewater. Typically, twin dosing tanks equipped with automatic siphons are employed to supply the nozzles with wastewater [12,60].

To optimize the performance of trickling filters, there should be uniform biofilm growth together with a continuous removal of excess biofilm through sloughing. These conditions can be achieved by establishing a proper wastewater application or dosing rate. Suggested dosing rates (in inches of water per pass) as a function of organic loading for rotary distributors are provided in Table 2.31.

Figure 2.58 Rotary wastewater distribution system.

TABLE 2.31 Typical Dose Rates for Trickling Filters*

Organic loading rate ($\text{lb BOD}_5/10^3\ \text{ft}^3$)	Dosing rate (in/pass)
<25	3
50	6
75	9
100	12
150	18
200	24

*Adapted from refs. [12,28].

The rate of biofilm sloughing depends on the instantaneous wastewater dosing rate, which, in turn, is a function of the rotational speed of the distributor or the on/off times for a fixed-nozzle wastewater distribution system. The required rotational speed for a rotary distributor to achieve a desired dosing rate may be estimated using Eq. (2.44). Example 2.18 illustrates the use of Eq. (2.44) in establishing the proper rotational speed of a rotary distributor.

$$n = \frac{1.6Q_T}{ADR} \tag{2.44}$$

where n = rotational speed of distributor, rev/min
Q_T = total applied hydraulic loading, gal/ft^2 · min ($Q + Q_R$)
Q = influent hydraulic loading, gal/ft^2 · min
Q_R = recycle hydraulic loading, gal/ft^2 · min
A = number of arms in rotary distributor assembly
DR = dosing rate, inches per pass of distributor arm

For effective wastewater treatment, the speed of the rotary distributor may be controlled by any of the following approaches:

1. Reversing the location of some existing nozzles to the front of the distributor arm
2. Adding reversed deflectors to the exiting nozzles
3. Employing a variable-speed electric drive

Example 2.18 A low-rate rock medium trickling filter is being employed to treat a municipal wastewater flow rate of 2 million gallons per day (2 MGD) having an average BOD_5 concentration of 150 mg/liter. If the trickling filter is 150 ft in diameter and 6 ft in depth, estimate the dosing rate (inches per pass) and the rotational speed of the distributor arm. Assume that the recirculation rate is 50 percent of the influent flow rate and that the rotary distributor has two arms.

solution

Step 1. Estimate the BOD_5 mass loading rate in pounds per day:

$$\frac{\text{lb BOD}_5}{\text{day}} = \text{BOD}_5 \text{ conc. (mg/liter)} \cdot \text{flow rate (MGD)} \cdot \frac{8.34 \text{ lb}}{\text{MG}\left(\dfrac{\text{mg}}{\text{liter}}\right)}$$

$$= 150 \text{ mg/liter} \cdot 2 \text{ MGD} \cdot 8.34 = \frac{2502 \text{ lb BOD}_5}{\text{day}}$$

Step 2. Estimate the organic loading rate in units of lb $BOD_5/1000$ ft^3·day.

Volume of trickling filter:

$$\text{Volume (ft}^3) = \frac{\pi \text{ (diameter)}^2}{4} \cdot \text{height}$$

$$= \frac{3.14 \text{ (150 ft)}^2}{4} \cdot 6 \text{ ft} = 106{,}071 \text{ ft}^3$$

Organic loading rate:

$$\text{Organic loading rate}\left(\frac{\text{lb BOD}_5}{10^3 \text{ ft}^3 \cdot \text{day}}\right) = \frac{\dfrac{2502 \text{ lb BOD}_5}{\text{day}}}{106.071 \cdot 10^3 \text{ ft}^3}$$

$$= \frac{23.6 \text{ lb BOD}_5}{10^3 \text{ ft}^3 \cdot \text{day}}$$

Step 3. Estimate the dosing rate from Table 2.31. Since the organic loading rate is less than 25 lb $BOD_5/10^3$ ft^3·day, the dosing rate DR should be set at 3 in per pass.

Step 4. Estimate the total applied hydraulic loading rate Q_T:

$$Q_T\left(\frac{\text{gal}}{\text{ft}^2 \cdot \text{min}}\right) = \frac{\text{plant flow + recirculation flow}}{\text{area of trickling filter}}$$

$$= \frac{\dfrac{2 \cdot 10^6 \text{ gal}}{\text{day}} \cdot \left(\dfrac{\text{day}}{1440 \text{ min}}\right) + \dfrac{1 \cdot 10^6 \text{ gal}}{\text{day}} \cdot \left(\dfrac{\text{day}}{1440 \text{ min}}\right)}{\dfrac{\pi \text{ (150 ft)}^2}{4}}$$

$$= \frac{0.12 \text{ gal}}{\text{ft}^2 \cdot \text{min}}$$

Step 5. Estimate the rotational speed of the distributor using Eq. (2.44):

$$n = \frac{1.6\,Q_T}{A \cdot DR}$$

$$= \frac{1.6 \cdot \dfrac{0.12\text{ gal}}{\text{ft}^2 \cdot \text{min}}}{2 \cdot \dfrac{3\text{ in}}{\text{pass}}} = 0.032\text{ rev/min (one revolution every 31 min)}$$

2.5.2.3 Underdrain system. The function of the trickling filter underdrain system is to (1) convey wastewater and solids discharged from the filter to the secondary clarification system, (2) ensure adequate ventilation for oxygen transfer to the biofilm, and (3) provide physical support of the filter media [7,28,30]. Once conveyed to the secondary clarification system, sloughed solids are separated from the treated wastewater. The design of the secondary clarification system for trickling filters is similar to primary settling tanks except that the surface loading rate is based on the plant flow plus recirculation flow (if employed). Unlike the activated-sludge process, all microbial solids contained in the secondary clarifier underflow in trickling filter operations are transported to downstream sludge-processing operations.

Since ambient air is the source of oxygen for the biofilm, adequate ventilation is critical to successful operation of a trickling filter. The principal mechanism responsible for airflow in an open-top trickling filter is natural ventilation or draft. *Draft,* which is defined as the pressure resulting from the temperature difference between the ambient air and the air within the medium pores, may be estimated using Eqs. (2.45) and (2.46).

$$D_{\text{air}} = 7.64\left(\frac{1}{T_c} - \frac{1}{T_h}\right)Z \qquad \text{(U.S. units)} \qquad (2.45)$$

$$D_{\text{air}} = 35.3\left(\frac{1}{T_c} - \frac{1}{T_h}\right)Z \qquad \text{(SI units)} \qquad (2.46)$$

where D_{air} = natural air draft, in H_2O (mm H_2O)
T_c = cold temperature, °R (460 + °F), K
T_h = warm temperature, °R (460 + °F), K
Z = height of filter, ft (m)

Since the pore temperature will vary with location within the trickling filter, the log mean temperature difference should be employed in calculating an average pore temperature [Eq. (2.47)]. Example 2.19 illustrates the approach for estimating the magnitude of natural draft in trickling filter operation.

$$T_{\text{pore}} = \frac{T_2 - T_1}{\ln (T_2/T_1)} \qquad (2.47)$$

where T_{pore} = average pore temperature
T_1 = colder pore temperature
T_2 = warmer pore temperature

Example 2.19 A 35-ft modular plastic medium trickling filter is being employed to treat municipal wastewater. From an investigation of the temperature profile within the filter, you have determined that a maximum temperature of 69.2°F (20.7°C) occurs at a depth of 18 ft within the filter, while a minimum temperature of 64.3°F (17.9°C) occurs at a filter depth of 4 ft. Estimate the draft in the filter (in in H_2O) if the average ambient air temperature is 85°F (29°C).

solution

Step 1. Estimate the average pore temperature in degrees Rankine using Eq. (2.47):

$$T_{\text{pore}} = \frac{T_2 - T_1}{\ln (T_2/T_1)} = \frac{69.2°\text{F} - 64.3°\text{F}}{\ln (69.2°\text{F}/64.3°\text{F})} = 66.7°\text{F}$$

$$= 66.7°\text{F} + 460°\text{R} = 526.7°\text{R}$$

Step 2. Convert the ambient air temperature from degrees Fahrenheit to degrees Rankine:

$$\text{Ambient air temperature (°R)} = 85°\text{F} + 460°\text{R}$$

$$= 545°\text{R}$$

Step 3. Estimate the draft in inches of H_2O using Eq. (2.45):

$$D_{\text{air}} = 7.64 \left(\frac{1}{T_c} - \frac{1}{T_h} \right) Z$$

$$= 7.64 \left(\frac{1}{526.7°\text{R}} - \frac{1}{545°\text{R}} \right) 35 \text{ ft}$$

$$= 0.017 \text{ in } H_2O$$

It should be noted that when the applied wastewater is colder than the ambient air, the pore air will sink, and the direction of airflow will be downward. If the applied wastewater is warmer than the ambient air, the pore air will rise, and the overall airflow will be upward [12,30]. Under certain circumstances, the natural draft will be insufficient to overcome the frictional energy losses imparted by the filter medium and will result in air stagnation. Since proper operation of the filter depends on adequate oxygen transfer, care must be taken to minimize the occurrence of air stagnation [12]. Natural draft has proven adequate for supplying oxygen to tricking filters provided the following precautions are taken:

1. Underdrains and collecting channels are designed to flow no more than half full.
2. Ventilating manholes are used with open grating installed at both ends of a central collection channel.
3. Large-diameter filters are equipped with branch collecting channels with ventilating manholes or vent stacks.
4. For rock media, an open area is provided for ventilating manholes or vent stacks equal to or exceeding 1 ft^2 per 250 ft^2 of filter cross-sectional area.
5. For plastic media, the ventilation area is sufficient to allow for 90 ft^3/min of air per 100 lb/day BOD_5 applied (5.6 m^3/min per 100 kg BOD_5/day).

In extremely deep or heavily loaded trickling filters there may be an advantage in using forced-air ventilation to supply oxygen. At a minimum, forced-air ventilation should provide for an airflow of at least 1 standard cubic foot of air per square foot of filter cross-sectional area (1 $ft^3/ft^2 \cdot min$ or 0.3 $m^3/m^2 \cdot min$) in either direction [12]. Finally, it may be necessary during periods of low ambient air temperature to restrict the airflow though the filter to prevent the moisture from freezing.

The underdrain system for a rock medium filter usually consists of precast blocks of vitrified clay or fiberglass grating laid on a reinforced-concrete floor [12,30]. The floor and underdrains must have sufficient strength to support the medium, microbial film growth, and wastewater. The underdrain normally will slope to a central or peripheral drainage channel at a 1 to 5 percent grade [28,60]. The underdrains and support system for plastic medium consist of either a beam-and-column or grating configuration [30]. The former configuration typically has precast beams supported by columns or posts. Readers interested in additional information on trickling filter underdrain design and construction are referred to the following references [7,28,30,60].

2.5.2.4 Recirculation in trickling filters. *Recirculation* is the process in which a portion of the treated wastewater effluent is recycled and mixed with the influent wastewater. Once a trickling filter has been built, recirculation of treated wastewater is the only approach available to adjust the hydraulic and organic loading rates independently [28]. In other words, by employing recirculation of treated wastewater effluent, the hydraulic loading can be increased to an appropriate value while maintaining a constant organic loading rate (Fig. 2.59). This is particularly important when treating a high-strength wastewater that is associated with a hydraulic loading rate inadequate to maintain the required wetting conditions and/or proper

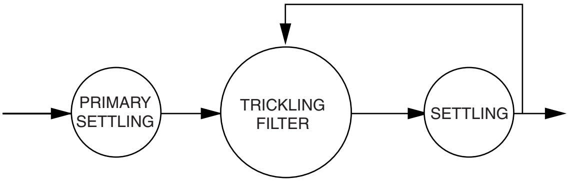


Figure 2.59 Diagram of a trickling filter employing recirculation.

sloughing of the biofilm [28,30]. Because there is a minimum hydraulic loading necessary to keep all of the medium wet and to prevent surface ponding, some facilities vary the amount of recirculation throughout the day [60].

Another instance where effluent recirculation enhances trickling filter performance is in the control of filter flies. These insects, which are a particular problem in low-rate trickling filters with excessive biofilm accumulation, are a nuisance and a potential health hazard [28]. To control insect growth, the hydraulic loading can be increased through effluent recirculation to reduce the biofilm thickness, or the filter can be flooded periodically to kill larvae. If neither technique is effective, periodic spraying of insecticide may be necessary [12].

Finally, in some trickling filters, odors have been identified as both a nuisance and a major source of public concern [28]. Odors emanating from trickling filters are generally the result of anaerobic conditions caused by excessive biofilm thickness and/or organic loadings. To alleviate anaerobic conditions, the hydraulic loading rate should be increased through effluent recirculation to increase the rate of biofilm sloughing and/or dilution of the incoming wastewater strength [60].

2.5.2.5 Trickling filter performance. Performance of a trickling filter is affected by many parameters, including hydraulic loading rate, organic loading rate, degree of recirculation, temperature, wastewater characteristics, etc. Because the relationship of these variables is not completely understood, empirical design equations or models normally are employed in trickling filter design. The design equations used for trickling filters usually are constructed to permit calculation of the required filter depth necessary to reduce the pollutant concentration to a specific value [30]. After a particular design has been selected based on both the design equation results and economic analysis, a pilot plant should be operated to verify the system's ability to produce the desired effluent quality and to estimate the production of biological solids.

A universally accepted design equation for trickling filters is not available. However, Eq. (2.48) has proven reliable for describing the organic matter removal of trickling filters employing a plastic medium [28]:

$$\frac{S_e}{S_i} = \exp\,[-k_{20}D\,(Q_v)^{-n}] \tag{2.48}$$

where S_e = total BOD_5 of settled effluent from filter, mg/liter
S_i = total BOD_5 of wastewater applied to filter, mg/liter
k_{20} = treatability constant corresponding to a specific medium of depth D at 20°C (68°F)
D = depth of filter, ft
Q_v = hydraulic loading rate, gal/ft^2 · min $[(Q + Q_R)/\text{area}]$
n = constant (normally 0.5)
Q = plant flow (gal/min)
Q_R = recirculation flow (gal/min)

Typical treatability constants k_{20} for treating a variety of wastewaters in a 20-ft trickling filter employing plastic medium are given in Table 2.32.

When a treatability constant measured at one trickling filter depth is used to design a trickling filter of a different depth, the treatability constant must be corrected for the new depth using Eq. (2.49). Example 2.20 illustrates the use of Eqs. (2.48) and (2.49) to design trickling filters employing plastic media.

$$k_2 = k_1\left(\frac{D_1}{D_2}\right)^x \tag{2.49}$$

where k_2 = treatability constant corresponding to filter of depth D_2
k_1 = treatability constant corresponding to filter of depth D_1
D_1 = depth of filter 1, ft
D_2 = depth of filter 2, ft
x = 0.3 for cross-flow plastic medium filters
= 0.5 for vertical-flow plastic medium filters

TABLE 2.32 Treatability Constants *k* at 20°C (68°F) for a 20-ft Trickling Filter Employing Plastic Medium*

Type of wastewater	Treatability constant k_{20}
Domestic septage	0.065–0.10
Domestic and food waste	0.060–0.08
Meat packing	0.030–0.05
Paper mill wastes	0.020–0.04
Potato processing wastes	0.035–0.05
Refinery	0.020–0.07

*Adapted from ref. [28].

Example 2.20 A 30-ft circular trickling filter employing vertical-flow modular plastic medium is designed to treat a municipal wastewater flow of 5 million gallons per day (5 MGD) having an organic matter concentration of 200 mg/liter BOD_5. If the desired effluent organic matter concentration is 30 mg/liter BOD_5, estimate the diameter of the filter if the recirculation rate is maintained at 50 percent of the influent flow rate. Assume that data from a 10-ft pilot-plant filter using identical wastewater indicated that to achieve the desired effluent quality required a treatability constant k_{20} of 0.09.

solution

Step 1. Using the pilot-plant data and Eq. (2.49), estimate the required treatability constant k_{20} to use in the full-scale design:

$$k_2 = k_1 \left(\frac{D_1}{D_2}\right)^x = 0.09 \left(\frac{10}{30}\right)^{0.5}$$

$$= 0.052$$

Step 2. Determine total volumetric flow rate to the filter in gallons per minute:

$$\text{Plant flow } Q = \frac{5 \cdot 10^6 \text{ gal}}{\text{day}} \cdot \frac{\text{day}}{1440 \text{ min}} = \frac{3472 \text{ gal}}{\text{min}}$$

$$\text{Recirculation flow } Q_R = \frac{2.5 \cdot 10^6 \text{ gal}}{\text{day}} \cdot \frac{\text{day}}{1440 \text{ min}} = \frac{1736 \text{ gal}}{\text{min}}$$

$$\text{Total flow to filter } (Q + Q_R) = \frac{3472 \text{ gal}}{\text{min}} + \frac{1736 \text{ gal}}{\text{min}} = \frac{5208 \text{ gal}}{\text{min}}$$

Step 3. Estimate the cross-sectional area of the filter by rearranging Eq. (2.48):

$$\frac{S_e}{S_i} = \exp\left[-k_{20}D\,(Q_v)^{-n}\right] = \exp\left[-k_{20}D\left(\frac{Q + Q_R}{\text{area}}\right)^{-n}\right]$$

$$\left(\frac{Q + Q_R}{\text{area}}\right)^{-n} = \frac{-\ln(S_e/S_i)}{k_{20}D}$$

$$\text{Area} = (Q + Q_R)\left[\frac{-\ln(S_e/S_i)}{k_{20}D}\right]^{1/n}$$

$$= \frac{5208 \text{ gal}}{\text{min}}\left[\frac{-\ln(30/200)}{0.052 \cdot 30}\right]^{1/0.5} = 7702 \text{ ft}^2$$

Step 4. From geometric considerations, estimate the diameter of the trickling filter:

$$\text{Area} = \frac{\pi\,(\text{diameter})^2}{4}$$

$$7702\ \text{ft}^2 = \frac{\pi\,(\text{diameter})^2}{4}$$

$$\text{Diameter} = 99.0\ \text{ft}$$

Finally, because the airflow through the trickling filter acts to cool the wastewater, consideration must be given to the effects of temperature on the organic matter removal efficiency. One approach to account for the effects of temperature in the design of trickling filters is to employ Eq. (2.50):

$$k_2 = k_1\theta^{T_2 - 20} \tag{2.50}$$

where k_2 = treatability constant at temperature T_2 (Celsius)
k_1 = treatability constant at 20°C (68°F)
θ = temperature correction coefficient (normally 1.035)

Because of the irregular shape of gravel and crushed limestone, the National Research Council (NRC) has developed empirical expressions to predict the performance of trickling filters employing rock media. The NRC equations are primarily applicable to single and multistage rock media filters with varying recirculation rates. For a single-stage rock media filter, Eq. (2.51) may be used for trickling filter design:

$$E_1 = \frac{100}{1 + 0.0561\,(W/VF)^{0.5}} \tag{2.51}$$

where E_1 = efficiency of BOD_5 removal in the first filter at 20°C (68°F), percent
W = organic loading to filter, lb BOD_5/day
V = volume of filter media, $10^3\ \text{ft}^3$
F = recirculation factor

The recirculation factor F employed in Eq. (2.51) may be estimated using Eq. (2.52). Example 2.21 illustrates the use of Eqs. (2.51) and (2.52) in the design of a rock medium filter.

$$F = \frac{1 + R}{[1 + (R/10)]^2} \tag{2.52}$$

where R = recycle ratio, Q_R/Q

Q_R = recycle flow, MGD

Q = wastewater flow, MGD

If a second-stage rock filter is employed (e.g., multistage trickling filter system), the NRC expression that should be employed is given in Eq. (2.53):

$$E_2 = \frac{100}{1 + [0.0561/(1 - E_1)\,]\,(W'/VF)^{0.5}} \tag{2.53}$$

where E_2 = efficiency of $\mathrm{BOD_5}$ removal in second stage at 20°C (68°F), percent

E_1 = efficiency of $\mathrm{BOD_5}$ removal in the first stage at 20°C (68°F), percent

W' = organic loading applied to second stage filter, lb $\mathrm{BOD_5}$/day

V = volume of filter media, 10^3 $\mathrm{ft^3}$

F = recirculation factor

Example 2.21 A single-stage circular rock medium trickling filter 6 ft in depth is used to treat a municipal wastewater having a flow rate of 1.2 million gallons per day and an influent organic matter concentration of 180 mg/liter $\mathrm{BOD_5}$. If the desired effluent concentration is 30 mg/liter $\mathrm{BOD_5}$, estimate the diameter of the filter if the recirculation flow is maintained at 600,000 gal/day.

solution

Step 1. Estimate the efficiency of $\mathrm{BOD_5}$ removal:

$$\mathrm{BOD_5}\text{ removal efficiency } E = \frac{S_i - S_e}{S_i} \cdot 100$$

$$= \frac{180\text{ mg/liter} - 30\text{ mg/liter}}{180\text{ mg/liter}} \cdot 100$$

$$= 83.3\text{ percent}$$

Step 2. Estimate the recirculation factor F using Eq. (2.52):

$$R = \frac{Q_R}{Q} = \frac{0.6\text{ MGD}}{1.2\text{ MGD}} = 0.5$$

$$F = \frac{1 + R}{[1 + (R/10)]^2} = \frac{1 + 0.5}{[1 + (0.5/10)]^2} = 1.36$$

Step 3. Estimate the organic loading rate to the trickling filter (W) in lb $\mathrm{BOD_5}$ per day:

$$\frac{\text{lb BOD}_5}{\text{day}} = \text{BOD}_5 \text{ conc. (mg/liter)} \cdot \text{flow rate (MGD)} \cdot \frac{8.34 \text{ lb}}{\text{MG} \cdot \text{(mg/liter)}}$$

$$= \frac{180 \text{ mg}}{\text{liter}} \cdot 1.2 \text{ MGD} \cdot \frac{8.34 \text{ lb}}{\text{MG} \cdot \text{(mg/liter)}} = \frac{1801 \text{ lb BOD}_5}{\text{day}}$$

Step 4. Estimate the volume of the trickling filter using Eq. (2.51):

$$E_1 = \frac{100}{1 + 0.0561\,(W/VF)^{0.5}}$$

$$0.833 = \frac{100}{1 + 0.0561\left(\dfrac{1801 \text{ lb BOD}_5\text{/day}}{\text{V} \cdot 1.36}\right)^{0.5}}$$

$$V = 150.07 \quad (\text{or } 150{,}070 \text{ ft}^3)$$

Step 5. Using geometry, estimate the diameter of the filter:

$$\text{Volume} = \frac{\pi\,(\text{diameter})^2}{4} \cdot \text{height}$$

$$150{,}070 \text{ ft}^3 = \frac{\pi\,(\text{diameter})^2}{4} \cdot 6 \text{ ft}$$

$$\text{Diameter} = 178 \text{ ft}$$

2.5.2.6 Biological solids production in trickling filters. To properly design solids management facilities, it is necessary to determine the mass of biological solids generated by the trickling filter system. The daily production of biological solids (pounds volatile solids per day) from a full-scale trickling filter wastewater treatment system is estimated by multiplying the rate of BOD_5 removed (pounds per day) by the observed microbial yield (Y_{obs} – pounds volatile solids produced per pound BOD_5 removed) determined from a pilot-plant study.

A major limitation to this approach for estimating biological solids production has been the determination of a reasonable microbial yield value [12]. This limitation stems from the fact that the observed microbial yield is estimated by dividing the mass of sloughed solids by the mass of BOD_5 removed over a given time period during pilot-plant operations. Since the rate of sloughing is highly variable, significant fluctuations in the microbial yield value will occur if the data used in the calculation are collected over short periods of time [12,30]. Therefore, the aggregate mass of both the BOD_5 removed and sloughed biological solids generated over the entire duration of the pilot study should be used in calculating the microbial yield. Analyses of several full-scale and pilot-plant trickling filter operations have resulted in observed microbial yield Y_{obs} ranging from 0.48 to 0.92 lb volatile solids (VS) per lb BOD_5 removed (with an average value of 0.67 lb VS per lb BOD_5) [12]. Example 2.22 illustrates the approach for estimating the mass of biological solids (dry basis) generated in a typical trickling filter operation.

Example 2.22 The Walcott County Water Reclamation Facility is treating 4.5 million gallons per day of domestic wastewater in a multistage rock medium trickling filter system. The average influent organic matter concentration is 220 mg/liter BOD_5. If the effluent BOD_5 concentration from the last trickling filter was 25 mg/liter BOD_5, estimate the daily mass of biological solids produced (dry basis) if the observed microbial yield estimated from the pilot-plant study was 0.6 lb volatile solids per pound BOD_5 removed.

solution

Step 1. Estimate the daily rate of BOD_5 removed (lb/day):

$$\left(\frac{\text{lb BOD}_5}{\text{day}}\right)_{\text{removed}} = [\text{BOD}_5\text{ (influent)} - \text{BOD}_5\text{ (effluent)}] \cdot \text{flow rate (MGD)} \cdot \frac{8.34\text{ lb}}{\text{MG (mg/liter)}}$$

$$= (220\text{ mg/liter} - 25\text{ mg/liter}) \cdot 4.5\text{ MGD} \cdot \frac{8.34\text{ lb}}{\text{MG (mg/liter)}} = \frac{7318.4\text{ lb BOD}_5}{\text{day}}$$

Step 2. Estimate the daily rate of biological solids produced (lb VS/day, dry basis) from the multistage rock filter system:

$$\frac{\text{lb VS}}{\text{day}} = Y_{\text{obs}}\left(\frac{\text{lb BOD}_5}{\text{day}}\right)_{\text{removed}}$$

$$= \frac{0.6\text{ lb VS}}{\text{lb BOD}_5} \cdot \frac{7318.4\text{ lb BOD}_5}{\text{day}}$$

$$= \frac{4391.0\text{ lb VS}}{\text{day}}$$

Although estimating the daily biological solids production rate from trickling filter systems is critical for the design of solids management operations, in many cases, irregular sloughing rates result in biological solids production rates that are considerably different from what is predicted theoretically from the observed organic removal rate. Irregular sloughing rates may be due to variations in the influent organic load, rapid climatic changes, or biochemical factors that cause unusually large amounts of sloughing [12]. Low-rate trickling filters in particular demonstrate seasonal variations in sloughing. In these systems, the rate of sloughing is significantly enhanced during warm weather when microbial activity is greater compared with cold weather operation [12,28].

2.5.2.7 Settling tanks for trickling filter effluent. The size of secondary clarifiers following trickling filters is based on the anticipated hydraulic loading. Settled effluent quality is limited primarily by the performance of the biological treatment system and not the settling tanks. The average range of hydraulic loading for secondary clarifiers following trickling filters is 400 to 600 gal/ft^2·day, with peak loadings of 1000 to 1200 gal/ft^2·day. When sizing secondary clarifiers to accompany trickling filters, average and peak flow conditions should be compared. The conditions that lead to the larger settling tank surface area should then serve as the basis for the overall design [28,66].

2.5.3 Rotating biological contactors

A rotating biological contactor (RBC) is an aerobic fixed-film secondary wastewater treatment system used to remove soluble organic matter and, in some instances, ammonia. Rotating biological contactors consist of parallel circular disks attached perpendicularly to a horizontal rotating shaft (Fig. 2.60). The entire assembly is placed into a steel or concrete tank with the shaft positioned slightly above the wastewater surface so that the disks are approximately half immersed [12,37,44].

During wastewater treatment operations, the microbial cells responsible for organic matter removal become attached to the disk surface resulting in the formation of a thin biofilm layer [12,44]. The rotation of the disks through the wastewater provides a constant shear force that establishes a relatively uniform biofilm thickness by inducing

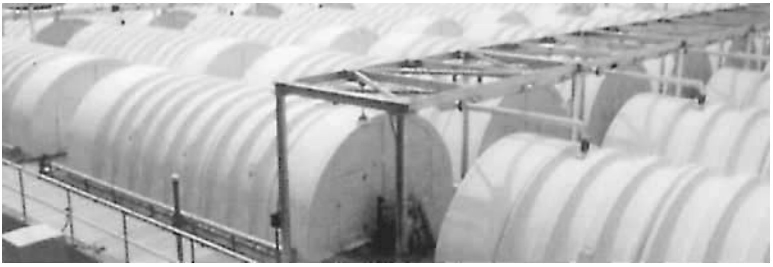


Figure 2.60 A rotating biological contactor system. (*Courtesy of US Filter/Envirex Products.*)

periodic sloughing. The sloughed biological solids are conveyed to a secondary clarifier, where they are separated from the treated wastewater. The biological solids in the clarifier underflow are transported to downstream sludge-treatment processes, whereas the clarified effluent also may receive additional treatment (e.g., filtration, disinfection, etc.) prior to discharge.

A major advantage in employing RBCs in domestic wastewater treatment is that like trickling filters, the microbial populations in fixed-film systems are more resistant to shock loads than those found in suspended-growth secondary wastewater treatment systems (e.g., activated sludge). Another operational advantage of RBCs is the fact that the biofilm is passed through the wastewater rather than the wastewater being passed over the biofilm, as in the case of trickling filters. This design feature alleviates the need for effluent recirculation because complete wetting of the biofilm surface occurs regardless of the influent flow rate [12].

A major disadvantage associated with RBC systems is the absence of reserve treatment capacity to minimize fluctuations in effluent quality [12]. Like trickling filters, once an RBC is constructed, there are few adjustment options available to accommodate significant changes in the characteristics of the influent wastewater. However, adjusting the rotational speed of the disks as well as the degree of disk submergence can alter the intensity of contact between the biofilm and the wastewater and thus help maintain treatment efficiency.

Properly designed, the RBC system can achieve superior performance to suspended-growth biological treatment systems as well as other fixed-film systems due to lower organic loading per mass of biological solids, longer solids detention time, and better control of short-circuiting [44]. In addition, because of low maintenance requirements, RBCs are ideal systems for small communities that cannot afford full-time operators. A typical RBC application for domestic wastewater is shown in Fig. 2.61.

Oxygen is provided to the RBC biofilm by two mechanisms. First, as the disk rotates, part of the biofilm rises above the wastewater surface, allowing atmospheric oxygen to diffuse into it. Second, the liquid turbulence caused by the disk rotation entrains atmospheric oxygen into the wastewater. The liquid turbulence is also responsible for the mixing action that results in a relatively uniform dissolved oxygen concentration throughout the RBC tank [37,44].

RBC disks typically are manufactured of high-density polyethylene (HDPE) or polyvinyl chloride (PVC) and are provided in different configurations or corrugated patterns (Fig. 2.62). Corrugation increases the available surface area for microbial attachment and enhances structural stability [12].

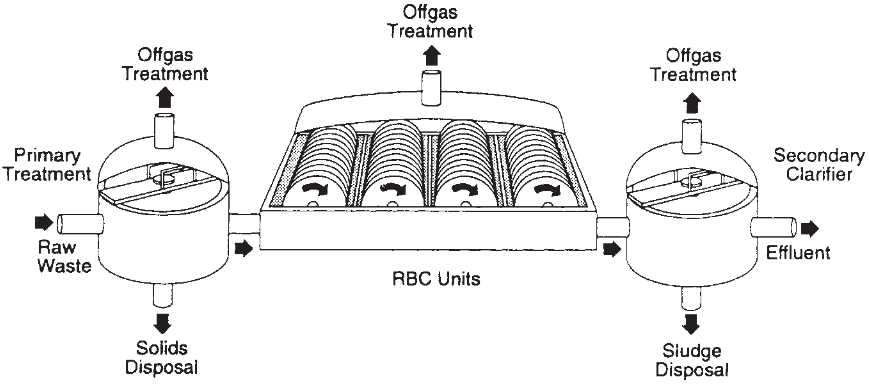


Figure 2.61 Schematic diagram of a rotating biological contactor treatment system.

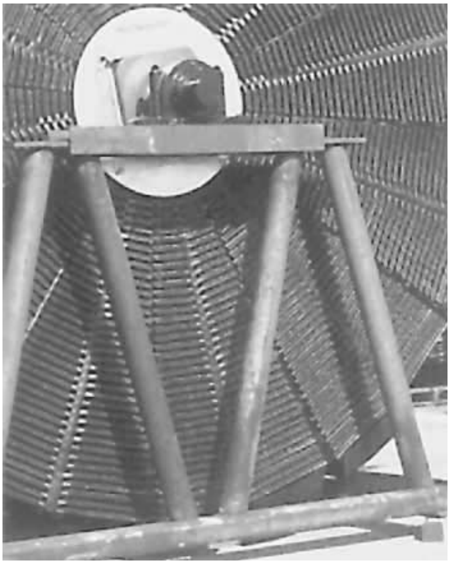


Figure 2.62 Typical corrugated RBC medium. (*Courtesy of US Filter/Envirex Products.*)

Most RBC systems are designed as a series of stages, with each stage consisting of one shaft of disks. The total disk surface area per stage and the number of stages required to meet effluent quality standards are factors that must be determined during the design process. For a given influent wastewater flow rate, increases in both the number of disks and/or the disk surface area will improve the organic removal efficiency [12,44].

In general, full-scale RBCs have a disk diameter of 12 ft (3.6 m) and are supported and rotated by specially designed shafts (Fig. 2.63). Maximum shaft lengths of 27 ft (8.23 m) with 25 ft (7.62 m) occupied

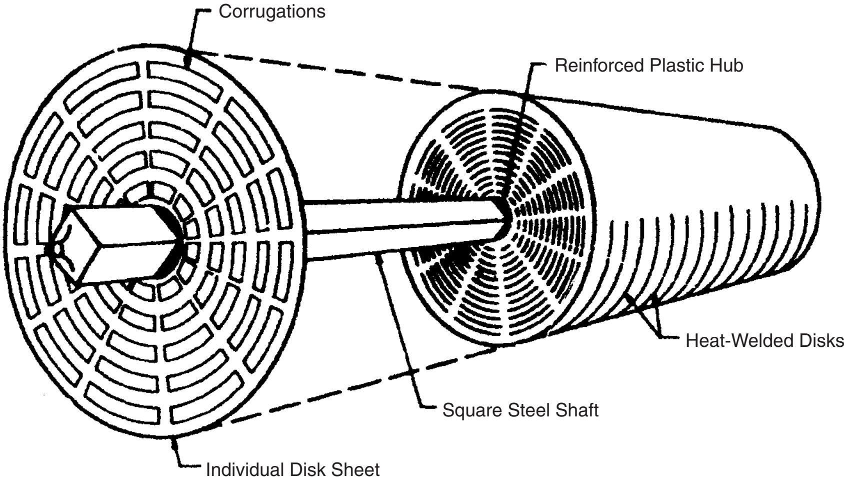


Figure 2.63 RBC disk construction.

by disk medium are a typical stage design, although shaft lengths as small as 5 ft are available [12]. The shape and design details of RBC shafts vary among manufacturers [12].

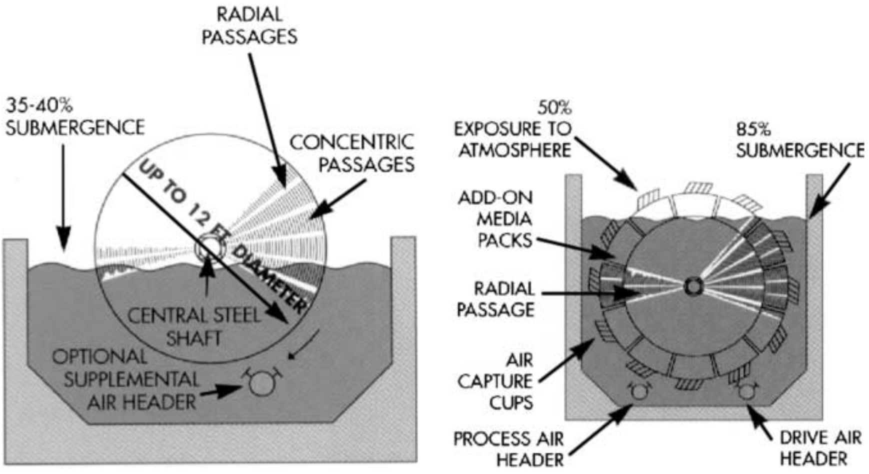
RBC disk media are classified as low (or standard) density, medium density, and high density based on the total disk area per shaft length. Standard density media, defined as disk media with a surface area of 100,000 ft^2 (9290 m^2) per 27 ft (8.23 m) of shaft length, have large void spaces between media layers and typically are employed in the first stage of an RBC system [12]. Medium- and high-density disk media have surface areas of 120,000 to 180,000 ft^2 (11,149–16,723 m^2) per 27 ft (8.23 m) of shaft length and are used in the middle and final stages of an RBC system. Since the microbial activity in RBC systems is associated with disks of defined surface area, operational loading criteria (e.g., hydraulic loading rate, organic loading rate, ammonia loading rate, etc.) are based on the available disk surface area. Readers interested in more detail regarding the design of RBC shafts and disks are referred to the following references [12,37,44].

An important operational factor that influences the performance of an RBC system is the rotational speed of the disks. The disk rotational speed affects several critical RBC system processes, including (1) the rate of aeration, (2) the intensity of contact between the wastewater and the biofilm, and (3) the intensity of mixing in each stage [12]. In addition to affecting these system processes, increasing the rotational speed reduces the mass-transfer limitations associated with the transport of organic matter to the biofilm, resulting in an overall improvement in treatment efficiency [12,44]. However, for treatment of domestic wastewater, the benefit of increasing the disk rotational speed is marginal when the peripheral disk speed is above 2 ft/s (0.61 m/s), which for a 12-ft-diameter disk is equivalent to a rotational speed of 3.2 rev/min [12]. The relationship between rotational speed (rev/min), peripheral (or tip) speed (ft/min), and disk diameter (ft) is given by Eq. (2.54):

$$\text{rev/min} = \frac{V_T}{\pi d} \tag{2.54}$$

where V_T = peripheral disk speed, ft/min (m/s)
d = diameter of disk, ft (m)
π = 3.14 (pi)

To achieve disk rotation, the central shaft normally is attached to a mechanical drive unit. Most specifications for mechanically driven units call for a 7- to 10-hp motor [12,44]. Although the normal rotational speed for a 12-ft-diameter (3.6-m) disk is 1.6 rev/min (which produces a peripheral speed of 1 ft/s), variable-speed drives can be employed for flexibility. In addition to the mechanical drives, rotational air-drive units are also available (Fig. 2.64). The air-drive systems consist of deep plastic cups attached to the perimeter of the disks, an



(a) (b)

Figure 2.64 (*a*) Conventional mechanical drive rotational system. (*b*) Air-drive rotational system for an RBC system. (*Courtesy of US Filter/Envirex Products.*)

air header located beneath the medium, and an air compressor [12]. The discharge of air into the cups creates a buoyant force that causes the shaft to rotate.

In addition to the RBC disk size and rotational speed, other physical design features that affect system performance include (1) tank volume and (2) stage enclosures. Proper sizing of the RBC tank is a critical step in system design because it determines the detention time of the wastewater within each stage [12].

Pilot- and full-scale RBC studies have indicated that no increase in organic matter removal is achieved in municipal wastewater treatment when the specific tank volume is greater than 0.12 gal/ft^2 of disk surface area (0.005 m^3/m^2) [12,37]. Therefore, the minimum RBC tank volume (in gal or m^3) required for treatment of municipal wastewater may be obtained by multiplying the disk surface area in ft^2 (m^2) by 0.12 (0.005). For example, a stage disk area of 100,000 ft^2 (9290 m^2) would have a minimum tank volume of 12,000 gal (45.4 m^3). Based on this volume and a design hydraulic loading rate of 3 gal/ft^2·day (0.12 m^3/m^2·day), a wastewater detention time in this stage would be 0.96 hour. Example 2.23 illustrates the approach for estimating the wastewater detention time in an RBC stage.

Example 2.23 The Drew County Water Reclamation Facility has constructed an RBC system to treat its domestic wastewater flow. The RBC system consists of three stages in series having surface areas of 100,000, 120,000, and 150,000 ft^2, respectively. If the hydraulic loading rate for each stage is limited to 2.5 gal/ft^2·day, estimate the tank volume of each stage and the total wastewater detention time in the RBC system.

solution

Step 1. Estimate the volume of each tank assuming that the specific tank volume is limited to 0.12 gal/ft^2.

Stage 1:

$$\text{Tank volume (gal)} = 100{,}000\ \text{ft}^2 \cdot \frac{0.12}{\text{ft}^2} = 12{,}000\ \text{gal}$$

Stage 2:

$$\text{Tank volume (gal)} = 120{,}000\ \text{ft}^2 \cdot \frac{0.12}{\text{ft}^2} = 14{,}400\ \text{gal}$$

Stage 3:

$$\text{Tank volume (gal)} = 150{,}000\ \text{ft}^2 \cdot \frac{0.12}{\text{ft}^2} = 18{,}000\ \text{gal}$$

Step 2. Estimate the wastewater detention time in each of the stages and sum to find the total detention time.

Stage 1:

$$\text{Detention time (h)} = \frac{\text{stage tank volume}}{\text{flow rate}}$$

$$= \frac{12{,}000\text{ gal}}{\dfrac{2.5\text{ gal}}{\text{ft}^2 \cdot \text{day}} \cdot 100{,}000\text{ ft}^2 \cdot \dfrac{\text{day}}{24\text{ h}}} = 1.15\text{ h}$$

Stage 2:

$$\text{Detention time (h)} = \frac{\text{stage tank volume}}{\text{flow rate}}$$

$$\frac{14{,}000\text{ gal}}{\dfrac{2.5\text{ gal}}{\text{ft}^2 \cdot \text{day}} \cdot 120{,}000\text{ ft}^2 \cdot \dfrac{\text{day}}{24\text{ h}}} \quad 1.15\text{ h}$$

Stage 3:

$$\text{Detention time (h)} = \frac{\text{stage tank volume}}{\text{flow rate}}$$

$$= \frac{18{,}000\text{ gal}}{\dfrac{2.5\text{ gal}}{\text{ft}^2 \cdot \text{day}} \cdot 150{,}000\text{ ft}^2 \cdot \dfrac{\text{day}}{24\text{ h}}} = 1.15\text{ h}$$

$$\begin{aligned}\text{Total detention time (h)} &= \theta_1 + \theta_2 + \theta_3 \\ &= 1.15\text{ h} + 1.15\text{ h} + 1.15\text{ h} \\ &= 3.46\text{ h}\end{aligned}$$

Like the sizing of RBC tanks, employment of an adequate enclosure to protect the disks is critical for maintaining system performance. Reasons for enclosing RBC system stages include

1. Protection of the plastic medium from deterioration due to ultraviolet light
2. Protection of the process from low temperatures
3. Protection of the medium and equipment from damage
4. Controlling algae growth

Segmented fiberglass-reinforced plastic enclosures typically are provided for each stage [12]. In some cases, RBC systems have been housed in a building for (1) protection against cold weather, (2) improved access, or (3) aesthetic reasons.

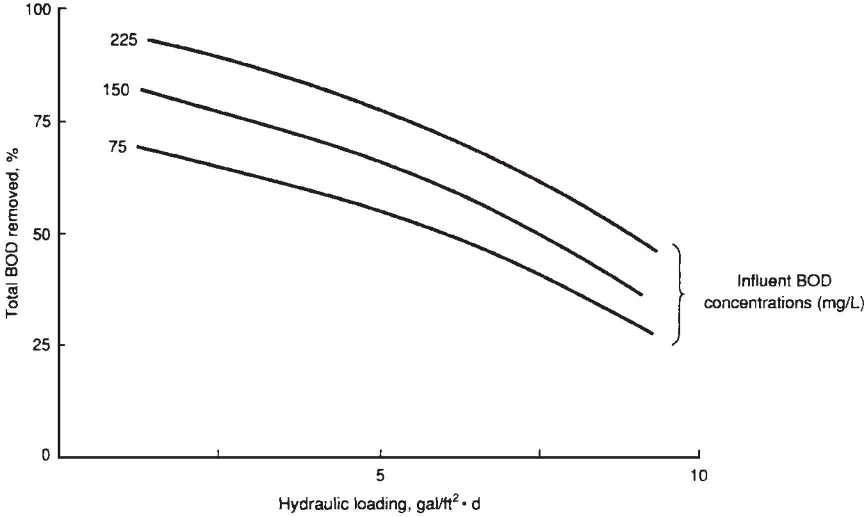


Figure 2.65 Impact of hydraulic loading rate on BOD removal efficiency.

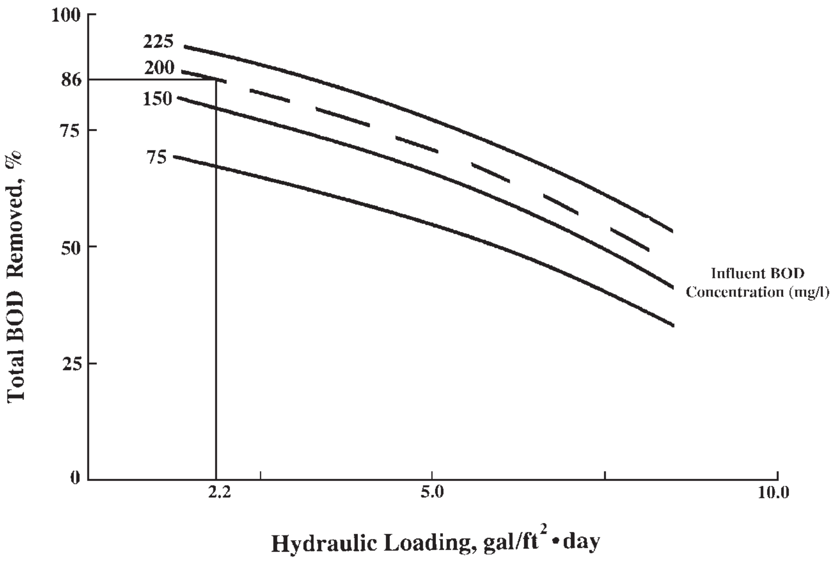
2.5.3.1 RBC process design. Since there is no universally accepted model for designing RBCs, the typical design approach involves evaluating the performance of numerous pilot- and full-scale RBC facilities. Operational data from these facilities typically are aggregated to develop empirical relationships between design parameters. For example, Fig. 2.65 summarizes the BOD_5 removal efficiency data from full-scale RBC facilities as a function of the hydraulic loading rate and influent BOD loading [12,44].

To employ Fig. 2.65 in estimating the permissible hydraulic loading, both the influent and desired effluent BOD_5 concentrations must be known or estimated. Example 2.24 illustrates the use of Fig. 2.65 in RBC process design.

Example 2.24 Carver County Sewer District is considering constructing an RBC secondary wastewater treatment system to treat its 800,000 gal/day domestic wastewater flow (0.8 MGD). If the regulatory authority has established a minimum BOD_5 removal efficiency of 86 percent on all secondary wastewater treatment systems, estimate the hydraulic loading rate and total disk surface area for the RBC system. Assume that the influent flow has an average organic matter concentration of 200 mg/liter BOD_5.

solution

Step 1. Estimate the hydraulic loading rate in gal/ft^2·day from Fig. 2.65 (see the graph on the next page). With a minimum BOD_5 removal of 86 percent and an influent organic matter concentration of 200 mg/liter BOD_5, the design hydraulic loading estimated from Fig. 2.65 is 2.2 gal/ft^2·day.



Step 2. Estimate the design disk surface area required to achieve this organic matter removal efficiency:

$$\text{Disk surface area (ft}^2) = \frac{\text{volumetric flow rate (gal/day)}}{\text{hydraulic loading rate (gal/ft}^2 \cdot \text{day)}}$$

$$= \frac{800{,}000 \text{ gal/day}}{2.2 \text{ gal/ft}^2 \cdot \text{day}} = 363{,}636 \text{ ft}^2$$

At a minimum, the following operational and design parameters must be considered in the design of an RBC system:

1. Staging of the RBC units
2. Loading criteria
3. Effluent characteristics
4. Settling tank requirements

Staging is defined as the arrangement of RBC disks to form a series of independent compartments. The objective of staging is to allow the development of microbial populations optimally suited for the removal of organic matter and/or ammonia [12]. As the wastewater flows through the RBC system, each stage receives an influent flow containing a lower pollutant concentration than the preceding stage, resulting in the attachment of specifically adapted microbial populations to the disks. Typical RBC staging configurations are illustrated in Fig. 2.66.

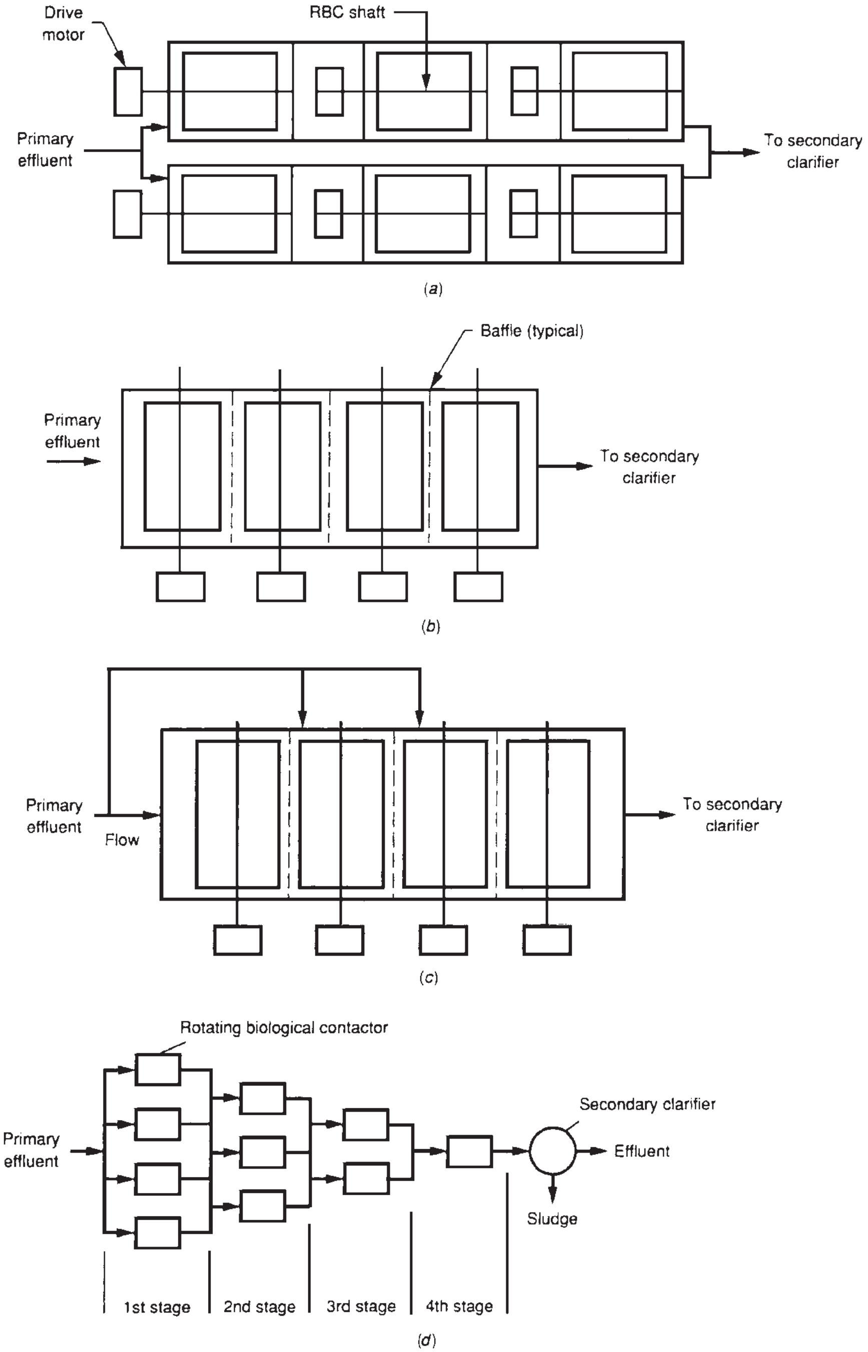


Figure 2.66 Staging patterns in RBC process configurations: (*a*) flow parallel to the shaft; (*b*) flow perpendicular to the shaft; (*c*) step feed; (*d*) tapered feed.

For smaller plants (i.e., less than 3 MGD), RBC drive shafts are oriented parallel to the direction of flow, with disk stages separated by baffles [44]. This stage configuration results in a greater organic loading at the front end of the stage compared with the effluent end. Due to the plug-flow nature of this RBC design, both biofilm thickness and dissolved oxygen concentrations have been found to decrease from feed to effluent end of the stage [12,44].

In larger facilities, shafts are mounted perpendicular to flow, with several stages forming a process train. Introduction of the wastewater perpendicular to the shaft results in a biofilm thickness and dissolved oxygen level that are relatively uniform throughout the stage. For uninterrupted RBC operations, two or more parallel process trains should be installed at the wastewater treatment plant so that stages requiring repair can be isolated.

Since the efficiency of wastewater treatment depends on the mass of attached biofilm, all loading rates for RBC operation are defined in terms of disk area. For example, in the treatment of domestic wastewater, the range of hydraulic loading rates that result in meeting effluent quality standards has been found to vary from 2 to 4 gal of wastewater per square foot of disk surface area per day (i.e., 2–4 $\text{gal/ft}^2\cdot\text{day}$). Values of other design parameters together with effluent wastewater characteristics typically observed in RBC systems are provided in Table 2.33. It should be noted that the organic loading rate may be defined in terms of either total BOD_5 per unit of disk surface area ($\text{lb TBOD}_5/10^3\ \text{ft}^2\cdot\text{day}$) or soluble BOD_5 per unit of disk surface area ($\text{lb SBOD}_5/10^3\ \text{ft}^2\cdot\text{day}$).

Poor effluent quality has been observed when RBC systems are organically overloaded, resulting in low dissolved oxygen, odors, and poor first-stage organic matter removal. The operational problems associated with overloading may be mitigated by removing baffles between the first and second stages to increase both the hydraulic loading rate and oxygen transfer. Other approaches to alleviating overloaded conditions include (1) addition of a supplemental air system, (2) use of step feed, or (3) dilution of the influent organic strength by effluent recirculation [44].

A critical step in RBC design is the estimation of the total disk area required to treat the influent wastewater to effluent quality standards. The total disk area typically is estimated by dividing the average daily flow rate (gal/day) by the design hydraulic loading rate ($\text{gal/ft}^2\cdot\text{day}$). This approach may be modified if information is available suggesting that significant variations in daily flow rate occur during the year. Another approach to estimating the total disk area is based on the organic loading rate. The total organic rate ($\text{lb BOD}_5/\text{day}$) may be divided by the RBC design organic loading rate ($\text{lb BOD}_5/10^3\ \text{ft}^2\cdot\text{day}$) to determine the total disk area.

TABLE 2.33 Design Information for Rotating Biological Contactor*

Item	Treatment level		
	Secondary	Combined nitrification	Separate nitrification
Hydraulic loading rate (gal/ft^2·day)	2.0–4.0	0.75–2.0	1.0–2.5
Organic loading			
lb soluble BOD$_5$/10^3 ft^2·day	0.75–2.0	0.5–1.5	0.1–0.3
lb total BOD$_5$/10^3 ft^2·day	2.0–3.5	1.5–3.0	0.2–0.6
Maximum permissible organic loading rate for the first RBC stage			
lb soluble BOD$_5$/10^3 ft^2·day	4.0–6.0	4.0–6.0	
lb total BOD$_5$/10^3 ft^2·day	8.0–12.0	8.0–12.0	
Ammonia loading (lb NH$_3$/10^3 ft^2·day)		0.15–0.3	0.2–0.4
Hydraulic retention time, h	0.7–1.5	1.5–4.0	1.2–2.9
Effluent BOD$_5$, mg/liter	15–30	7–15	7–15
Effluent NH$_3$, mg/liter		<2.0	1.0–2.0

*Adapted from refs. [12,28].

Regardless of the approach used, after the total disk surface area requirement has been estimated, the influent wastewater BOD$_5$ concentration and the estimated total disk area are used to calculate the first-stage organic loading rate. The first-stage organic loading rate is then compared with the maximum permissible first-stage organic loading rate (approximately 8–12 lb TBOD$_5$/10^3 ft^2 or 4–6 lb SBOD$_5$/10^3 ft^2) to determine if the oxygen-transfer capacity will be exceeded [12,28]. If the first-stage organic loading rate is found to be greater than the maximum permissible first-stage organic loading rate, the RBC design must be modified. Example 2.25 illustrates use of the design information provided in Table 2.33 for estimating the total surface area requirement for an RBC system.

Example 2.25 Addis County Water Reclamation Plant is treating a domestic flow of 2 million gal/day using an RBC system. If the average influent total BOD$_5$ concentration is 175 mg/liter and the desired effluent total BOD$_5$ concentration is 10 mg/liter, estimate the total RBC disk area required both to treat the organic matter and to achieve nitrification. Based on historical records, assume that the maximum peak TBOD$_5$ concentration is 400 mg/liter.

solution

Step 1. From Table 2.33, to achieve an effluent TBOD$_5$ concentration of 10 mg/liter and effluent nitrification, an acceptable organic loading rate is 1.8 TBOD$_5$/10^3 ft^2·day.

Step 2. Estimate the required disk surface area based on the organic loading rate:

$$\text{Disk surface area (ft}^2\text{)} = \frac{\text{BOD}_5 \text{ loading (lb BOD}_5\text{/day)}}{\text{design organic loading rate}\left(\dfrac{\text{lb BOD}_5}{10^3\ \text{ft}^2 \cdot \text{day}}\right)}$$

$$= \frac{175 \text{ mg BOD}_5\text{/liter} \cdot 2 \text{ MGD} \cdot 8.34}{\dfrac{1.8 \text{ lb BOD}_5}{10^3\ \text{ft}^2 \cdot \text{day}}} = 1622 \cdot 10^3\ \text{ft}^2$$

Step 3. Check the first-stage organic loading rate during peak conditions:

$$\text{Organic loading rate}\left(\frac{\text{lb TBOD}_5}{10^3\ \text{ft}^2 \cdot \text{day}}\right) = \frac{\text{peak BOD}_5 \text{ loading (lb BOD}_5\text{/day)}}{\text{disk surface area (}10^3\ \text{ft}^2\text{)}}$$

$$= \frac{400 \text{ mg BOD}_5\text{/liter} \cdot 2 \text{ MGD} \cdot 8.34}{1622 \cdot 10^3\ \text{ft}^2}$$

$$= \frac{4.11 \text{ lb TBOD}_5}{10^3\ \text{ft}^2 \cdot \text{day}}$$

Since the maximum organic loading rate is much less than the maximum permissible first-stage organic loading rate given in Table 2.33, the system will not be overloaded.

Step 4. Check the hydraulic loading rate to determine whether it falls within the range of typical secondary treatment/nitrification combination systems:

$$\text{Hydraulic loading rate}\left(\frac{\text{gal}}{\text{ft}^2 \cdot \text{day}}\right) = \frac{\text{plant flow rate (gal/day)}}{\text{disk surface area (ft}^2\text{)}}$$

$$= \frac{2 \cdot 10^6 \text{ gal/day}}{1622 \cdot 10^3\ \text{ft}^2} = \frac{1.23 \text{ gal}}{\text{ft}^2 \cdot \text{day}}$$

Since the hydraulic rate does fall within the range for combination systems, the given design is acceptable.

Another critical factor that affects effluent quality from an RBC system is the temperature of the influent wastewater. For influent wastewater having a temperature above 55°F (13°C), the impact of influent wastewater temperature on effluent quality has been found to be negligible [12]. However, when the influent wastewater temperature is less than 55°F (13°C), organic matter removal rates may decrease significantly, resulting in poor effluent quality. To compensate for cold temperature effects, the disk surface area must be increased to maintain effluent quality [44].

The operational approach used to compensate for the lower influent wastewater temperatures is to multiply the design temperature hydraulic loading rate by a correction factor. The magnitude of the correction factor depends on the influent wastewater temperature, as illustrated in Fig. 2.67.

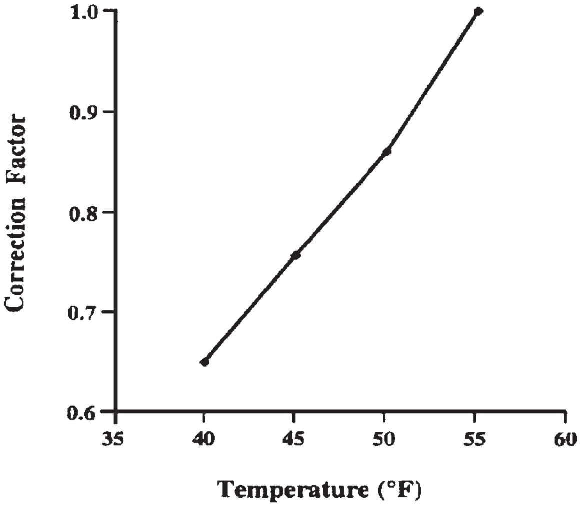


Figure 2.67 Hydraulic flow rate temperature correction factor. (*Adapted from ref. [12].*)

Once the correction factor is determined, the percentage increase in disk surface area needed to maintain system performance may be estimated using Eq. (2.55). For example, from Fig. 2.67, if the wastewater temperature is 42°F (5.6°C), the temperature correction factor is approximately 0.70. Therefore, either the hydraulic loading rate should be reduced by 30 percent from the design hydraulic loading rate or, using Eq. (2.55), the surface area should be increased by 43 percent to maintain treatment efficiency.

$$\text{Percentage change in disk surface area} = \frac{\text{HLR}_i - \text{HLR}_N}{\text{HLR}_N} \cdot 100 \tag{2.55}$$

where HLR_i = design hydraulic loading rate at 55°F (13°C) or above, gal/ft^2 · day (m^3/m^2 · day)

HLR_N = hydraulic loading rate at temperatures below 55°F (13°C), gal/ft^2 · day (m^3/m^2 · day)

Example 2.26 illustrates the approach for estimating the necessary changes in hydraulic loading rate and disk surface area required to treat wastewater at temperatures below 55°F (13°C).

Example 2.26 Banneker County Wastewater Treatment Plant is currently operating its RBC treatment system at a hydraulic loading rate of 2.5 gal/ft^2·day to treat its 2 million gal/day flow rate. Because of a recent increase in groundwater infiltration, the average temperature of the influent wastewater has been reduced from 64 to 48°F (18–9°C). To maintain treatment efficiency, estimate the new hydraulic loading rate and the fractional increase in disk surface area.

solution

Step 1. Estimate the new hydraulic loading rate by multiplying the design hydraulic loading rate by the correction factor. From Fig. 2.67, at a temperature of 48°F (9°C), the correction factor is 0.8.

$$\mathrm{HLR}_N = \mathrm{HLR}_i \cdot \text{correction factor}$$

$$= \frac{2.5\ \text{gal}}{\text{ft}^2\text{day}} \cdot 0.8 = \frac{2.0\ \text{gal}}{\text{ft}^2 \cdot \text{day}}$$

Step 2. Estimate the fractional increase in disk surface area required to achieve the new hydraulic loading rate:

$$\text{Percentage change in disk surface area} = \frac{\mathrm{HLR}_i - \mathrm{HLR}_N}{\mathrm{HLR}_N} \cdot 100$$

$$= \frac{2.5 - 2.0}{2.0} \cdot 100 = 25 \text{ percent}$$

2.5.3.2 Settling tanks following RBC systems. The design and operation of settling tanks following RBCs are similar to those following trickling filters in that all the solids contained in the clarifier underflow are transferred to downstream sludge process facilities. Table 2.34 provides typical design information for settling tanks that follow RBC systems.

To determine the design basis for settling tanks, average and peak conditions should be compared. The conditions leading to the larger settling tank surface area will serve as the basis for the overall tank design [28,66].

TABLE 2.34 Design Information for Settling Tanks Following RBCs*

	Overflow rate ($\mathrm{gal/ft^2 \cdot day}$)		Solids loading rate ($\mathrm{lb/ft^2 \cdot h}$)	
	Average	Peak	Average	Peak
Secondary effluent	400–800	1000–1200	0.8–1.2	2.0
Nitrified effluent	400–600	800–1000	0.6–1.0	1.6

NOTE: Settling tanks typically have a depth of 10 to 15 ft.
*Adapted from ref. [12].

2.5.3.3 Biological solids generation in RBCs. As in trickling filters, biological solids are generated in RBCs by the continual sloughing of the biofilm. The average daily solids production can be estimated by multiplying the mass removal rate of BOD_5 from the treatment system (lb BOD_5/day) by the observed microbial yield Y_{obs} (lb total solids per lb BOD_5 removed). Unfortunately, limited data are available on RBC biological solids production. Microbial yields from RBC units have been found to vary from 0.40 to 0.82 lb total solids per pound of BOD_5 removed [12,60]. Example 2.27 illustrates an approach for estimating the daily biological solids production from RBC operations.

Example 2.27 The Bethune County Wastewater Treatment Plant is employing an RBC system to treat its 5 million gal/day domestic sewage flow rate. If the influent BOD_5 concentration is 250 mg/liter and the effluent BOD_5 is 25 mg/liter, estimate the following:

1. Daily solids production rate (lb/day)
2. Volume of sludge flow from the secondary clarifier (ft^3/day)

Assume that the following data apply:

1. Observed yield Y_{obs}: 0.5 lb total solids/lb BOD_5 removed
2. Total solids content (P_S) of clarifier underflow: 3 percent
3. Specific gravity of clarifier underflow (S_{BS}): 1.03

solution

Step 1. Estimate the daily rate of BOD_5 removed (lb/day):

$$\left(\frac{\text{lb BOD}_5}{\text{day}}\right)_{\text{removed}} = [\text{BOD}_5\ (\text{influent}) - \text{BOD}_5\ (\text{effluent})] \cdot \text{flow rate (MGD)} \cdot \frac{8.34\ \text{lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= (250\ \text{mg/liter} - 25\ \text{mg/liter}) \cdot 5.0\ \text{MGD} \cdot \frac{8.34\ \text{lb}}{\text{MG (mg/liter)}} = \frac{9382.5\ \text{lb BOD}_5}{\text{day}}$$

Step 2. Estimate the daily rate of total solids (TS) produced from the multistage RBC system:

$$\text{Solids production rate}\left(\frac{\text{lb}}{\text{day}}\right) = Y_{\text{obs}}\left(\frac{\text{lb BOD}_5}{\text{day}}\right)_{\text{removed}}$$

$$= \frac{0.5\ \text{lb TS}}{\text{lb BOD}_5} \cdot \frac{9382.5\ \text{lb BOD}_5}{\text{day}}$$

$$= \frac{4691.3\ \text{lb TS}}{\text{day}}$$

Step 3. Estimate the volume flow rate of biological solids from the secondary clarifier using Eq. (2.20):

$$\text{Volume flow rate}\left(\frac{\text{ft}^3}{\text{day}}\right) = \frac{W}{\rho S_{BS} P_S}$$

$$= \frac{4691.3 \text{ lb/day}}{\dfrac{62.4 \text{ lb}}{\text{ft}^3} \cdot 1.03 \cdot 0.03}$$

$$= \frac{2433 \text{ ft}^3}{\text{day}}$$

2.5.4 Combination suspended-growth/fixed-film systems

A large number of secondary treatment systems can be developed by combining suspended-growth systems with fixed-film treatment systems. The principal reason for combining processes is that they provide the stability and resistance to shock loads characteristic of fixed-film systems with the higher-quality effluent and operational flexibility characteristic of suspended-growth systems. Examples of the more common combination systems include (1) activated biofilters, (2) trickling filters followed by a solids contact reactor, (3) roughing trickling filter followed by an activated-sludge process, (4) biofilter followed by an activated-sludge process, and (5) trickling filter followed by an activated-sludge process. Figure 2.68 provides schematic diagrams of these combination systems. In all cases, the first stage in the combination system acts as a roughing process in that it functions to reduce the organic loading to the following secondary treatment system.

The biological solids production from combination systems can be estimated using the methods described previously for each individual process. The solids production from each individual process is summed to generate the total biological solids production.

2.5.5 Septage generation and management

Septage generation rates will vary from month to month depending on the amount of domestic wastes generated as well as weather and geography. For example, frozen ground may limit pumping of septic tanks to warmer months, whereas high groundwater conditions can cause septic tank effluent to rise to the ground surface, requiring an increase in the frequency of pumping.

The most accurate method for estimating the septage generation rate is to collect disposal information from the records of local septage

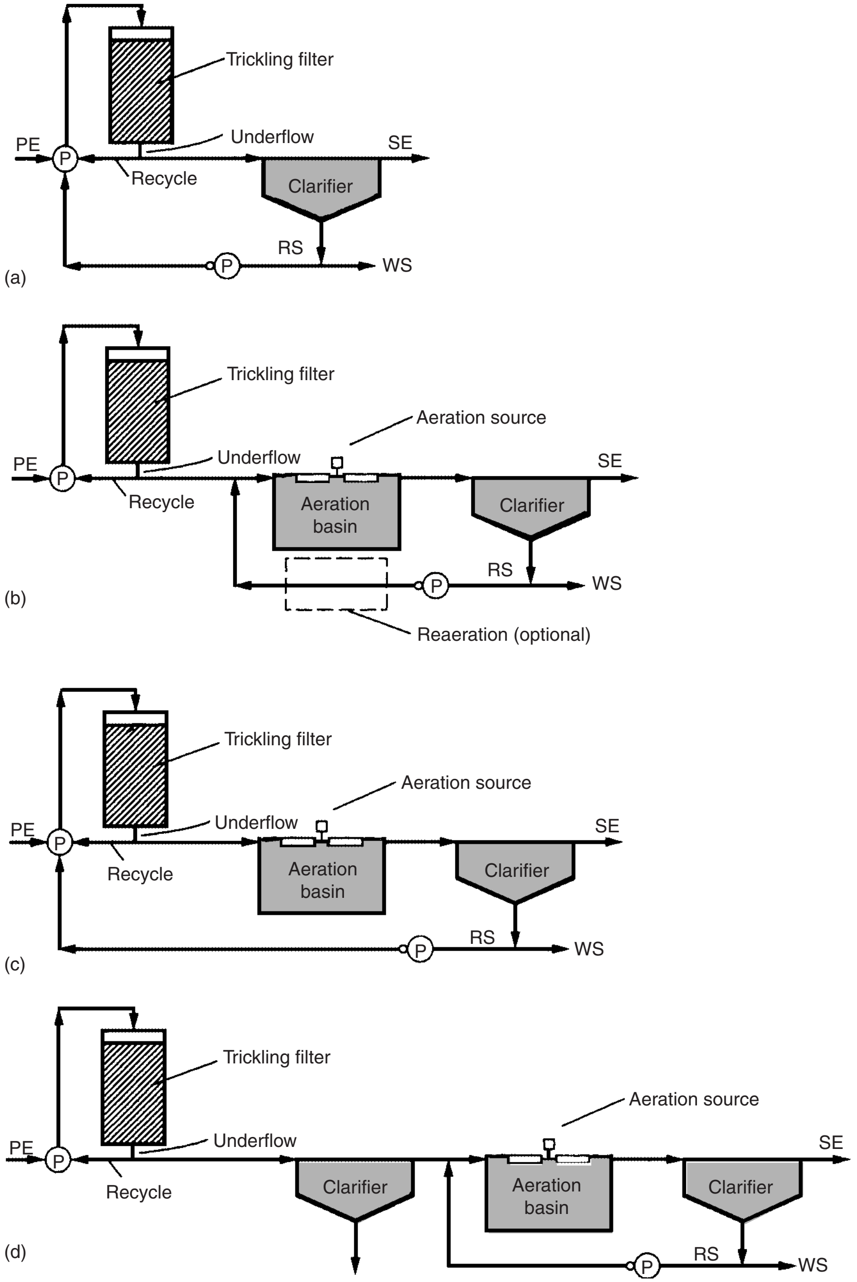


Figure 2.68 Combination secondary wastewater treatment systems: (*a*) activated biofilter, (*b*) trickling filter solids contact and roughing filter/activated sludge, (*c*) biofilter/activated sludge, and (*d*) series trickling filter/activated sludge. PE = primary effluent, SE = secondary effluent, P = pump, RS = return sludge, WS = waste sludge [28].

TABLE 2.35 Peaking Factors Used to Estimate Septage Flow*

	Ratio of peak monthly to mean monthly septage volumes
Essex, Connecticut	2.0
Salem, New Hampshire	1.3
Lebanon, Ohio	1.8
Winston-Salem, North Carolina	1.8
Heisted, Norway	1.7
Lillehammer, Norway	1.9

*Adapted from ref. [34].

haulers, treatment plants receiving septage, or independent septage treatment plants [34]. This approach takes into account the variation in septage generation rates and thus provides data specific to the municipality. Moreover, the use of *peaking factors* (defined as the ratio of the maximum to average septage quantity received) allows the design engineer to estimate the range of septage flow conditions to be expected. Table 2.35 lists peaking factors from various septage treatment facilities in the United States and Norway [34]. Although little data exist on weekly and daily peaking factors, various planning studies in the United States have recommended weekly peaking factors ranging from 1.8 to 3.6 and daily peaking factors ranging from 4.0 to 4.8 [28,34].

Another approach to estimating the septage generation rate is to multiply the number of septic tanks in a municipality by an assumed average septic tank volume and pumpout frequency (normally 3–5 years). Although this method generates a less reliable estimate of septage generation than using actual septage disposal records, it leads to a conservative estimate that may be used for facility design purposes. When no site-specific data are available, the septage generation rate also may be estimated by multiplying the number of residents in a municipality by an assumed per capita septage generation rate (typically 60 gallons per capita per year). Example 2.28 illustrates the latter two approaches for estimating septage generation rates.

Example 2.28 State census records indicate that there are 4200 persons living within Baldwin County. The Baldwin County Public Health Department estimates that there are approximately 850 septic tanks within its jurisdiction. Assuming that the average septic tank has a volume of 1000 gal and a pumpout frequency interval of 4 years, estimate the annual septage generation rate based on both the number of septic tanks in the county and the number of residents. Assume a per capita septage generation rate of 60 gal/person-year.

solution

Step 1. Estimate the septage generation rate based on the number of septic tanks in the county:

$$\text{Septage generation rate (gal/year)} = \frac{850 \text{ septic tanks} \cdot \dfrac{1000 \text{ gal}}{\text{septic tanks}}}{\text{4-year pumping interval}}$$

$$= \frac{212{,}500 \text{ gal}}{\text{year}}$$

Step 2. Estimate the septage generation rate based on the number of residents in the county:

$$\text{Septage generation rate (gal/year)} = 4200 \text{ persons} \cdot \frac{60 \text{ gal}}{\text{person-year}}$$

$$= \frac{252{,}000 \text{ gal}}{\text{year}}$$

Once the septage generation rate has been determined, alternatives for the treatment and disposal of septage must be identified. Most of the septage treatment and disposal alternatives fall into one of the following categories:

1. Land application
2. Treatment at a wastewater treatment plant
3. Treatment at an independent septage treatment plant

Advantages and disadvantages of each of these categories are presented in Table 2.36.

Land application of septage is the most common means of septage disposal in the United States [46,50]. However, availability of suitable land with adequate buffer separation from residential areas is limited in many urban and suburban areas, and therefore, this septage disposal option is usually practiced in rural communities.

Disposal at an existing wastewater treatment plant is a viable and economical option if the facility is reasonably close to the source and has sufficient capacity to handle septage loadings. Independent septage treatment plants are the most costly of the three septage treatment alternatives, and municipalities generally consider them only if the first two options are not technically or economically feasible [34,46].

2.5.5.1 Land application of septage. The 40 CFR Part 503 rule, (Subchapter O) contains the minimum requirements for the applica-

TABLE 2.36 Approaches to Septage Treatment and Disposal*

Method	Advantages	Disadvantages
Land application	1. Simple and economical 2. Recycles organic material and nutrients to the land 3. Low energy use	1. Need for holding facility during periods of frozen or saturated soil 2. Need for relatively large, remote land area
Treatment at wastewater treatment plants	1. Most plants are capable of handling some septage 2. Centralizes waste treatment operations	1. Potential for plant upset if septage addition is not properly controlled 2. Increased solids handling and disposal requirements
Treatment at independent septage treatment plants	1. Provides regional solution to septage management	1. High capital and operation and maintenance costs 2. Requires high skill levels for operation

*Adapted from ref. [34].

tion of domestic septage to land used infrequently by the general public [39,52]. Such sites, referred to as *nonpublic contact sites,* include agricultural fields, forest land, and reclamation sites [39,52]. For septage application to land where public exposure potential is high or for application of residual solids from an independent septage treatment facility, the septage is subject to the more restrictive wastewater treatment plant biosolids provisions of the 40 CFR Part 503 regulations. Federal regulations also limit the land application of septage under the following conditions: (1) endangered species are threatened, (2) the ground is flooded, frozen, or snow covered, and (3) application area is within 10 m (33 ft) of wetlands or surface waters [52].

Septage may be applied to land using either surface or subsurface methods. The various methods of land applying septage are summarized in Tables 2.37 and 2.38. Although domestic septage does not have to meet the same pollutant concentration requirements as wastewater biosolids before being applied to land, federal regulations (40 CFR Part 503) do require that one of the following three options be implemented to reduce vector attraction:

1. Subsurface injection
2. Incorporation (surface application followed by plowing within 6 hours)

TABLE 2.37 Surface Application Methods for Septage Land Treatment*

Method	Advantages	Disadvantages
Spray irrigation	1. Can be used on steep or rough land 2. Minimizes disturbance of soil by trucks	1. Large land area required 2. Possible pathogen dispersal and vector attraction if not stabilized 3. Storage tank or lagoon required during periods of wet or frozen ground 4. Potential of nozzle plugging
Ridge and furrow irrigation	1. Lower power requirements and odor potential than spray irrigation	1. Limited to 0.5 to 1.5 percent slopes 2. Storage lagoon required
Hauler truck spreading	1. Same truck can be used for transport and disposal	1. High odor potential 2. Storage tank or lagoon required during periods of wet or frozen ground 3. Slope may limit vehicle operation 4. Truck weight causes compaction of soil
Farm tractor and wagon spreading	1. Allows for application of liquid or septage solids 2. Increases opportunities for application compared to hauler tank spreading	1. High odor potential 2. Storage tank or lagoon required 3. Requires additional equipment

*Adapted from refs. [39,52].

3. Alkali stabilization (maintaining septage at a pH of 12 or greater for 30 minutes prior to land application)

The simplest septage land-application method consists of opening a valve on the septage hauler truck and driving across the land-application site. In this approach, a splash (or spreader) plate is employed to improve the septage distribution onto the soil surface. The septage typically is discharged through a simple screen or basket located on the truck between the outlet pipe and the spreader plate that prevents nondegradable materials such as plastics and other objectionable trash from being applied to the soil.

To meet federal vector attraction reduction requirements, septage that is applied to land using this approach must be stabilized prior to disposal [46,50]. Stabilization is a treatment method designed to

TABLE 2.38 Subsurface Application Methods for Septage Land Treatment*

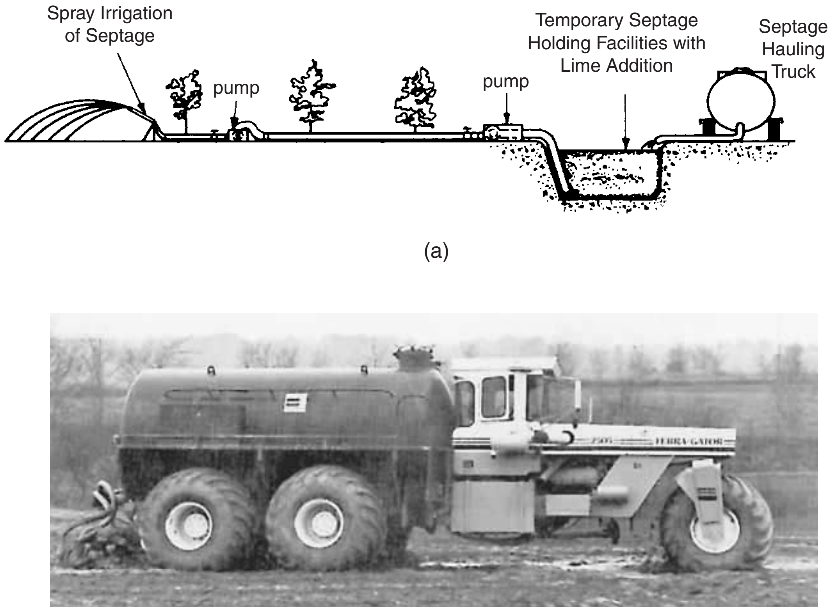
Method	Advantages	Disadvantages
Plow and furrow cover	1. Minimal odor and vector attraction compared with surface application 2. Satisfies EPA criteria for reduction of vector attraction	1. Slope may limit vehicle operation 2. Storage tank or lagoon required during periods of wet or frozen ground
Subsurface injection	1. Minimal odor and vector attraction compared with surface application 2. Satisfies EPA criteria for reduction of vector attraction	1. Slope may limit vehicle operation 2. Storage tank or lagoon required during periods of wet or frozen ground 3. Specialized equipment and vehicle may be costly to purchase and to maintain

*Adapted from refs. [46,52].

reduce levels of pathogenic organisms, lower the potential for putrefaction, and reduce odors. Septage may be stabilized within the septage hauler truck by adding sufficient lime or other alkali material to raise the pH to at least 12 for a minimum of 30 minutes. Typically, this requires 20 to 25 lb of lime (as CaO or quicklime) per 1000 gal of septage, although septage characteristics and lime requirements vary widely.

The direct surface application of septage from the hauler truck to soil offers the least flexibility and control from a management perspective. In addition, soil may become compacted, and trucks not designed for off road use may have difficulty driving on the site. This approach normally is used in small rural land-application operations where little environmental or human health risk is likely. Another approach to septage land application is to use a manure spreader or a special liquid-waste application vehicle that removes screened septage from a holding tank and injects it on or below the soil surface (Fig. 2.69). If the septage is incorporated into the soil by plowing or subsurface injection, lime stabilization is unnecessary [46,50].

A third approach to land application is to pretreat the septage (minimum of screening) during discharge from the septage hauler truck into a holding/mixing tank. The septage is lime-stabilized within the holding tank and then sprayed on the land surface using commercially available sludge application equipment. Although lime stabilization reduces odors and potentially eliminates the need to incorporate septage into the soil, good practice dictates that septage should be incorporated into soil. Whether lime is added to the sep-



(a)

(b)

Figure 2.69 (*a*) Schematic diagram of surface application of lime-stabilized septage. (*b*) Photograph of a vehicle used for subsurface injection of septage. (*Courtesy of Ag-Chem Equipment Co., Inc.*)

tage hauler truck or to a holding/mixing tank, the pH must be measured to ensure that a pH of 12 is achieved and maintained for 30 minutes.

The maximum annual volume of domestic septage applied to all but land-reclamation sites depends on the septage nitrogen content, the nitrogen requirements of the crop, and the crop yield. This septage application rate, known as the *agronomic rate,* may be estimated using procedures described in Chaps. 5 and 6.

2.5.5.2 Septage treatment at wastewater treatment plants. Disposal at the local municipal wastewater treatment plant is often a convenient and environmentally sound method for septage management. The majority of wastewater treatment plants that accept septage do so either at the headworks of the plant or at a manhole upstream of the plant. Some wastewater treatment plants that treat large volumes of septage provide receiving and equalization facilities to control septage additions at a relatively constant rate. The three approaches that can be used for treatment of septage at wastewater treatment plants include

1. Treatment within the liquid treatment system
2. Treatment within the sludge handling train
3. Combinations of the above

In general, wastewater treatment plants with primary clarifiers are best suited to receive septage in the liquid treatment system because a significant portion of the suspended solids can be removed prior to secondary waste treatment operations. Septage introduced into the sludge or biosolids handling train is normally screened and degritted prior to being transferred to aerobic or anaerobic digesters, sludge holding tanks, or gravity thickeners. This approach allows the blending of septage with primary or secondary sludge within the thickening, stabilization, or dewatering process. The advantages and disadvantages for the various options for handling septage at wastewater treatment plants are summarized in Table 2.39.

Estimating the amount of septage that can be managed effectively at a wastewater treatment plant is complex. High ratios of septage to wastewater flow may adversely affect plant operations. Figure 2.70 illustrates an approach for determining the allowable rates of septage addition based on the current versus design wastewater treatment plant loadings. Figure 2.70 assumes that a holding tank is provided and that septage is added to the sewage flow on a semicontinuous basis [46,50].

From Fig. 2.70, a conventional waste activated-sludge system (with primary clarification) with a design capacity of 2 million gal/day and operating at 50 percent of design capacity should be capable of receiving a septage flow of 1.4 percent of plant capacity, or 28,000 gal/day. Allowable septage volumes may be reduced from this level due to septage characteristics, treatment plant operations, or sewage flow patterns. In all cases, a safety factor should be used in estimating allowable septage discharge volumes.

The adverse impacts of septage addition on wastewater treatment plant operations will increase significantly if septage is discharged as a slug load. For example, a 1000-gal load of septage adds an organic load equivalent to approximately 35,000 gal of sewage [28,34,50]. For unequalized septage additions to a wastewater treatment plant, the allowable septage addition rates determined from Fig. 2.70 should be divided by 5 to avoid plant upset [34,50]. In addition to protection from slug loads, many sewage treatment plants or independent septage treatment facilities require that grab samples of “random” septage loads be collected and analyzed to deter the discharging of wastes that would cause problems with treatment plant operation, performance, or permit compliance [34].

TABLE 2.39 Summary of Options for Handling Septage at Wastewater Treatment Plants*

Method	Advantages	Disadvantages
Septage addition to upstream sewer manhole	1. Very simple 2. May provide dilution of septage prior to reaching wastewater treatment plant 3. Low capital and O&M costs	1. Odor potential near manhole 2. Potential to accumulate grit and debris in sewer 3. Only feasible for large sewers
Septage addition to wastewater treatment plant headworks	1. Simple 2. Allows control of septage discharge by plant operators	1. May affect wastewater treatment plant operations if septage addition is uncontrolled 2. Increases odor potential at treatment plant
Septage addition to biosolids or sludge handling process	1. Reduces loading to liquid steam processes 2. Eliminates the potential for affecting effluent quality	1. May affect biosolids or sludge treatment processes 2. If not screened and/or degritted septage may cause clogging and wear on pumps 3. Expensive due to receiving station costs
Septage addition to both liquid stream and sludge handling processes	1. Provides more concentrated sludge for processing 2. Increases flexibility of subsequent processing steps	1. Requires increased operations for septage pretreatment at receiving station 2. Expensive due to receiving station costs

*Adapted from refs. [34,46,50].

When adding septage to the solids handling train, allowable loadings must be estimated based on site-specific process information. Solids processes to which septage typically is added include thickening, aerobic or anaerobic digestion, dewatering, chemical stabilization, and composting [34]. The normal procedure for estimating the maximum amount of septage that can be added to the solids treatment train involves comparing the current versus design hydraulic and solids loading to a particular process. After estimation of the available process capacity, conservative estimates of the volumes of septage that can be processed without exceeding design capacity are determined [34,46].

In addition to increasing both the hydraulic and organic loads, septage addition to a wastewater treatment plant will increase the oper-

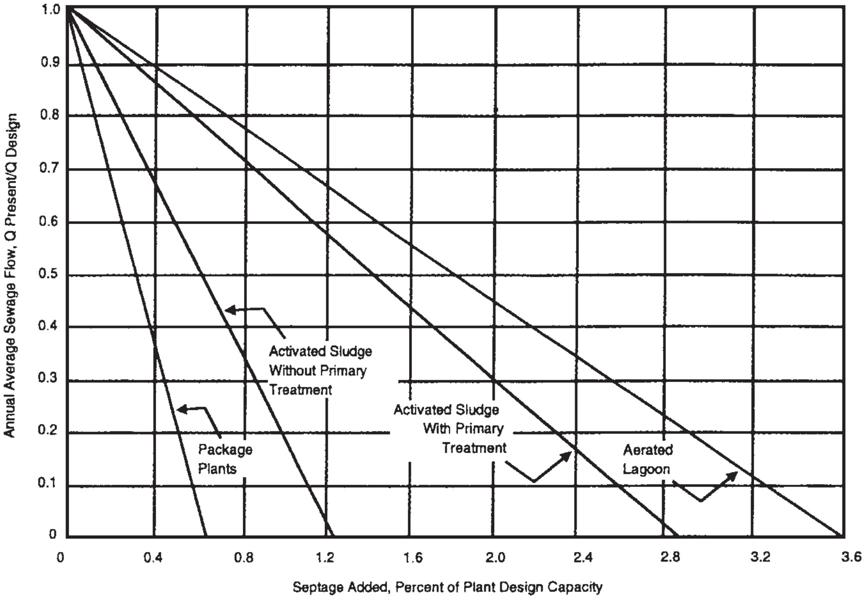


Figure 2.70 Permissible septage addition rates to wastewater treatment plants [46,50].

ation and maintenance requirements as well as the administrative tasks associated with recordkeeping and billing of septage haulers. Monitoring requirements at the wastewater treatment plant are not likely to increase significantly with septage addition because a well-operating wastewater treatment plant will employ a data-collection program sufficient to maintain compliance. During peak septage loadings, aeration basin dissolved oxygen concentrations should be monitored frequently to ensure that they are at least 2.0 mg/liter. Other operational data that should be monitored to assess the impact of septage addition on wastewater treatment plant performance include biosolids or sludge production, chemical and power consumption, cake solids content from dewatering, and grit/screening volumes. A summary of the impacts of septage addition on wastewater treatment plant operations is provided in Table 2.40.

2.5.5.3 Septage treatment at independent septage treatment plants. For situations where land is unavailable and/or wastewater treatment plants are too far away or of insufficient capacity, use of independent septage treatment facilities may be warranted. The waste treatment processes found at independent septage treatment facilities can be quite complex and are comparable with operations found at conventional sewage treatment plants [34]. Moreover, when septage is treated at an

TABLE 2.40 Impacts of Septage Addition to a Wastewater Treatment Plant*

- Increased volume of screenings and grit requiring disposal
- Increases odor emissions from headworks
- Scum accumulation in clarifiers
- Increased organic loadings to biological processes
- Potential odor and foaming problems in aeration basins
- Increased loadings to biosolids or sludge handling processes
- Increased biosolids volumes requiring final disposal
- Increased housekeeping requirements

*Adapted from refs. [34,50].

independent septage treatment facility, the residual solids generated at the facility are no longer considered septage and are subject to the more restrictive wastewater biosolids provisions of 40 CFR Part 503 [50,52].

At independent septage treatment plants, the first treatment steps typically consist of the removal of large solids and grit using bar screens and a grit chamber (these processes are collectively called *pretreatment*). Following pretreatment, septage normally is stabilized using one of the treatment processes identified in Table 2.41.

Lime stabilization of septage is an attractive treatment process because it has the benefit of both stabilizing and conditioning septage prior to dewatering. Moreover, lime stabilization followed by a sand drying bed dewatering process represents one of the simplest and most cost-effective approaches to treating septage prior to final disposal [34,46]. Aerobic and anaerobic digestion are also effective processes for stabilizing septage. However, these processes are expensive to build and operate and thus are found at facilities typically treating at least 10,000 gal of septage per day [34,46].

Composting is also an attractive septage stabilization process, particularly when bulking agents (e.g., wood chips) are plentiful and the demand for the finished composted product is high. Although septage is amenable to composting as either a liquid or a dewatered cake, bulking agent requirements for liquid septage composting are substantial. Unless a low-cost bulking agent is available, septage should be dewatered prior to composting [34,50]. Typical composting processes used to stabilize septage include the windrow and aerated static pile systems (Fig. 2.71). Operational descriptions of these two composting processes are provided in Chap. 3.

After stabilization by any process other than composting, septage may be applied to land in liquid form or may be dewatered prior to

TABLE 2.41 Septage Stabilization Processes*

Method	Advantages	Disadvantages
Alkali stabilization (lime stabilization)	1. Very simple; minimal operator attention 2. Provides temporary reduction in sulfide odors 3. Low capital and O&M costs 4. Reduces EPA site restrictions requirements for land application	1. Increases mass of solids to be disposed of 2. Lime feed and mixing equipment require regular maintenance 3. Lime handling may cause dust problems
Aerobic digestion	1. Relatively simple 2. Provides reduction in odors 3. Low capital and O&M costs	1. High power costs 2. Large tanks or basins required 3. Cold temperatures require much longer digestion periods
Anaerobic digestion	1. Generates methane gas 2. Provides reduction in odors	1. Requires skilled operators 2. High maintenance requirements 3. High capital costs
Composting	1. Final product is potentially marketable 2. High odor potential 3. High operating costs	1. Requires skilled operators

*Adapted from refs. [34,50,52].

composting and/or final disposal. In general, septage is difficult to dewater, and the demand for conditioning chemicals (e.g., lime, ferric chloride, and polymers) is high and variable from load to load. Unit operations typically used for dewatering conditioned septage include (1) screw presses, (2) centrifuges, (3) filter presses, (4) gravity- and vacuum-assisted drying beds, and (5) sand drying beds [34]. When evaluating septage dewatering options, it must be remembered that in mechanical dewatering operations (e.g., screw presses, centrifuges, filter presses, etc.), a significant liquid side stream is generated that must be managed. Management options for the liquid side stream include (1) treatment and discharge to the land or a surface water or (2) disposal at a wastewater treatment plant. Design and operational details of the various dewatering processes may be found in Chap. 3.

2.5.5.4 Septage receiving stations. Septage receiving facilities are a critical component for septage handling at municipal wastewater treatment plants, independent septage treatment plants, and in many

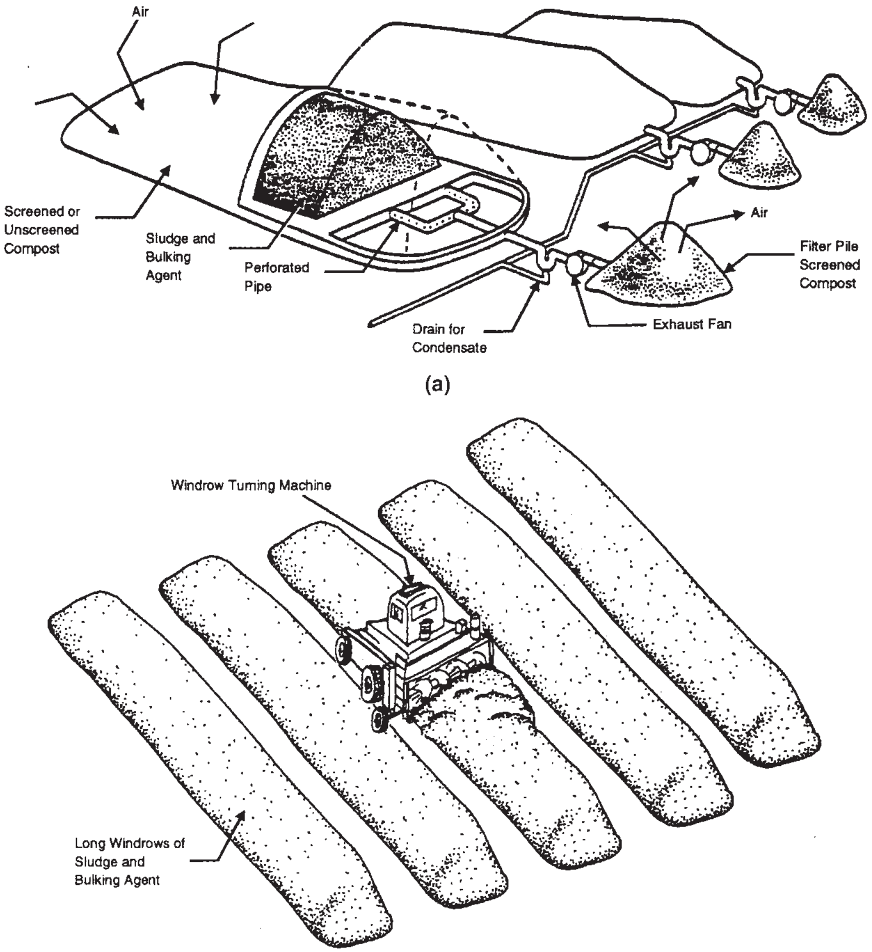


Figure 2.71 Schematic diagram of (*a*) aerated static pile composting system and (*b*) windrow composting system.

cases, septage land-treatment sites. The primary functions of a septage receiving station are (1) to transfer septage from the hauler trucks, (2) preliminary treatment of septage (i.e., screening and grit removal), and (3) storage and equalization of septage flows. In addition to these functions, the septage receiving station may be the repository of septage records (i.e., septage sources, septage volumes, and sampling data).

The design of the septage receiving station will vary depending on the amount of anticipated septage flow, septage hauler truck design, type of preliminary treatment to be provided, odor control, downstream treatment processes, and final disposal alternative. Some examples of septage receiving station designs are provided in Fig. 2.72.

The five principal elements of a septage receiving station are (1) the dumping station, (2) screening, (3) grit removal, (4) storage/equalization, and (5) odor control. The dumping station is the initial point of reception of septage at a receiving facility. It should have a sloped ramp to tilt the hauler truck for complete drainage and to facilitate hosing down of spillage to a central drain.

Figure 2.73 illustrates a receiving station at which septage is being transferred from a smaller to a larger hauler truck. Hoses and other wash-down equipment should be provided and should be located conveniently at the dumping station to facilitate cleanup by each individual hauler. In colder climates, hot steam equipment should be provided for thawing frozen valves, hose lines, etc.

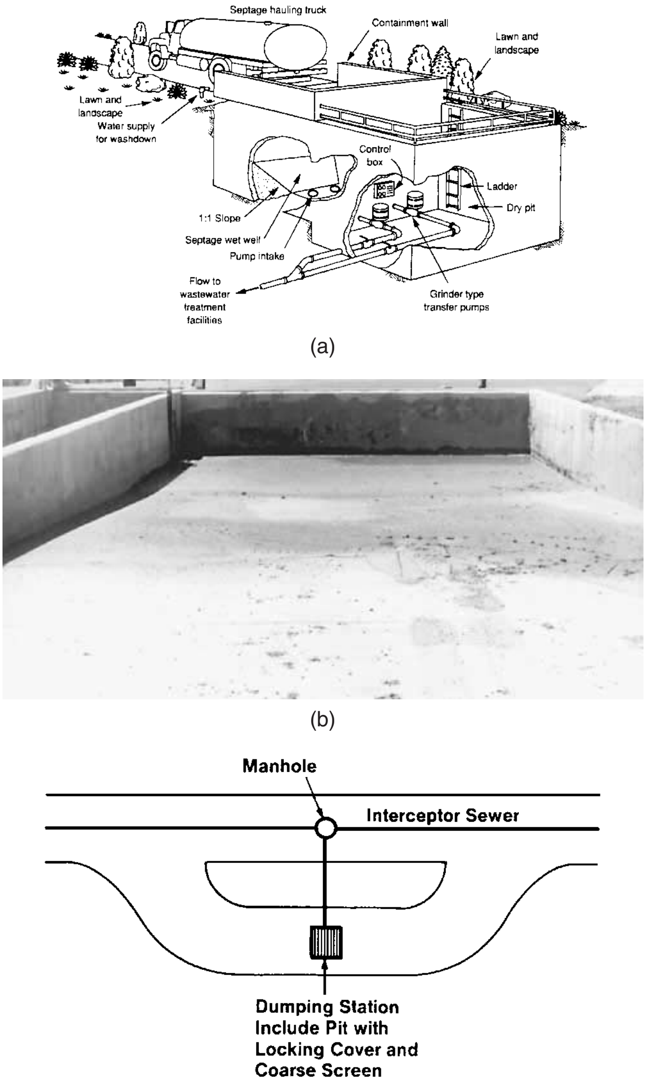
Septage should be discharged from the hauler truck through a hose extending from the rear of the vehicle to the dumping station. The hose should be connected to a quick-disconnect discharge tube in the dumping station [34,46]. The dumping station should not be designed to allow tank trucks to discharge septage without a hose connection.

The anticipated amount of septage to be received and handled at a dumping station and the rate at which it passes through the pretreatment facility must be estimated accurately when designing the septage receiving station. The limiting factor affecting a dumping station's peak flow capacity may be the number of discharge points (i.e., unloading docks and hose connections). Multiple discharge points should be considered where high traffic is anticipated during peak handling periods.

Septage discharged to the receiving station may be transferred directly to a sewage-collection system, larger hauler truck, wastewater treatment plant headworks, holding tank, or septage lagoon. Unless directly discharged to a sewer, septage normally undergoes pretreatment (screening and degritting) prior to being treated by subsequent waste treatment operations.

2.5.5.5 Septage pretreatment. Septage generally will contain various forms of untreatable debris (e.g., rags, plastics, cans, stones, etc.). Such debris typically is separated from the liquid septage by coarse bar screens. Figure 2.74 is a photograph of bar screens being manually cleaned of debris. To minimize health and safety concerns of operators, mechanically cleaned bar screens are desirable for all septage/wastewater handling facilities.

The screenings from septage contain substantial amounts of moisture and should be dewatered prior to disposal at a municipal landfill. Smaller treatment plants receiving septage typically use a drained-screw conveyor to dewater screenings [34,46].



(a)

(b)

(c)

Figure 2.72 (*a*) Schematic diagram of a septic receiving station typically found at independent septage treatment facilities or municipal wastewater treatment plants. (*b*) Photograph of containment wall at a septage receiving station. (*c*) Septage receiving station associated with discharge to a municipal sewer [34].

In addition to large solid material, septage will contain substantial amounts of grit. Grit consists of solid materials such as sand, gravel, cinders, etc. In septage, these solid materials normally are enmeshed in a lighter-weight organic matter (e.g., grease and scum), making separation difficult [34]. Grit from septage usually is removed through

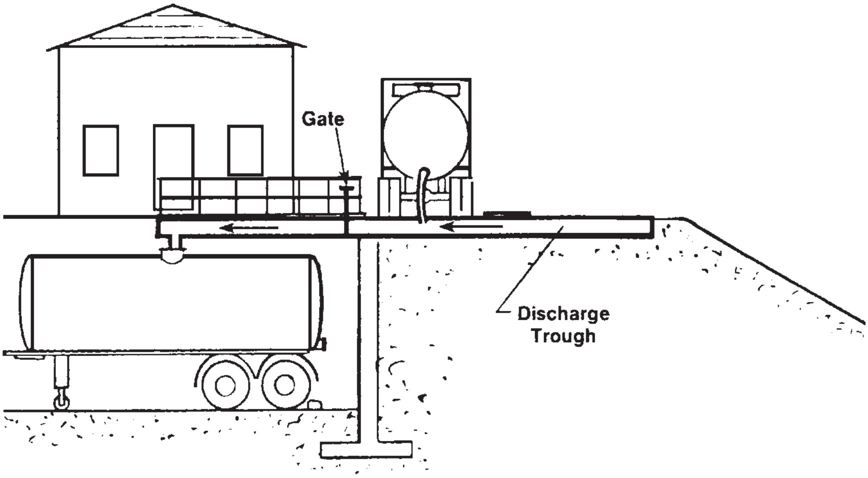


Figure 2.73 Septage transfer to larger hauler truck.

Figure 2.74 An operator manually cleaning a bar screen.

gravity settling in either a horizontal-flow-type or aerated grit chamber system [28,34,60].

The horizontal-flow type of system enhances particle settling by increasing the flow detention time through the grit chamber [28,34,60]. This process is not effective in removing grit from septage because, due to its enmeshment in organic matter, the grit particles have low settling rates. In the aerated grit chamber, the diffused air that is pumped into the chamber imparts a spiral motion to the septage flow. The flow pattern enhances the breakup and settling rate of the grit, making this process more amenable to septage treatment (Fig. 2.75). Basic design data for an aerated grit chamber are provided in Table 2.42.

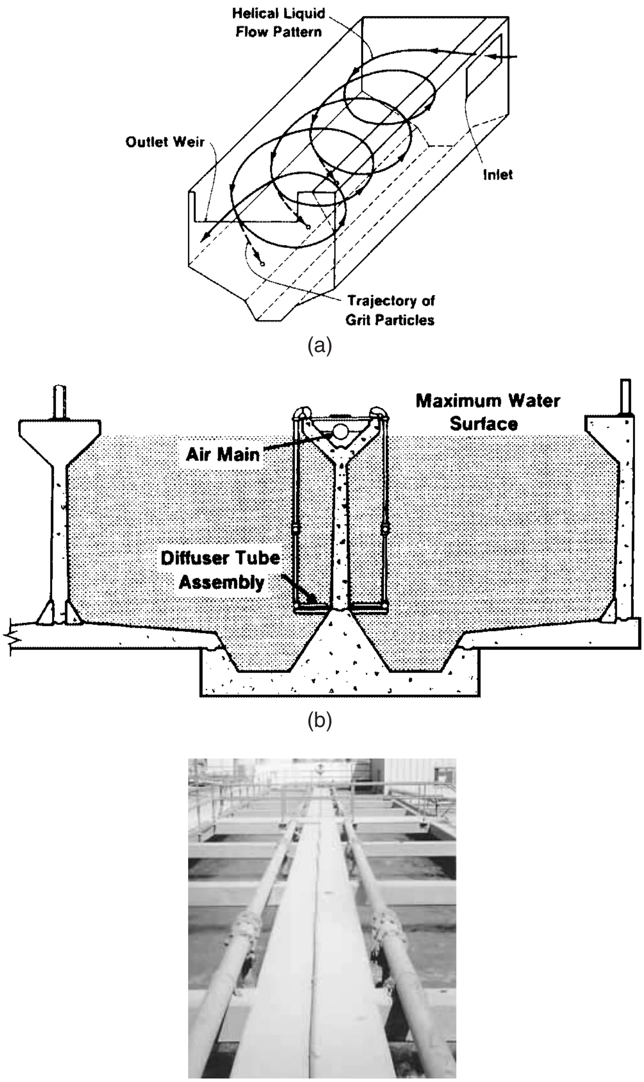
Cyclone degritters are also effective in the pretreatment of septage because the mixing action achieved in this unit operation is similar to that achieved in an aerated grit chamber (Fig. 2.76). An added advantage of the cyclone degritter is that it generates less odor than an aerated grit chamber because no forced aeration occurs [28,34].

The limiting design factor of a cyclone degritter is the flow velocity that must be maintained by septage pumping. The solids content of septage being pumped into these units should be less than 2 percent (i.e., 20,000 mg/liter) for the flow velocity to be sufficient to maintain effective grit removal [28,34]. If individual loads of septage exceed this limit, they may require equalization or dilution. Grit removed from septage normally is transferred to a sanitary landfill for disposal.

2.5.5.6 Storage and equalization. Holding facilities are used to provide septage storage, equalization, mixing, and/or aeration prior to further treatment. The design of a holding facility depends on the prior and subsequent treatment of septage. For example, if septage is to be added directly from a holding tank to a sewer or to a wastewater treatment plant, the holding tank typically is equipped with mechanical or diffused-air aeration to improve treatability and prevent settling of organic solids. The recommended range of mechanical power input to achieve satisfactory mixing in a septage holding tank is 20 to 40 hp/MG of storage, whereas an aeration rate of 1.2 to 2.0 actual cubic feet per minute (acfm) per 1000 gal of septage has been found to give satisfactory results [34].

The most economical design for a septage holding facility is an open lagoon (sometimes aerated). Lagoons, however, require considerable land area and may create odor problems. For treatment at an existing septage treatment or wastewater treatment facility, enclosed tanks with provisions for mixing and aeration generally are recommended to control spillage and odors [28,34,65].

In cases where septage is being discharged to an independent septage treatment facility or municipal wastewater treatment plant,



(a)

(b)

(c)

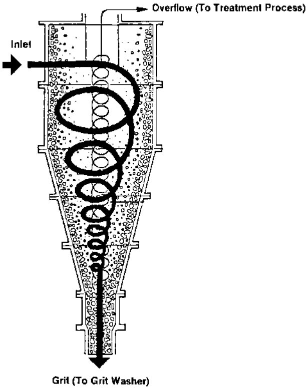
Figure 2.75 Schematic diagrams of an aerated grit chamber illustrating (*a*) spiral flow motion and (*b*) cross section of aerated grit chamber. (*c*) Aerated grit chamber with two parallel air mains.

enclosed septage holding tanks can be placed before or after pretreatment (i.e., screening and degritting) facilities. The advantage of providing storage capacity before pretreatment is that the grit chamber can be sized on the basis of a controlled flow rate averaged over a specified period of time. The disadvantage of providing equalization prior

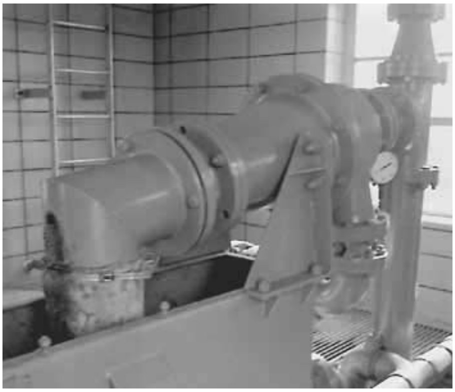
TABLE 2.42 Typical Design Data for an Aerated Grit Chamber*

	Range	Typical
Dimensions		
Depth, ft (m)	7–16 (2–5)	—
Length, ft (m)	25–66 (7.5–20.0)	—
Width, ft (m)	8–23 (2.5–7.0)	—
Detention time (min)	2–5	3
Airflow rate, ft^3/ft·min (m^3/m·min)	3.6–7.2 (0.3–0.6)	6.0 (0.5)
Transverse velocity, ft/s (m/s)	1.5–2.0 (0.4–0.7)	1.8 (0.6)

*Adapted from refs. [28,34,60].



(a)



(b)

Figure 2.76 (*a*) Schematic diagram of a cyclone degritter. (*b*) Photograph of a cyclone degritter. (*Courtesy of Dorr Oliver.*)

to pretreatment facilities is that pump wear is accelerated due to the abrasive properties of septage solids.

The major design criterion for a septage holding tank is detention time. As a rule, capacity of at least 1 day's maximum expected volume of septage should be available for storage. However, the design of the holding tank is site-specific and depends on the sensitivity of the downstream treatment processes and the expected variation in septage volumes. When other preliminary treatment operations are performed in conjunction with flow equalization (e.g., preaeration), then adequate detention times for these functions should be taken into consideration in the design [28,34].

2.5.5.7 Odor control. The most characteristic odor of septage is that of hydrogen sulfide, which is produced by the anaerobic conversion of sulfate to sulfide [46,50]. In addition to sulfide, other compounds responsible for causing odors in septage include (1) mercaptans, (2) amines, (3) aldehydes, (4) skatoles, and (5) volatile acids [34]. Since each truckload of septage can vary with respect to the amount of odorous gases it gives off when septage is emptied or aerated, odor intensity at a septage receiving or treatment facility will vary throughout the day.

The principal concern with odors is not related to their potential physical harm to humans but rather to the psychological stress they produce. In designing septage receiving and treatment facilities, odor control must be considered at the initiation of the project. Proper siting and/or the use of appropriate air pollution control technologies typically can solve odor problems at septage facilities [34]. Table 2.43 summarizes approaches for minimizing odors at septage receiving facilities, whereas Table 2.44 provides approaches for minimizing odors at septage land-application sites.

For septage handling systems, the best approach to control odors is to cover the source of odor and to exhaust these emissions to a suitable control system. Table 2.45 provides a summary of technologies used for the treatment of odorous air. Design and operation descriptions of the various odor treatment systems may be found in the following references [34,46].

Many of the odor treatment systems identified in Table 2.45 (e.g., scrubbers, carbon adsorption, thermal oxidation) may be inappropri-

TABLE 2.43 Guidelines for Minimizing Odor Emissions at Septage Receiving Facilities*

1. Use quick-disconnect fittings between pumper truck and receiving station.
2. Provide wash-down facilities to clean up any spills.
3. Avoid “free fall” of septage by extending receiving pipes below water surface.
4. At wastewater treatment plants, introduce septage at slow, controlled rates.
5. For holding tanks, ventilate tank and direct emissions to odor-control system.

*Adapted from refs. [34,46].

TABLE 2.44 Guidelines for Minimizing Odor Emissions at Septage Land-Application Sites*

1. Select remote sites, if possible.
2. Use subsurface injection.
3. Apply well-stabilized material.
4. Do not apply septage when wind conditions are unfavorable.
5. Use covered holding tanks and vent exhaust to odor-control system.

*Adapted from refs. [46,50,52].

TABLE 2.45 Odor Treatment Systems*

1. Packed-tower wet scrubber
2. Fine-mist wet scrubber
3. Activated-carbon adsorber
4. Biofilters
5. Thermal oxidizer
6. Diffusion into aeration basin

*Adapted from ref. [34].

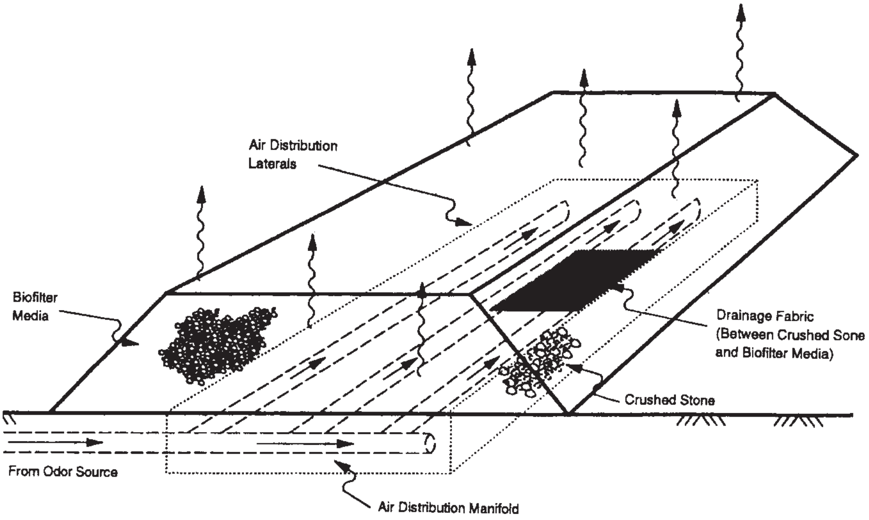


Figure 2.77 Schematic diagram of a biofilter used for septage odor control.

TABLE 2.46 Recommended Design Criteria for Biofilter Systems*

Detention time	>30 s
Media depth	>3 ft
Media pH	6–8
Pore volume	40–50 percent
Moisture content	50–60 percent

*Adapted from refs. [46,50].

ate for small septage handling facilities due to their high capital and operating costs. Interest in biofilters as an odor-control technology has increased recently due to their simplicity and low capital and operations costs (Fig. 2.77). In this system, odorous air is passed upward through a bed of porous material (e.g., composted septage). Odors are removed through a combination of mechanisms including absorption, adsorption, and biological oxidation [34,46,50].

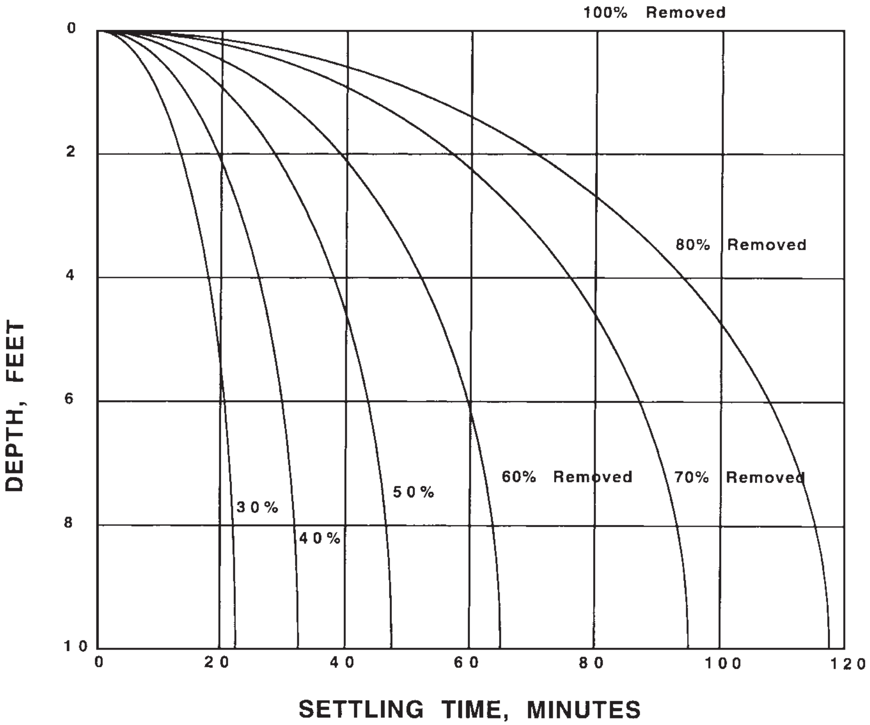
Although biofilters are very effective at odor control, design criteria are limited. Typical design criteria for biofilters are given in Table 2.46. Additional design data for biofilter systems may be found in the following references [46,50,60].

2.6 Problems

2.1 The Gusau City Water Reclamation Plant has determined that a grit chamber must be installed prior to the primary clarifiers to reduce the loading of inert materials entering the wastewater treatment system. If the grit chamber is to be designed to operate with an average overflow rate of 4800 gal/ft^2·h, estimate the fraction of particles removed from the system assuming that the discrete-particle settling behavior is followed. Laboratory column tests provided the following data:

Settling velocity (ft/min)	Weight fraction remaining
80	0.89
45	0.82
17	0.71
10	0.59
6	0.42
3	0.22
2	0.12
1	0.05

2.2 The Kaduna City Sewage Works has decided to install a settling basin to reduce the solids loading to its primary clarification system. Assuming that the settling basin has a depth of 10 ft, estimate the percent solids removal if the hydraulic retention time in the settling basin is 60 minutes. To assist in the design of the settling basin, influent wastewater samples were used in laboratory column tests to generate the following isoconcentration curves:



2.3 The Sokoto County Sewer Improvement District is planning to upgrade its primary clarification system to include four primary clarifiers, each of which is rated at an average overflow rate of 1000 gal/ft^2·day. If the average volumetric flow rate and suspended solids concentration to the facility are 8 million gal/day and 190 mg/liter, respectively, determine the following:

A. Area of clarifier based on overflow rate
B. Underflow solids concentration for each clarifier
C. Minimum area for each clarifier based on solids flux

Assume that the underflow rate is to be maintained at 5 percent of the overflow rate and that the laboratory sludge settling results at the top of the next page have been obtained from column tests.

2.4 The Kumasi City Water Reclamation Plant is currently treating 2 million gal/day of municipal wastewater with an average suspended solids concentration of 200 mg/liter. If the primary treatment system consists of four clarifiers, each of which receives 25 percent of the influent flow, determine the following:

A. Underflow velocity required to maintain a 3 percent sludge underflow concentration

Solids concentration (mg/liter)	Settling velocity (ft/h)
80	25
220	25
480	21
900	19
2,300	12
4,700	9
6,700	6
9,000	4
13,400	2
18,000	0.9
29,000	0.4
48,000	0.1
57,000	0.07

B. Minimum clarifier area to maintain a 3 percent sludge underflow concentration

C. Underflow pumping rate (Q_u) to maintain a 3 percent sludge underflow concentration

Assume that the following laboratory sludge settling results have been obtained from column tests:

Solids concentration (mg/liter)	Settling velocity (ft/h)
90	22
200	22
390	22
700	20
1,900	18
3,600	12
4,800	8
6,400	5
9,200	2
12,500	1
21,000	0.6
33,000	0.4
45,000	0.2
58,000	0.1

2.5 The Makurdi City Wastewater Treatment Plant is upgrading its primary treatment system to include polymer addition. If the mixing system consists of a paddle mixer with six blades (4-ft diameter), estimate the power requirements if the blades are rotating at a rate of 20 rev/min (0.33 rev/s). Assume that the average influent water temperature is 60°F (15°C).

2.6 The Kitui City Wastewater Treatment Plant has decided to add a coagulation/flocculation tank to it primary wastewater treatment system. If the tank will be a paddle-type flocculation system, estimate the power requirement and paddle area assuming that the following data apply:

Coagulation/flocculation tank volume: 200,000 gal

Desired velocity gradient: 50 fps/ft

Coefficient of drag on paddles C_D: 2.0

Paddle tip speed: 1.4 ft/s

Relative velocity of paddles: 0.5 times paddle tip speed

Water temperature: 60°F (15°C)

2.7 The Lodwar County Wastewater Reclamation Facility is presently treating 10 million gal/day (10 MGD) of domestic wastewater. Alum is being added in a flash mixer with the influent domestic wastewater to increase the efficiency of primary sedimentation. If 200 lb of alum is added per million gallons of wastewater treated, determine the mass of chemical solids produced.

2.8 If the primary sludge generated from the Lodwar County Wastewater Treatment Plant (see Prob. 2.7) has a specific gravity of 1.03 and a moisture content of 97 percent, estimate the daily volume of primary sludge generated per million gallons of wastewater treated. Assume that the influent suspended solids concentration is 200 mg/liter and that the suspended solids removal efficiency in the primary treatment system is 80 percent.

2.9 The Lodwar County Wastewater Reclamation Facility (see Prob. 2.7) has decided to evaluate the use of ferric chloride to enhance the removal of suspended solids. Assuming that 50 percent of the suspended solids is removed in the primary clarifier without chemical addition and that, with the addition of ferric chloride, the removal efficiency increases to 80 percent, estimate the mass and volume of sludge produced in primary treatment with and without chemical addition. Assume that the following data also apply:

Wastewater flow rate: 10.0 MGD

Influent suspended solids concentration: 200 mg/liter

Ferric chloride added: 40 lb/MG

Sludge specific gravity (with $FeCl_3$): 1.04

Sludge specific gravity (w/o $FeCl_3$): 1.02

Moisture content (with $FeCl_3$): 94.0

Moisture content (without $FeCl_3$): 96.0

2.10 The Moyale County Sewer Improvement District has decided to upgrade its wastewater treatment operations to include a conventional plug flow activated-sludge system designed to receive an average flow rate of 10 million gal/day. Given the following conditions, estimate the daily mass (dry basis) and volume of secondary sludge that will be produced from the system:

Influent BOD_5: 200 mg/liter

BOD_5 removal efficiency: 96 percent

Maximum yield coefficient Y: 0.60

Mean cell residence time: 8 days

Endogenous decay coefficient k_d: 0.04 day·liter

Moisture content of settled sludge: 98 percent

Specific gravity of settled sludge: 1.02

Conversion of BOD_5 to BOD_L: 0.68

2.11 The Moyale County Sewer Improvement District (see Prob. 2.10) has decided to increase the mean cell residence time of its activated sludge system to 20 days. If all other conditions remain unchanged, estimate the daily mass (dry basis) and volume of secondary sludge that will be produced from the system.

2.12 The Odongo City Wastewater Treatment Plant is currently operating an oxidation ditch activated-sludge system at a mean cell residence time of 15 days. Given the following conditions, estimate the sludge pumping rate if sludge is (a) wasted from the secondary clarifier recycle line and (b) wasted from the aeration basin.

Aeration basin mixed-liquor VSS concentration: 2900 mg/liter

Recycle flow mixed-liquor VSS concentration: 14,000 mg/liter

Aeration basin volume: $2.0 \cdot 10^6$ gal

Concentration of solids from effluent of settling tank: 0.0 mg/liter

Influent plant flow rate: $16 \cdot 10^6$ gal/day

2.13 An operator from the Odongo City Wastewater Treatment Plant (see Prob. 2.12) desires to establish the return activated-sludge (RAS) flow rate for the activated-sludge system using a 1-liter settleometer. Assume that after a 1-liter sample of mixed liquor from the aeration tank is allowed to settle for 30 minutes, the volume occupied by the settled solids is approximately 220 ml. Given the results of the settleometer test, estimate the RAS flow rate if the average plant flow rate is 16 million gallons per day (MGD) and the sludge volumetric wastage rate from the recycle line is maintained at 280,000 gal/day.

2.14 The plant supervisor from the Odongo City Wastewater Treatment Plant (see Prob. 2.12) desires to establish the return activated-sludge (RAS) flow rate for the activated sludge system using a mass-balance approach. If the mixed-liquor volatile suspended solids (MLVSS) concentration in the aeration basin and return recycle line are maintained at 2900 and 14,000 mg/liter, respectively, estimate the return activated-sludge (RAS) flow rate. Assume that the plant influent flow and sludge wastage rate from the return recycle line are held constant at $16 \cdot 10^6$ and 280,000 gal/day, respectively.

2.15 The Geidam County Wastewater Treatment Plant is currently treating 2 MGD of municipal wastewater in a completely mixed activated-sludge system equipped with a diffused-air aeration system. Given the following conditions, estimate the minimum pounds of oxygen that must be supplied by the aeration system each day to treat the organic load. Assume that the sludge wasting occurs from the recycle line and that the suspended solids concentration discharged from the secondary clarifier effluent is negligible.

Influent BOD_5: 200 mg/liter
Effluent BOD_5: 15 mg/liter
Aeration basin volume: $1 \cdot 10^6$ gal
Mean cell residence time: 20 days
MLVSS (aeration basin): 2800 mg/liter
MLVSS (sludge recycle): 13,000 mg/liter
Conversion factor BOD_5 to BOD_L: 0.68

2.16 For the Geidam County Wastewater Treatment Plant (see Prob. 2.15), estimate the minimum compressor horsepower necessary to meet the wastewater's oxygen demand if the average compressor efficiency is 70 percent. Assume that the following conditions apply:

Compressor inlet air temperature: 60°F (530°R)
Compressor inlet air pressure: 14.0 psia
Compressor outlet air pressure: 180.0 psia
Weight fraction of oxygen in air: 0.232
n (constant for air): 0.283
Gas constant R: 53.3 $\text{ft} \cdot \text{lb}_f/\text{lb}_{air} \cdot {}^\circ\text{R}$

2.17 The Lamu City Wastewater Treatment Plant is currently treating 12 MGD of municipal wastewater in a conventional plug flow activated-sludge process. If the plant manager desires to maintain a dissolved oxygen concentration of at least 2.0 mg/liter in the mixed liquor, estimate the number of 30-hp surface aerators that must be installed in the aeration tank. Assume that the following conditions apply:

Average wastewater temperature: 18°C
Biochemical oxygen demand (influent): 200 mg/liter BOD_5
Biochemical oxygen demand (effluent): 25 mg/liter BOD_5
Volume of aeration basin: $2 \cdot 10^6$ gal
Mean cell residence time: 20 days
Mixed-liquor volatile suspended solids (aeration basin): 3500 mg/liter
Volatile suspended solids concentration (sludge recycle): 15,000 mg/liter
Altitude of treatment plant above sea level: 200.0 ft
Theoretical oxygen transfer rate of surface aerator (N_o): 2.0 lb O_2/hp·h
α: 0.82
β: 1.0
Conversion factor BOD_5 to BOD_L: 0.68

2.18 An operator at the Lamu City Wastewater Treatment Plant (see Prob. 2.17) notices that the mixed-liquor suspended solids are settling poorly in the secondary clarifiers and suspects that the cause is bulking sludge. Estimate the minimum nitrogen loading necessary to ensure that the activated-sludge process is not nutrient-limited given the following operational conditions:

Biochemical oxygen demand (influent): 200 mg/liter BOD_5
Biochemical oxygen demand (effluent): 25 mg/liter BOD_5
Volume of aeration basin: $2 \cdot 10^6$ gal
Mean cell residence time: 20 days
Y, maximum yield coefficient: 0.5 lb VSS/lb BOD_L
k_d, decay coefficient: 0.04 day·liters
Conversion factor BOD_5 to BOD_L: 0.68

2.19 The Tororo City Water Reclamation Plant is currently operating two low-rate trickling filters in parallel to treat a municipal wastewater flow rate of 6 MGD having an average BOD_5 concentration of 180 mg/liter. If the trickling filter is 120 ft in diameter and 6 ft in depth, estimate the dosing rate (inches per pass) and the rotational speed of the distributor arm if each trickling filter receives 50 percent of the influent flow rate. Assume that the recirculation rate is 70 percent of the influent flow rate and that each rotary distributor has two arms.

2.20 The Tororo City Water Reclamation Plant (see Prob. 2.19) is considering upgrading its trickling filter system by installing a 40-ft modular plastic medium roughing filter to reduce the organic loading to the rock filters. From a pilot investigation of the temperature profile within the plastic medium filter, a maximum temperature of 64.2°F (18°C) occurs at a depth of 22 ft within the filter, while a minimum temperature of 59.5°F (15°C) occurs at a filter depth of 6 ft. Given the temperature profile within the trickling filter, estimate the draft in the filter (in inches of H_2O) if the average ambient air temperature is 82°F (28°C).

2.21 If the 40-ft modular plastic medium vertical-flow filter installed by Tororo City Water Reclamation Plant (see Prob. 2.19) is to reduce the influent wastewater from 180 to 60 mg/liter BOD_5, estimate the diameter of the filter if the recirculation rate is maintained at 80 percent of the influent flow rate. Assume that data from a 10-ft pilot-plant filter using identical wastewater indicated that, to achieve the desired effluent quality, a treatability constant k_{20} of 0.08 is required.

2.22 The Turkana City Water Reclamation plant is installing a dual-stage circular rock medium trickling filter operating in series to treat a 4 MG/day of municipal wastewater flow. If the influent organic matter concentration is 250 mg/liter BOD_5, estimate the diameter of each stage if both stages have a depth of 6 ft and a recycle rate of 150 percent of the influent flow. Assume that the trickling filter system should be designed to achieve an effluent BOD_5 concentration of 100 and 25 mg/liter from the first and second stages, respectively.

2.23 The Calabar County Sewer Improvement District has constructed a rotating biological contactor (RBC) system to treat its daily wastewater flow.

The RBC system consists of three stages in series having surface areas of 110,000, 140,000, and 160,000 ft^2, respectively. If the desired hydraulic loading rate for each stage is 2.2 $\mathrm{gal/ft}^2\cdot\mathrm{day}$ and the specific tank volume is limited to 0.10 $\mathrm{gal/ft}^2$, estimate the tank volume of each stage and the total wastewater detention time in the RBC system.

2.24 The Geneina County Public Health Department estimates that there are approximately 2200 septic tanks within its jurisdiction. If the population of Geneina County is 9200 and the average septic tank volume is 1500 gal, estimate the annual septage generation rate based on both the number of septic tanks in the county and the number of residents. Assume a per capita septage generation rate of 70 gal/person·year and that the pump-out frequency of the septic tanks is once every 5 years.

2.7 References

1. Aerstin, F., and G. Street (1978), *Applied Chemical Process Design,* Plenum Press, New York.
2. Bailey, J. E., and D. F. Ollis (1986), *Biochemical Engineering Fundamentals,* McGraw-Hill, New York.
3. Bastian, R. K. (1997), "The Biosolids (Sludge) Treatment, Beneficial Use and Disposal Situation in the USA," *European Water Pollution Control,* pp. 62–79.
4. Butler, J. N. (1982), *Carbon Dioxide Equilibria and Their Applications,* Addison-Wesley, Reading, Mass.
5. Chaney, R. L. (1984), "Potential Effects of Sludge-Born Heavy Metals and Toxic Organics on Soils, Plants, and Animals, and Related Regulatory Guidelines," in *Workshop on the International Transportation and Utilization or Disposal of Sewage Sludge, Including Recommendations* (December 12–15), final report PNSP/85-01, Pan American Health Organization, Washington, p. 56.
6. Corey, R. B. (1992), "Phosphorus Regulations: Impact of Sludge Regulations," *Crops Soils Newsletter* 20:5–10.
7. Davis, M. L., and D. A. Cornwell (1991), *Introduction to Environmental Engineering,* McGraw-Hill, New York.
8. Eckenfelder, W. W. (1989), *Industrial Water Pollution Control,* 2d ed., McGraw-Hill, New York.
9. Falahi-Ardakani, A., J. C. Bouwkamp, F. R. Gouin, and R. L. Chaney (1988), "Growth Response and Mineral Uptake of Lettuce and Tomato Transplants Grown in Media Amended with Composted Sewage Sludge," *Journal of Environmental Horticulture* 6:130–132.
10. Fogler, H. S. (1992), *Elements of Chemical Reaction Engineering,* 2d ed., Prentice-Hall, Englewood Cliffs, N.J.
11. Garvey, D., C. Guarino, and R. Davis (1993), "Sludge Disposal Trends Around the Globe," *Water/Engineering & Management,* pp. 17–20.
12. Grady, C. P. L., and H. C. Lim (1980), *Biological Wastewater Treatment,* Marcel Dekker, New York.
13. Jewell, W. J. (1980), "Use and Treatment of Municipal Wastewater and Sludge in Land Reclamation and Biomass Production Projects: An Engineering Assessment," in *Utilization of Municipal Wastewater and Sludge: An Engineering Assessment for Land Reclamation and Biomass Production* (EPA 430/9-81-012), EPA, Washington, pp. 448–480.
14. Jewell, W. J. (1988), "Anaerobic Sewage Treatment," *Environmental Science and Technology* 22:14–21.
15. Linsley, R. K., J. B. Fanzini, D. L. Freyberg, and G. Tchobanoglous (1992), *Water Resource Engineering,* McGraw-Hill, New York.

16. Loehr, R., W. Jewell, J. Novak, W. Clarkson, and G. Friedman (1979), *Land Application of Wastes,* Vol. 2, Van Nostrand Reinhold, New York.
17. McCabe, W. L., J. C. Smith, and P. Harriot (1993), *Unit Operations of Chemical Engineering,* McGraw-Hill, New York.
18. McCarty, P. L. (1968), “Anaerobic Treatment of Soluble Wastes,” in E. F. Gloyna and W. W. Eckenfelder (eds.), *Advances in Water Quality Improvement,* University of Texas Press, Austin.
19. National Research Council (1996), *Use of Reclaimed Water and Sludge in Food Crop Production,* National Academy Press, Washington.
20. Parkin, W. P. (1993), *The Complete Guide to Environmental Law,* Specialty Technical Publishers.
21. Peavy, H. S., D. R. Rowe, and G. Tchobanoglous (1985), *Environmental Engineering,* McGraw-Hill, New York.
22. Perry, R. H., and C. H. Chilton (1973), *Chemical Engineers Handbook,* McGraw-Hill, New York.
23. Porter, M. C. (1990), *Handbook of Industrial Membrane Technology,* Noyes Publications.
24. Riet, K. and J. Tramper (1991), *Basic Bioreactor Design,* Marcel-Dekker, New York.
25. Sawyer, C. N., P. L. McCarty, and G. F. Parkin (1996), *Chemistry for Environmental Engineers,* McGraw-Hill, New York.
26. Stumm, W., and J. J. Morgan (1996), *Aquatic Chemistry,* Wiley, New York.
27. Tchobanoglous, G. (1981), *Wastewater Engineering: Collection and Pumping of Wastewater,* McGraw-Hill, New York.
28. Tchobanoglous, G. (1991), *Wastewater Engineering: Treatment Disposal and Reuse,* 3d ed., McGraw-Hill, New York.
29. U.S. Environmental Protection Agency (1975), *Process Design Manual for Suspended Solids Removal* (EPA/625/1-75/003a), Washington.
30. U.S. Environmental Protection Agency (1978), *Upgrading Trickling Filters* (EPA/430/9-78/004), Washington.
31. U.S. Environmental Protection Agency (1979), *Fate of Priority Pollutants in Publicly Owned Treatment Works* (EPA/440/1-79/300), Washington.
32. U.S. Environmental Protection Agency (1979), *Process Design Manual for Sludge Treatment and Disposal* (EPA/625/1-79/011), Washington.
33. U.S. Environmental Protection Agency (1984), *Environmental Regulations and Technology: Use and Disposal of Municipal Wastewater Sludge* (EPA/625/10-841/003), Washington.
34. U.S. Environmental Protection Agency (1984), *Handbook: Septage Treatment and Disposal* (EPA/625/6-84/009), Washington.
35. U.S. Environmental Protection Agency (1985), *Design Manual: Odor and Corrosion Control in Sanitary Sewerage Systems and Treatment Plants* (EPA/625/1-85/018), Washington.
36. U.S. Environmental Protection Agency (1985), *Environmental Regulations and Technology: The Electroplating Industry* (EPA/625/10-85/001), Washington.
37. U.S. Environmental Protection Agency (1985), *Review of Current RBC Performance and Design Procedures* (EPA/600/2-85/033), Washington.
38. U.S. Environmental Protection Agency (1987), *Guidance Manual on the Development and Implementation of Local Discharge Limitations under the Pretreatment Program* (PB92-129188), Washington.
39. U.S. Environmental Protection Agency (1990). “40 CFR Part 503, National Sewage Sludge Survey: Availability of Information and Data and Anticipated Impacts on Proposed Regulations,” *Federal Register* 55(218):47210-47283.
40. U.S. Environmental Protection Agency (1991), *Residential and Commercial Toxic Pollutant Loadings and POTW Removal Efficiency* (PB93-209872), Washington.
41. U.S. Environmental Protection Agency (1991), *Sewer System Infrastructure Analysis and Rehabilitation* (EPA/625/6-91/030), Washington.
42. U.S. Environmental Protection Agency (1992), *Determining Wastewater User Service Charge Rates: A Step-by-Step Manual* (EPA/832/B-92/003), Washington.

43. U.S. Environmental Protection Agency (1992), *Environmental Regulations and Technology: Control of Pathogens and Vector Attraction in Sewage Sludge* (EPA/625/R-92/013), Washington.
44. U.S. Environmental Protection Agency (1992), *Rotating Biological Contactors* (EPA/540/S-92/007), Washington.
45. U.S. Environmental Protection Agency (1993a), *Technical Support Document for Land Application of Sewage Sludge,* Vol. I. (PB93-110575), Vol. II (PB93110583), Washington.
46. U.S. Environmental Protection Agency (1993), *Domestic Septage Regulatory Guidance: A Guide to the EPA 503 Rule* (EPA/832/B-92/005), Washington.
47. U.S. Environmental Protection Agency (1993), *Regulatory Impact Analysis of the Part 503 Sewage Sludge Regulation* (EPA/821/R-93/006), Washington.
48. U.S. Environmental Protection Agency (1994), *A Plain English Guide to the EPA Part 503 Biosolids Rule* (EPA/832/R-93/003), Washington.
49. U.S. Environmental Protection Agency (1994), *Biosolids Recycling: Beneficial Technology for a Better Environment* (EPA/832/R-94/009), Washington.
50. U.S. Environmental Protection Agency (1994), *Guide to Septage Treatment and Disposal* (EPA/625/R-94/002), Washington.
51. U.S. Environmental Protection Agency (1995), *Ground-Water and Leachate Treatment Systems* (EPA/625/R-94/005), Washington.
52. U.S. Environmental Protection Agency (1995), *Process Design Manual: Land Application of Sewage Sludge and Domestic Septage* (EPA/625/R-95/001), Washington.
53. U.S. Environmental Protection Agency (1995), *Process Design Manual: Surface Disposal of Sewage Sludge and Domestic Septage* (EPA/625/R-95/002), Washington.
54. U.S. Environmental Protection Agency (1995), *Part 503 Implementation Guidance* (EPA/833/R-95/001), Washington.
55. U.S. Environmental Protection Agency (1996). *Capsule Report: Evaporation Process* (EPA/625/R-96/008), Washington.
56. U.S. Environmental Protection Agency (1996), *Capsule Report: Reverse Osmosis Process* (EPA/625/R-96/009), Washington.
57. U.S. Environmental Protection Agency (1996), *Technical Support Document for the Round Two Sewage Sludge Pollutants* (EPA/822/R-96/003), Washington.
58. U.S. Environmental Protection Agency (1999), *Environmental Regulations and Technology: Control of Pathogens and Vector Attraction in Sewage Sludge* (EPA/625/R-92/013), Washington.
59. Vesilind, P. A., G. C. Hattman, and E. T. Shene (1986), *Sludge Management and Disposal for the Practicing Engineer,* Lewis Publishers.
60. Viessman, W., and M. J. Hammer (1985), *Water Supply and Pollution Control*, 4th ed., Harper Collins, New York.
61. Viessman, W., and C. Welty (1985), *Water Management,* Harper & Row, New York.
62. Water Pollution Control Federation (1985), *Clarifier Design, Manual of Practice* (FD-8), WPCF.
63. Water Pollution Control Federation (1987), *Activated Sludge, Manual of Practice* (OM-9), WPCF.
64. Water Pollution Control Federation (1988), *O&M of Trickling Filters, RBCs and Other Related Processes, Manual of Practice* (OM-10), WPCF.
65. Water Pollution Control Federation (1988), *Aeration, Manual of Practice* (FD-13), WPCF.
66. Weber, W. (1972), *Physicochemical Processes for Water Quality Control,* Wiley, New York.

Chapter 3

Biosolids and Sludge Processing

3.0 Introduction

Treated municipal wastewater sludges that are regulated under the 40 CFR Part 503 rule are generated during primary and secondary wastewater treatment as well as in domestic septic systems [56]. Depending on the regulatory requirements for a specific biosolids beneficial-use option together with transportation considerations, sludges may undergo various steps of processing before being applied to land. The four major sludge-processing operations that occur at the wastewater treatment plant are

- Thickening
- Stabilization
- Conditioning
- Dewatering

This chapter reviews these processing operations and their impact on the various biosolids beneficial use and disposal options.

3.1 Thickening

Thickening is normally the first processing operation after raw sludge has been generated at the wastewater treatment plant. *Thickening* is

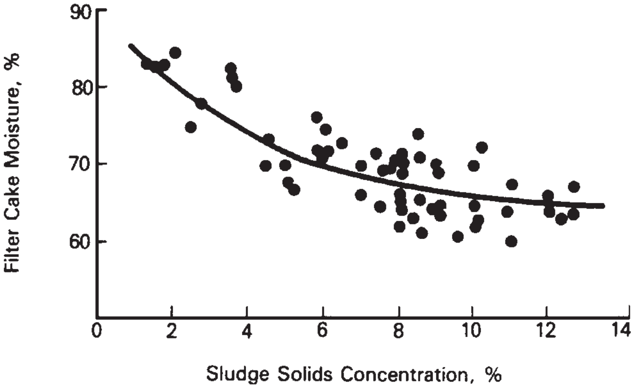


Figure 3.1 Effect of increased sludge thickening on the final moisture content of dewatered sludge [38].

defined as the removal of water from sludge to achieve an overall volume reduction. Volume reduction is desirable to reduce both the capital and operational costs of subsequent sludge-processing steps. For example, thickening from a 1 to a 2 percent solids concentration reduces the sludge volume by 50 percent, whereas increasing the solids content from 1 to 5 percent reduces the volume by 80 percent. Such levels of volume reduction will have a significant impact on the sizing of tanks, pumps, and other downstream sludge-processing equipment. Figure 3.1 illustrates the impact of sludge thickening on the final moisture content achieved in a dewatered sludge cake.

An important physical characteristic of thickened biosolids/sludge is that the resulting material is still fluid (i.e., pumpable). Therefore, the thickened biosolids/sludge may be conveyed within the wastewater treatment plant or to the land-application site by pipeline. The fluidity of the material after processing is a critical distinction between biosolids/sludge thickening and dewatering operations. Dewatered biosolids/sludges are not pumpable and therefore must be conveyed within and outside the wastewater treatment facility by means other than a pipeline (e.g., front-end loader, belt conveyor, truck, rail, barge, etc.). Transportation options for both thickened and dewatered biosolids/sludge are covered in Chap. 5.

Biosolids/sludge thickening generally is accomplished by physical means. The five major thickening processes found at municipal wastewater treatment facilities include

- Gravity (i.e., sedimentation) thickening
- Flotation thickening
- Centrifugal thickening

- Gravity belt thickening
- Rotary-drum thickening

Each of these thickening processes is discussed in the following sections.

3.1.1 Gravity thickeners

The primary objective in the operation of a gravity thickener is to generate a concentrated sludge underflow. Gravity thickening is accomplished in a tank equipped with a slowly rotating rake mechanism that breaks the cohesiveness of influent sludge particles (Fig. 3.2).

Sludge discharged to a gravity thickener disperses into the sedimentation zone, where the solids settle out of solution due to gravitational forces. Although the mechanisms that characterize particle removal in the gravity thickener include discrete, flocculent, and hindered (or zone) settling, owing to the nature and typical concentrations of solids found in gravity thickeners, hindered settling is the predominant settling mechanism [6].

Through the hindered settling mechanism, a thickened sludge blanket flows as a density current to the bottom of the thickener (called the *thickening zone*), where it is collected and removed. The thickened sludge is then transferred to downstream processing operations, while the clarified supernatant is normally recycled back to the wastewater treatment plant headworks or to the primary clarifier for reprocessing. Typical sludge moisture contents achievable by gravity thickeners are summarized in Table 3.1.

3.1.1.1 Design of gravity thickeners. The principal objective in gravity-thickener design is to determine the minimum cross-sectional (i.e., surface) area necessary to produce a thickened-sludge underflow with a specific solids content. The minimum required cross-sectional area of the thickener will be a function of the following process variables: (1) influent flow rate, (2) influent solids content, and (3) limiting solids flux.

Although the plant operator can easily measure and often control the influent flow rate and solids content, the limiting solids flux is not amenable to direct measurement. Instead, the limiting solids flux must be estimated by conducting a series of settling tests on actual sludge samples. The *limiting solid flux* is defined as the maximum rate at which solids can be transmitted through the thickener ($\mathrm{lb/ft^2 \cdot h}$). The concept of the limiting solid flux is illustrated in Fig. 3.3.

It is important to recognize that while the influent flow rate and solids content can be varied to some extent, these parameters are assumed to be constant in gravity-thickener design. In contrast, the

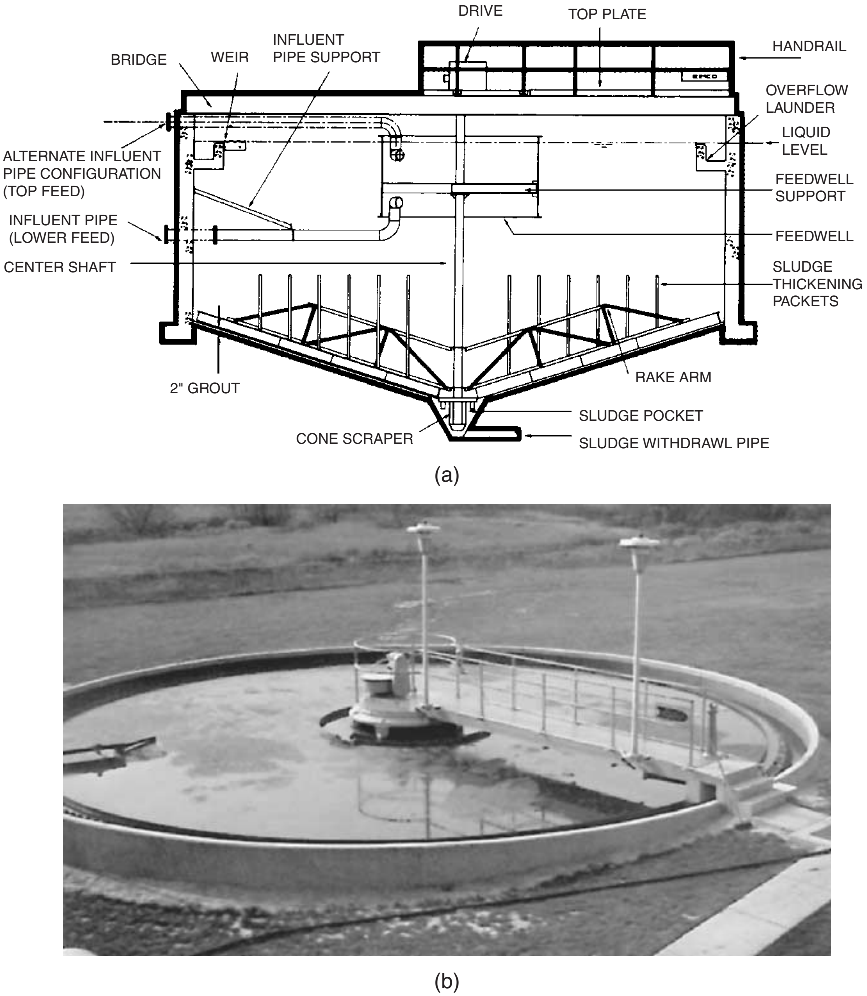


Figure 3.2 (*a*) Schematic diagram of a gravity thickener. (*Courtesy of EIMCO Process Equipment Company.*) (*b*) Sludge gravity thickener. (*Courtesy of Dorr Oliver.*)

magnitude of the limiting solids flux SF_L may be altered frequently through operational adjustments.

Before applying the concept of limiting solids flux to gravitational-thickener design and operation, the design engineer must have a thorough understanding of the concept of solids flux. With regard to gravity thickeners, the solids flux is the mass of solids that is transferred through a unit surface area over a given time (e.g., $lb/ft^2 \cdot h$, $kg/m^2 \cdot h$, etc.).

TABLE 3.1 **Moisture Contents Achievable by Gravity Thickeners***

Sludge type	Moisture content (%) Influent (raw biosolids)	Thickened (effluent biosolids)
Primary	2–7	5–10
Waste-activated sludge (WAS)	0.5–1.5	2–3
Trickling filter (TF)	1–4	3–6
Primary plus WAS	2–5	2–8
Primary plus TF	2–6	4–9

*Adapted from refs. [33,38].

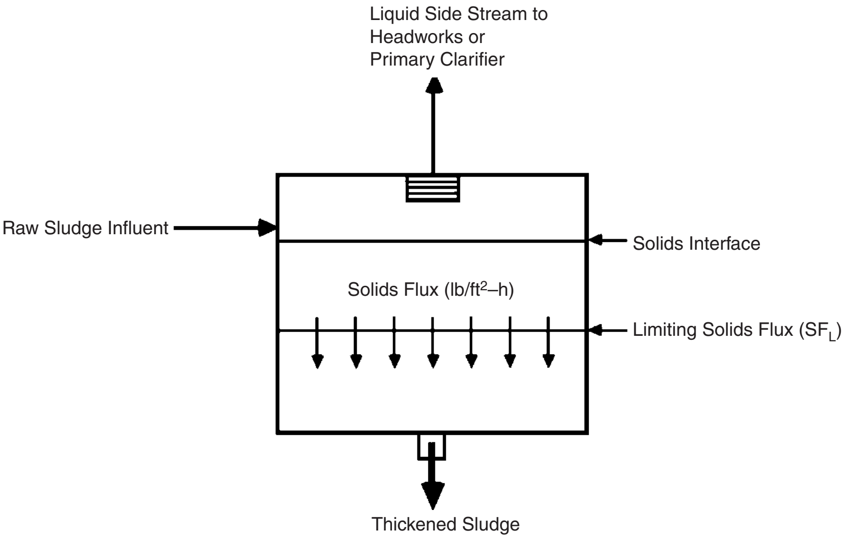


Figure 3.3 Conceptual illustration of the limiting solids flux SF_L.

In a continuously operating gravity thickener, the transmission of solids (i.e., total solids flux SF_T) is brought about by both gravitational forces and bulk fluid movement (i.e., advection). The solids flux due to gravity at any point in the gravity thickener may be estimated by Eq. (3.1):

$$SF_G = kC_iV_i \tag{3.1}$$

where SF_G = solids flux due to gravity, lb/ft² · h
C_i = concentration of solids at any point in the thickener, mg/liter
V_i = settling velocity of solids at concentration C_i, ft/h
k = conversion constant, 1/16,030

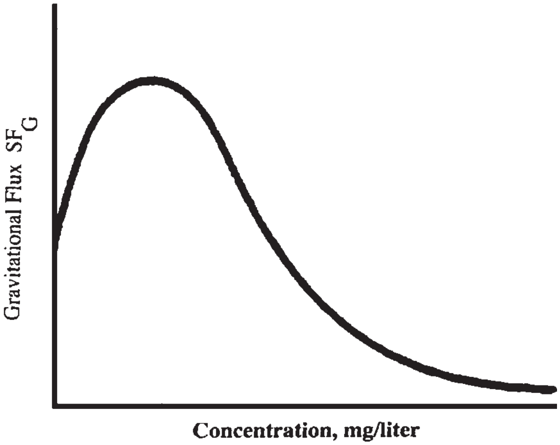


Figure 3.4 Gravitational (batch) flux curve.

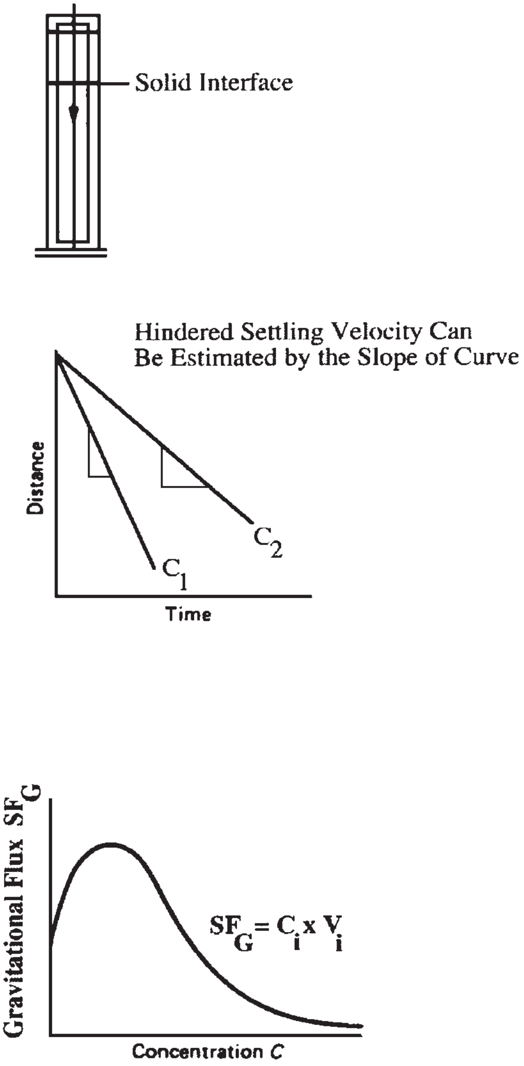
$$\text{NOTE: } k = \left(\frac{7.48 \text{ gal}}{\text{ft}^3}\right) \cdot \left(\frac{8.34 \text{ lb}}{(\text{mg/liter}) \cdot \text{MG}}\right) \cdot \left(\frac{\text{MG}}{10^6 \text{ gal}}\right) = \frac{1}{16{,}030}$$

where MG = million gallons.

Figure 3.4 illustrates a typical batch (i.e., gravity) flux curve that characterizes the gravitational flux of solids within a gravity thickener. The procedure for developing the batch flux curve is illustrated in Fig. 3.5.

As illustrated in Fig. 3.4, the gravitational solids flux depends on both the concentration of solids and the solids settling characteristics (i.e., settling velocity V_i) at that concentration. At low solids concentrations (e.g., less than 0.01 percent solids), the flux of solids due to gravity is small because the settling velocity is independent of solids concentration. If the solids settling velocity remains constant and the solids concentration (i.e., C_i) increases, the solids flux (i.e., kC_iV_i) will increase. At high solids concentrations (i.e., above 1 percent), hindered settling occurs, and the solids settle in a zone or blanket. Since the typical influent solids concentration discharged to gravity thickeners is above 1 percent (i.e., above 10,000 mg/liter), hindered settling is the predominant particle-settling mechanism [38,71].

When the solids concentration reaches a level where hindered settling becomes the predominant particle-removal mechanism, the average solids settling velocity begins to decrease rapidly with any further increase in solids concentration. The overall impact of a declining solids settling velocity is a decrease in the solids flux. The increase then decrease in the solids flux due to the increasing solids concentration indicates that there is a maximum solids flux that



Step 1. In a settling column, allow suspension of known concentration C to settle and monitor velocity of interface.

Step 2. For each concentration evaluated, there is a specific hindered settling velocity V_i.

Step 3. Multiplying the initial solids concentration by the settling velocity V_i gives the gravity flux SF_G.

Figure 3.5 Method of developing a gravitational (i.e., batch) flux curve [33].

occurs within the gravity thickener. If, beginning with a dilute solution, the solids flux is evaluated over a broad range of suspended solids concentrations, the solids flux will pass through the maximum value (see Fig. 3.5).

The solids flux due to bulk transport (i.e., advection) at any point in the gravity thickener is a linear function of the solids concentration C_i and the underflow velocity U_B [Eq. (3.2].

$$\mathrm{SF}_U = kC_iU_B \tag{3.2}$$

where SF_U = solids flux due to underflow, lb/ft² · h
C_i = concentration of solids at any point in the thickener, mg/liter
U_B = settling velocity due to bulk fluid movement, ft/h
k = conversion constant, 1/16,030

NOTE: $k = \left(\dfrac{7.48\ \text{gal}}{\text{ft}^3}\right) \cdot \left(\dfrac{8.34\ \text{lb}}{(\text{mg/liter}) \cdot \text{MG}}\right) \cdot \left(\dfrac{\text{MG}}{10^6\ \text{gal}}\right) = \dfrac{1}{16{,}030}$

The total mass flux of solids SF_T is then the sum of the gravitational flux SF_G and bulk fluid transport flux SF_U. An expression describing the total mass flux is given by Eq. (3.3).

The behavior of the total solids flux SF_T as a function of solids concentration is illustrated in Fig. 3.6. Within the range of hindered zone settling, there is a minimum or limiting solids flux SF_L that occurs within the gravity thickener. The limiting solids flux represents the maximum rate at which solids can be transmitted through the gravity thickener. At steady-state conditions, if the incoming solids loading rate (e.g, lb/ft² · h) is greater that the limiting solids flux, solids will begin to accumulate at the depth at which the limiting solids flux occurs. Moreover, if sufficient solids storage capacity (i.e., tank depth) is unavailable, solids eventually will overflow the effluent weir and be recycled to the headworks of the treatment plant [38].

$$\text{Total mass flux} = \text{gravity flux} + \text{bulk transport flux}$$

$$\mathrm{SF}_T = \mathrm{SF}_G + \mathrm{SF}_U \tag{3.3}$$

$$\mathrm{SF}_T = kC_iV_i + kC_iU_B$$

where SF_T = total solids flux, lb/ft² · h
SF_G = solids flux due to gravity, lb/ft² · h
SF_U = solids flux due to underflow, lb/ft² · h
C_i = concentration of solids at any point in the thickener, mg/liter
V_i = settling velocity of solids at concentration C_i, ft/h
U_B = settling velocity due to bulk fluid movement, ft/h
k = conversion constant, 1/16,030

NOTE: $k = \left(\dfrac{7.48\ \text{gal}}{\text{ft}^3}\right) \cdot \left(\dfrac{8.34\ \text{lb}}{(\text{mg/liter}) \cdot \text{MG}}\right) \cdot \left(\dfrac{\text{MG}}{10^6\ \text{gal}}\right) = \dfrac{1}{16{,}030}$

Figure 3.6 illustrates that the magnitude of both the total and limiting solids flux may be controlled by the underflow velocity U_B. In other

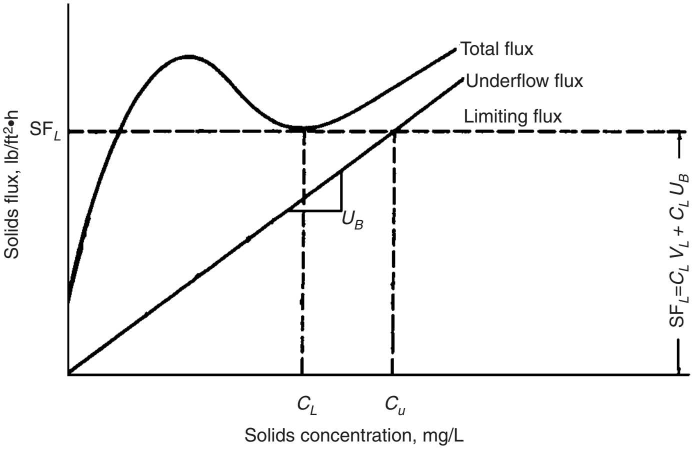


Figure 3.6 Total solids flux SF_T in a continuously operating gravity thickener. Note that the point of intersection of the limiting solids flux and the underflow flux determines the underflow solids concentration C_u.

words, by increasing or decreasing the underflow bulk velocity U_B, the total and limiting solids flux will increase or decrease. Since the underflow bulk velocity is equal to the underflow volumetric flow (or pumping) rate Q_u divided by the cross-sectional area of the thickener (i.e., Q_u/area), the total and limiting solids flux can be controlled by the wastewater treatment operator through a simple underflow discharge or pumping rate adjustment [6,38].

To obtain the limiting solids flux graphically from the total flux curve, draw a horizontal line tangent to the low point of the total flux curve in the hindered zone section. Its intersection with the y axis represents the limiting solids flux SF_L for the thickener (see Fig. 3.6). The corresponding underflow sludge concentration in the thickener (C_u) can be obtained by dropping a vertical line at the intersection of the line of tangency to the limiting solids flux and the underflow velocity (i.e., U_B).

This approach to evaluating the underflow concentration is permissible because, at the bottom of the thickener, the bulk transport solids flux is the predominant solids-removal mechanism (i.e., gravity settling is negligible at the bottom of the gravity thickener). To increase or decrease the solids concentration of the underflow (i.e., C_u), the bulk transport flow velocity U_B must be changed. From Fig. 3.6, if a more concentrated underflow is desired, the underflow bulk velocity U_B should be decreased (i.e., decreasing slope). This operational change would, in effect, increase the solids settling time within the gravity thickener. An increase in settling time would have the net effect of increasing the underflow solids concentration C_u while reducing the limiting solids flux SF_L.

The minimum cross-sectional area required to achieve a desired underflow concentration in a gravity thickener may be estimated by performing a mass balance on the solids that enter and leave the thickener using the principle of limiting solids flux SF_L. This approach, which is described by Eq. (3.4), assumes that all solids that enter the thickener leave in the underflow (i.e., no net solids accumulation or solids lost in clarified effluent).

$$\text{Incoming solids (lb/h)} = \text{maximum rate of solids transmitted through thickener (lb/h)}$$

$$Q_0 C_0 \cdot 8.34 = \text{area of thickener (ft}^2) \cdot \text{limiting solids flux } \mathrm{SF}_L \text{ (lb/ft}^2 \cdot \text{h)}$$

or

$$\text{Area of thickener (ft}^2) = \frac{Q_0 C_0 \cdot 8.34}{\mathrm{SF}_L} \tag{3.4}$$

where Q_0 = volumetric flow rate entering the thickener, million gallons, MG, per hour
C_0 = concentration of solids entering the thickener, mg/liter
$Q_0 C_0 \cdot 8.34$ = solid loading rate, pounds per hour
SF_L = limiting solids flux, lb/ft^2 · h
8.34 = 8.34/(mg/liter) · MG

Example 3.1 illustrates the use of Eq. (3.4) in estimating the minimum required settling area for a gravity thickener.

Example 3.1 The Kisumu Sewer Improvement District is currently treating 1.6 million gal/day of municipal wastewater using a conventional plug flow activated-sludge system. The two secondary sedimentation tanks used at the facility each receives 800,000 gal of wastewater per day and are being operated with an average overflow rate of 700 gal/ft^2 · day. Given the laboratory batch flux data at the top of the next page, estimate (1) underflow velocity U_B, (2) limiting solids flux (SF_L), (3) maximum underflow solids concentration X_u, and (4) minimum solids settling area. Assume that the mixed-liquor suspended solids (MLSS) concentration in the aeration basin is maintained at 2800 mg/liter and that the sedimentation tank recycle ratio (Q/Q_r) is 0.3.

solution

Step 1. Estimate the underflow velocity U_b given a recycle ratio Q_r/Q of 0.3:

$$U_B = \frac{Q_r}{\text{area}} = \left(\frac{Q + Q_r}{\text{area}}\right)\left(\frac{Q_r}{Q + Q_r}\right) = \left(\frac{Q + Q_r}{\text{area}}\right)\left(\frac{0.3Q}{Q + 0.3Q}\right)$$

MLSS (mg/liter)	ft/h	gal/ft · day*
800	16.00	2880
1,000	15.00	2700
2,000	14.00	2520
4,000	9.00	1620
6,000	6.50	1170
8,000	3.80	684
10,000	2.20	396
12,000	1.40	252
15,000	0.80	144
20,000	0.40	72
25,000	0.30	54
35,000	0.10	18
45,000	0.08	14.4

*gal/ft · day = 180 · ft/h.

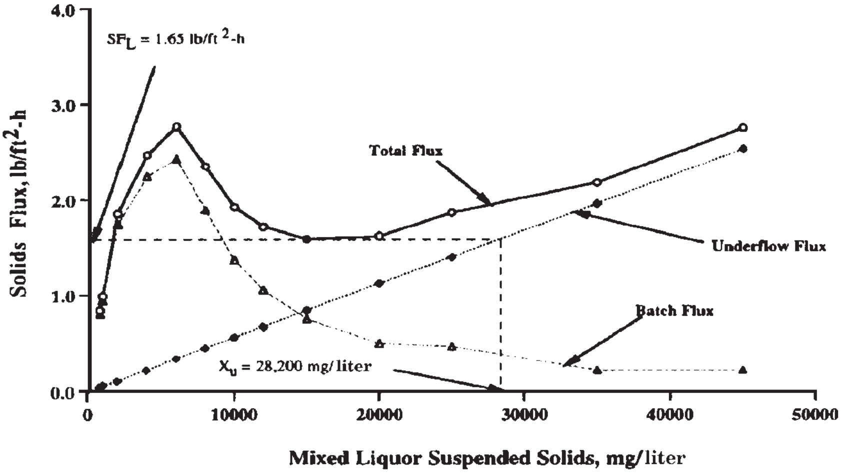
$$= \frac{700\ \text{gal}}{\text{ft}^2 \cdot \text{day}} \left(\frac{\text{ft}^3}{7.48\ \text{gal}}\right) \left(\frac{\text{day}}{24\ \text{h}}\right) \left(\frac{0.3Q}{Q + 0.3Q}\right)$$

$$= 0.9\ \text{ft/h}$$

Step 2. Using MLSS concentration and settling velocity data, estimate the batch flux SF_G using Eq. (3.1). Estimate the underflow flux SF_U by multiplying the MLSS concentration by the underflow velocity U_B. The total flux SF_T is estimated by adding the batch flux and underflow flux. The data are summarized in the following table and shown graphically in the figure on the next page.

MLSS (mg/liter)	ft/h	SF_G (lb/ft^2 · day)	SF_U (lb/ft^2 · day)	SF_T (lb/ft^2 · day)
800	16.00	0.80	0.04	0.84
1,000	15.00	0.94	0.06	0.99
2,000	14.00	1.75	0.11	1.86
4,000	9.00	2.25	0.22	2.47
6,000	6.50	2.43	0.34	2.77
8,000	3.80	1.90	0.45	2.35
10,000	2.20	1.37	0.56	1.93
12,000	1.40	1.05	0.67	1.72
15,000	0.80	0.75	0.84	1.59
20,000	0.40	0.50	1.12	1.62
25,000	0.30	0.47	1.40	1.87
35,000	0.10	0.22	1.96	2.18
45,000	0.08	0.22	2.53	2.75

Step 3. The limiting solids flux SF_L is determined by drawing a line tangent to the low point of the total flux curve to the y axis. From the figure, SF_L is estimated to be 1.65 lb/ft^2 · h. The maximum underflow MLSS concentration C_u is determined by dropping a vertical line from the intersection between the underflow flux line and the limiting solids flux. From the figure, C_u is estimated to be 18,200 mg/liter.



Step 4. Estimate the minimum sedimentation area needed for sludge thickening using Eq. (3.4):

$$\text{Area (ft}^2) = \frac{Q_0C_0}{\text{SF}_L} \; \frac{8.34 \text{ lb}}{\text{MG} \cdot \text{(mg/liter)}}$$

$$= \frac{(Q + Q_r)\, C_0}{\text{SF}_L} \cdot \frac{8.34 \text{ lb}}{\text{MG} \cdot \text{(mg/liter)}}$$

$$= \frac{(1 + 0.3)\, QC_0}{\text{SF}_L} \cdot \frac{8.34 \text{ lb}}{\text{MG} \cdot \text{(mg/liter)}}$$

$$= \frac{(1 + 0.3)\,(0.8 \text{ MG/day})\,(2800 \text{ mg/liter})}{1.65 \text{ lb/ } (\text{ft}^2 \cdot \text{h})\,(24 \text{ h/day})} \cdot \frac{8.34 \text{ lb}}{\text{MG} \cdot \text{(mg/liter)}}$$

$$= 613 \text{ ft}^2$$

Step 5. For a circular sedimentation tank, estimate the minimum diameter using the geometric relationship between diameter and area of a circle:

$$\text{Area} = \frac{\pi \,(\text{diameter})^2}{4}$$

$$613 \text{ ft}^2 = \frac{\pi \,(\text{diameter})^2}{4}$$

$$\text{Diameter} = 27.9 \text{ ft } (\sim 30 \text{ ft})$$

3.1.1.2 State-point method. An alternative graphic approach to using the total solids flux curve for estimating the required gravity-thickener

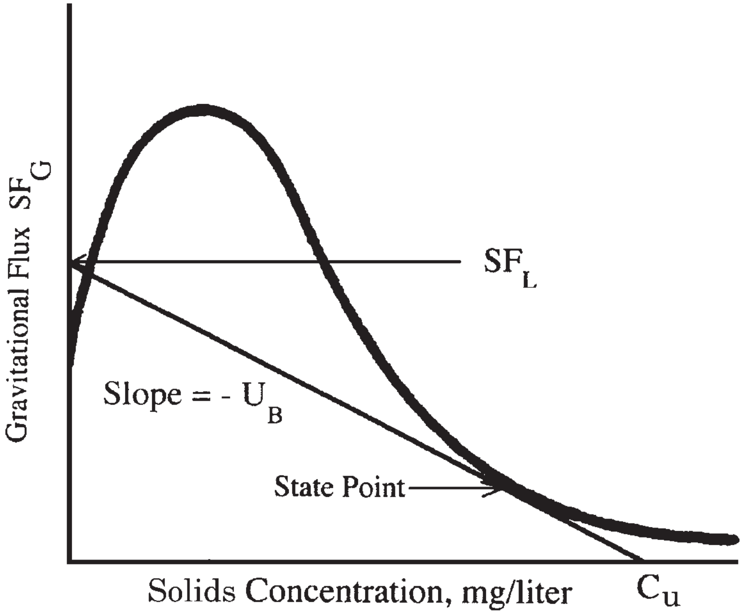


Figure 3.7 State-point method for estimating gravity thickener area (SF$_L$—limiting solids flux).

cross-sectional area is the state-point method. Like the total solids flux approach, the state-point method is employed to estimate the underflow volumetric flow rate Q_u required to obtain a desired underflow solids concentration C_u. One of the advantages of the state-point method is that only the batch (e.g., gravity) flux data are required.

In the state-point method, the limiting solids flux SF$_L$ is first obtained by drawing a line that passes through the desired underflow concentration C_u and is tangent to the batch flux curve (Fig. 3.7). The point of tangency is called the *state point,* and the point of intersection with the y axis is an estimate of the limiting solids flux SF$_L$. Moreover, by geometric comparison with the total flux curve (see Fig. 3.5), it can be shown that the slope of the tangent line in the state-point method is equivalent to the negative magnitude of the underflow bulk velocity (i.e., $-U_B$). Therefore, using only the batch flux curve, the limiting solids flux SF$_L$ and the underflow velocity U_B required to achieve a given underflow solids concentration C_u can be estimated. The limiting solids flux, in turn, can be used to estimate the minimum required area of the thickener through Eq. (3.4).

If an alternative underflow solids concentration C_u is desired, another tangent line should be drawn from the target underflow solids concentration to the y axis. Since the cross-sectional area is fixed, estimation of the new underflow solids concentration will require maintaining a different limiting solids flux SF_L and underflow pumping rate Q_u. Example 3.2 illustrates the process of using the batch flux curve for process control.

Example 3.2 The Walcott County Water Reclamation Facility has decided to install a new gravity thickener to improve its sludge-management program. The current combined primary and waste-activated sludge of 5000 lb/day (dry solids basis) has an initial solids concentration of 1.2 percent. To reduce overall capital and operational costs at the facility, it is desired to thicken this flow to a solids content of 4 percent before transporting the sludge to the anaerobic digesters. For the given operational conditions of the thickener, estimate the limiting solids flux, bulk discharge velocity of the sludge U_B, and cross-sectional area of the thickener. Assume that the following solids settling data were obtained from laboratory tests.

Solids concentration (mg/liter)	Initial settling velocity (ft/h)
2,500	12.5
5,000	7.0
7,500	2.5
10,000	1.2
15,000	0.5
30,000	0.1
50,000	0.05

solution

Step 1. Using Eq. (3.1), estimate the gravity flux:

$$\mathrm{SF}_G = kC_iV_i$$

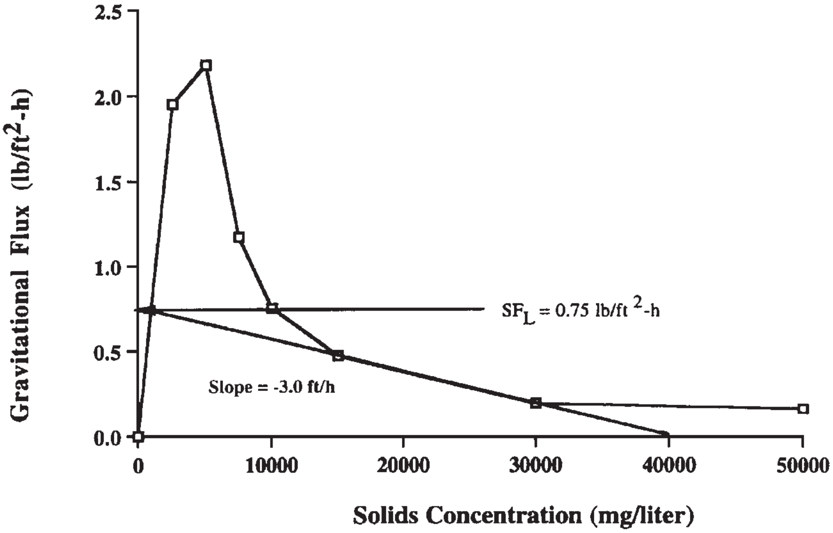
For $C_i = 5000$ mg/liter, SF_G is calculated as follows:

$$\mathrm{SF}_G = \frac{1}{16{,}030} \cdot 5000 \text{ mg/liter} \cdot 7.0 \text{ ft/h}$$

$$= 2.18 \text{ lb/ft}^2 \cdot \text{h}$$

The gravity flux at each solids concentration may be estimated using the same approach and the results presented in tabular form:

Solids concentration (mg/liter)	Initial settling velocity (ft/h)	Gravity flux ($\mathrm{lb/ft^2 \cdot h}$)
2,500	12.5	1.95
5,000	7.0	2.18
7,500	2.5	1.17
10,000	1.2	0.75
15,000	0.5	0.47
30,000	0.1	0.19
50,000	0.05	0.16



Step 2. Plot gravitational flux SF_G versus solids concentration. Draw a tangent line to the batch flux curve starting from an underflow concentration C_u of 40,000 mg/liter (4 percent). Where the tangent line intersects the y axis is the limiting solids flux SF_L. From the figure above, the limiting solids flux is estimated to be 0.75 lb/ft$^2 \cdot$ h. It should be noted that to find the precise point of tangency on the batch flux curve (i.e., the state point), more gravity flux data must be generated, particularly at the higher suspended solids concentrations (i.e., above 15,000 mg/liter).

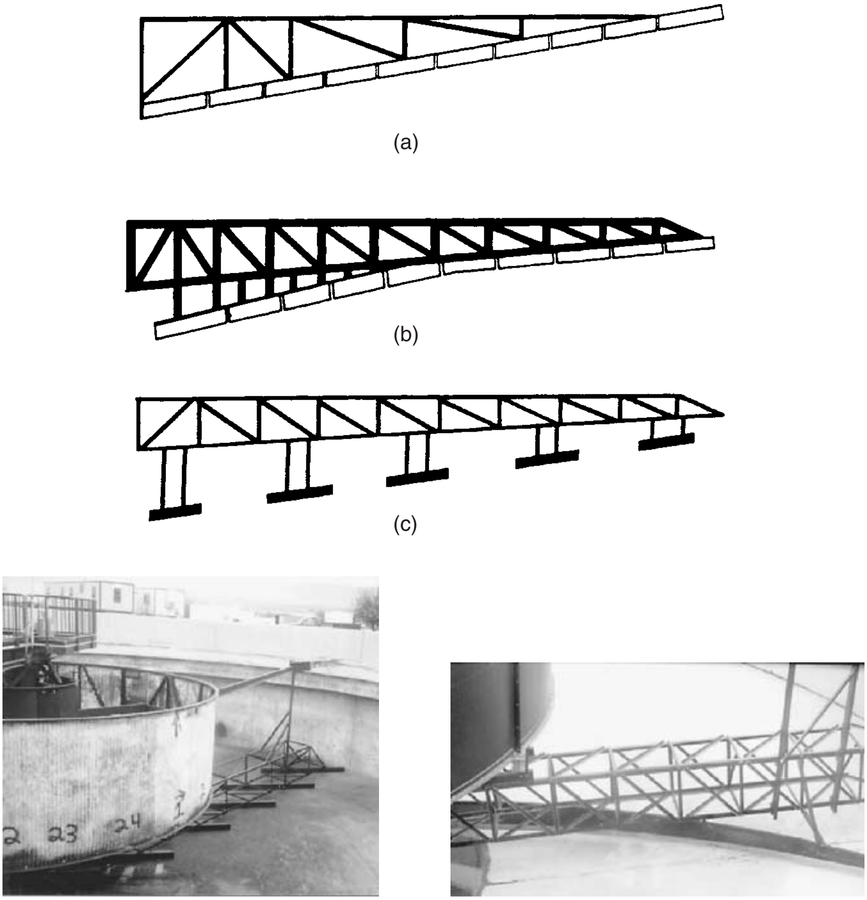
Step 3. Estimate the underflow rate U_B by taking the negative value of the tangent line. In this case, the tangent has a slope of −3.0 ft/h. Therefore, to achieve an underflow concentration of 4 percent, the bulk flow velocity must be maintained at 3.0 ft/h.

Step 4. Find the minimum required gravity-thickener cross-sectional area using Eq. (3.4). Note that solids are added to the thickener at a rate of 5000 lb/day (or 417 lb/h).

$$\text{Cross-sectional area of thickener (ft}^2) = \frac{Q_0 C_0}{\text{SF}_L}$$

$$= \frac{417 \text{ lb/h}}{\dfrac{0.75 \text{ lb}}{\text{ft}^2 \cdot \text{h}}} \cong 556 \text{ ft}^2$$

There are three basic designs of gravity thickeners to increase the solids content of wastewater sludges. These designs include (1) bridge-supported mechanism, (2) a center column–supported mechanism with a central drive, and (3) a center column–supported



(a)

(b)

(c)

(d)

(e)

Figure 3.8 Rake configurations used in gravity thickeners: (*a*) conventional rake; (*b*) rake used for thickeners with sloped bottoms; (*c*) rake used for thickening of high-solids-content sludge. (*Courtesy of EIMCO Process Equipment Company.*) (*d*) Rake blades. (*e*) Spiral rake blades used for rapid removal and compaction of sludge.

mechanism with the driving area at the tank periphery (this design is commonly known as a *traction thickener*). The primary differences in the design and operation of these systems include the method of supporting the rake-drive mechanism and the rake configuration. Figure 3.8 illustrates the various types of rakes commonly used in gravity thickeners.

Together with the estimated gravity-thickener area, rake-drive mechanism, and rake configuration, estimates of the drive torque requirements, total tank depth, and maximum hydraulic loading must be obtained to complete the gravitational-thickener design.

Typical ranges for these design parameters may be found in the following references [6,38].

3.1.1.3 Operational concerns in gravity thickening. Gravity thickening of raw or digested primary sludge is normally an efficient and cost-effective process. The use of gravity thickeners for concentrating mixtures of primary and waste-activated sludge has had mixed results [38]. Moreover, using these systems for sludge storage of more than a few hours can cause operational difficulties, including septicity. The poor results reported for gravity thickening of mixtures of primary and waste-activated sludge are normally due to one of the following reasons: (1) sludge mixture fed to the gravity thickener was greater than 0.5 percent solids, (2) primary sludge was septic, (3) waste-activated sludge flow rate was much greater than the primary sludge flow rate, or (4) thickened sludge was not removed continuously [33,71].

3.1.1.4 Use of primary clarifiers for thickening. While providing separate facilities for sludge-thickening operations is normally cost-effective for large wastewater treatment plants [i.e., those which process more than 5 million gallons per day (MGD) of wastewater], many smaller plants cannot justify the expense and will attempt to use the primary settling tanks to thicken raw primary and waste secondary solids [38]. Many of the problems associated with using the primary clarifier as a thickener are associated with excessive solids retention times that lead to significant odor generation.

When the primary clarifier must serve the dual function of clarifying the wastewater as well as delivering a concentrated sludge for stabilization or dewatering operations, the conventional primary clarifier design configuration is inappropriate. This is particularly true for primary clarifiers in smaller wastewater treatment plants, where it is typically necessary to maintain a 1.5- to 2.0-day solids retention time in order to develop a 3- to 5-ft (1.0–1.5 m) sludge blanket above the sludge withdrawal pipe. This solids retention time not only reduces clarification efficiency but can result in gasification, odor, and floating sludge [6].

In many instances, a primary clarifier can achieve adequate sludge thickening through modification of the floor design. Figure 3.9 illustrates the design of clarifier floors suitable to achieve both clarification and thickening within the primary clarifier.

3.1.2 Flotation thickening

Flotation thickening is a process for separating solids particles from water with the introduction of air. Fine bubbles of air adhere to the solids, which are then lifted to the surface by buoyancy forces.

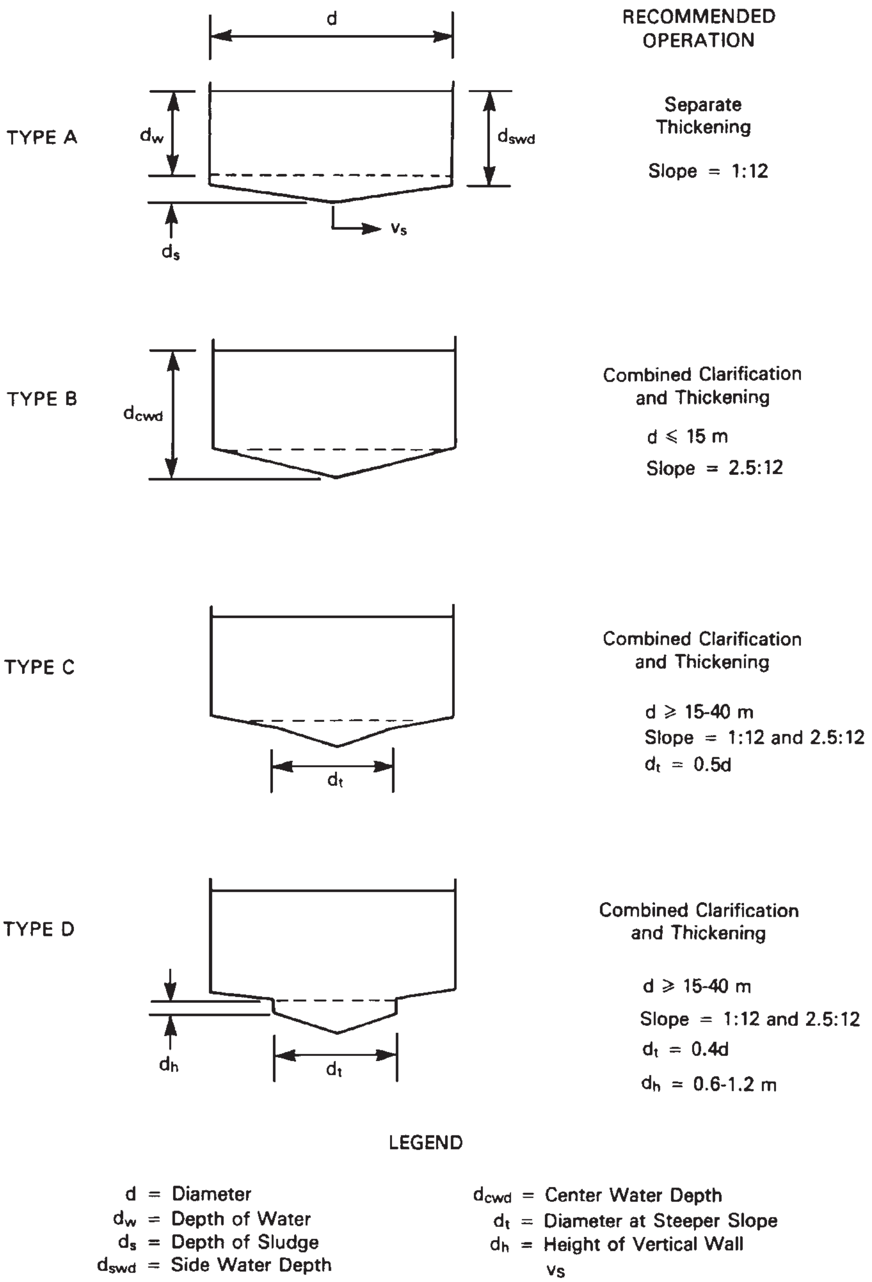


Figure 3.9 Floor bottom design to achieve solids thickening and wastewater clarification [33,38].

TABLE 3.2 Advantages/Disadvantages of DAF Thickening Process*

Advantages

1. Small/light particles can be removed more effectively and in a shorter period of time compared with sedimentation processes.
2. Because of more effective solids capture, higher solids loadings can be treated with DAF thickening as compared with gravity thickening.
3. DAF thickening takes up less space relative to gravity thickeners.

Disadvantages

1. Requires laboratory/pilot-scale tests for evaluation of specific design criteria (e.g., A/S ratio, hydraulic loading rate, etc.).
2. DAF systems are highly mechanized.
3. Operational and maintenance costs of DAF systems are significant.

*Adapted from ref. [33].

Although there are several types of flotation thickening processes available commercially, including (1) dispersed-air flotation, (2) biologic flotation, and (3) vacuum flotation, dissolved-air flotation (DAF) is by far the most prevalent flotation-thickening technology employed at wastewater treatment facilities [33,71]. Therefore, DAF will be the only flotation-thickening technology discussed in this text. Table 3.2 gives the advantages and disadvantages of the DAF thickening process in comparison with gravitational thickening. Readers interested in the full range of flotation-thickening options are directed to the following references [5,27,33].

3.1.2.1 Dissolved-air flotation. In the dissolved-air flotation (DAF) sludge-thickening process, pressurized air is added either directly to the incoming sludge stream or to a separate liquid recycle stream. In the solid-liquid separation tank, the pressure is reduced to atmospheric conditions, resulting in the liberation of air in excess of saturation into solution in the form of small bubbles [33,38]. The bubbles adhere to the suspended solids particles, imparting a buoyancy force to them. The buoyant sludge solids then float to the surface, where they are concentrated (i.e., thickened). The clarified effluent (i.e., subnatant) is removed from the DAF tank and recycled back to the headworks of the plant. To minimize the possibility of solids being lost in the subnatant, the thickened sludge blanket is removed continuously by skimmers (Fig. 3.10).

DAF thickeners can be designed to operate with or without polymer addition. The purpose of polymer addition (i.e., conditioning) is to increase particle aggregation, resulting in more effective capture of air bubbles and greater liquid-solid separation efficiency. Table 3.3 lists typical solids loadings for DAF units operated with and without polymer addition.

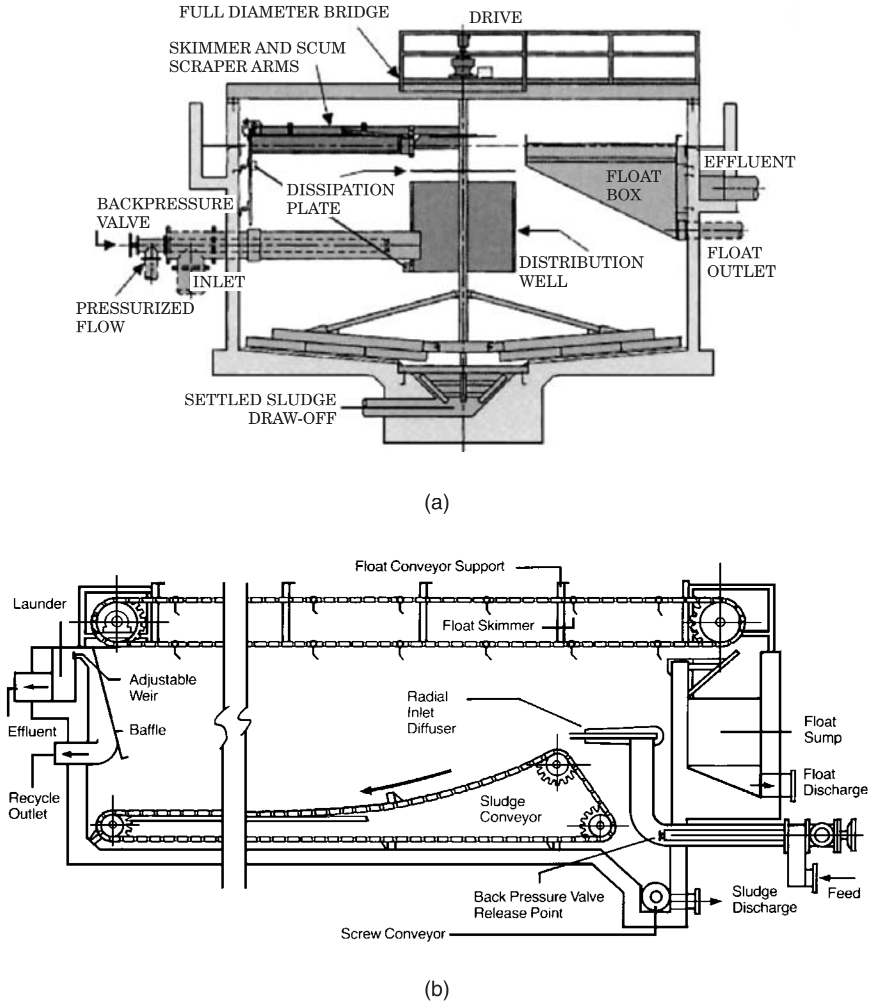


Figure 3.10 (*a*) Schematic diagram of circular DAF system. (*Courtesy of WesTech Engineering, Inc.*) (*b*) Schematic diagram of rectangular DAF system. (*Courtesy of EIMCO Process Engineering Co.*)

3.1.2.2 DAF design considerations. DAF systems can be designed to operate with or without effluent recycle. Smaller wastewater systems (i.e., plant flow of 2 MGD or less) typically will employ DAF systems without effluent recycle. In this operational mode, the entire sludge flow will be pressurized in a retention tank to between 40 and 50 psig (275 and 350 kPa) with compressed air added at the suction side of the pressurizing pump (Fig. 3.11*a*). The flow is held in the retention tank for several minutes to allow the air to dissolve. After dissolution of the air, the pressurized flow is transferred through a pressure-reducing

TABLE 3.3 Typical Solids Loading Rates for Dissolve-Air Flotation Units*

	Solids loading rate (lb/ft^2 · day)	
Sludge type	Without polymer	With polymer
Primary	20–30	~60
Waste-activated sludge (WAS)	14–20	~45
Trickling filter (TF)	14–20	~45
Primary plus WAS	14–30	~45
Primary plus TF	20–30	~60

*Adapted from refs. [33,38].

valve to the flotation tank, where the air is able to partition from the solution in the form of minute bubbles. In larger wastewater treatment plants, a portion of the clarified DAF effluent (15–120 percent) is recycled, pressurized, and semisaturated with air (Fig. 3.11*b*). The recycled flow is mixed with the unpressurized main sludge influent flow before introduction to the flotation tank. In the flotation tank, the air is released from solution in the form of minute bubbles.

Regardless of the mode of DAF operation, the critical design parameters for this type of thickening operation include (1) minimum required surface area, (2) minimum required pressure, (3) surface loading rate, and (4) solids loading rate. The minimum surface area (ft^2) required for effective thickening is determined by consideration of the rise velocity of the sludge particles. Particle rise velocities in DAF units typically vary from 0.027 to 0.535 ft/min (i.e., 0.2–4.0 gal/ft^2 · min) depending of bubble size and the extent of bubble attachment.

Since the air pressure influences bubble size and the sludge solids surface characteristics will have an impact on the extent to which the bubbles will attach, laboratory and/or pilot tests must be conducted to estimate the range of particle rise velocity. Moreover, the ratio of the quantity of air (in milliliters) to quantity of sludge (in milligrams) necessary to achieve satisfactory thickening is also established in laboratory/pilot tests (Fig. 3.12). This ratio, known as the *air-to-solids* (A/S) *ratio,* is an important design variable for DAF thickeners and normally varies from 0.01 to 4.0 [33,38].

To ensure that a significant fraction of particles will have an opportunity to reach the surface of the solid-liquid separation tank before the subnatant flow reaches the effluent weir, the design engineer will choose a hydraulic loading rate (gal/ft^2 · min) that is equal to the minimum rise velocity of the particles that are to be removed. The smaller the hydraulic loading rate chosen by the design engineer, the greater will be the solids capture efficiency of the DAF system.

Solids capture efficiency typically varies from 70 to 98 percent depending primarily on whether or not polymers are used [33,71]. Once the hydraulic loading rate is determined, the minimum surface area

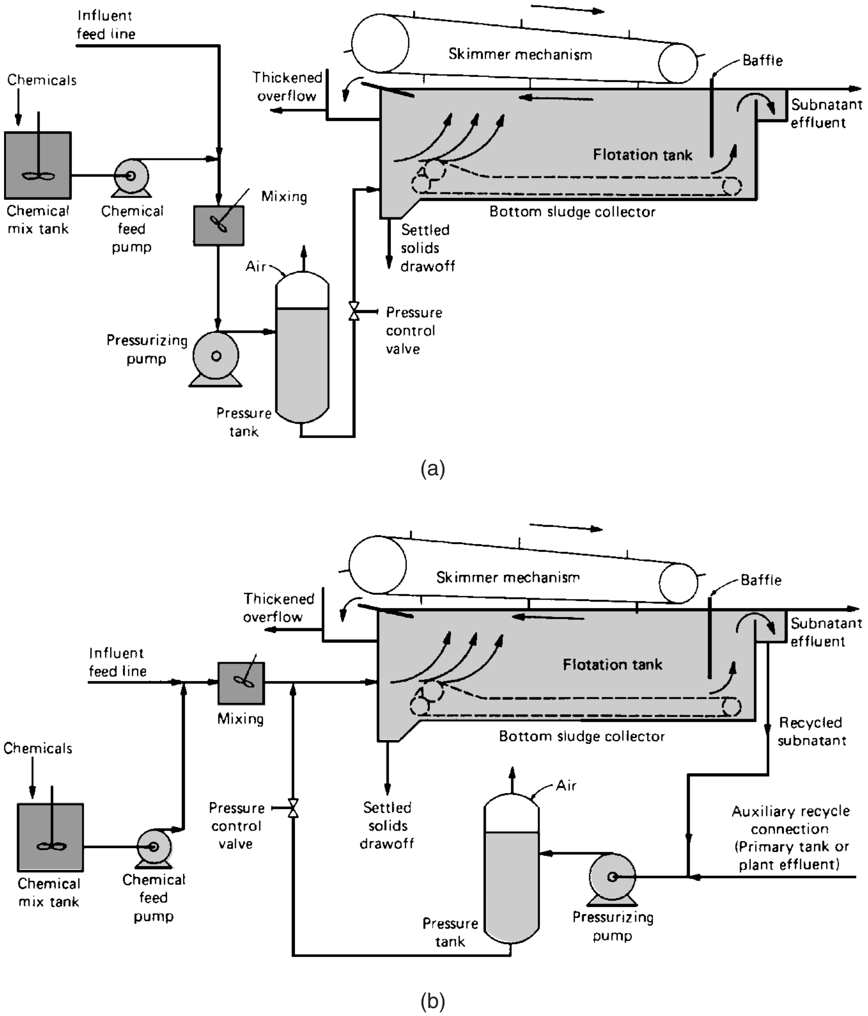


Figure 3.11 (*a*) Schematic diagram of a DAF system without effluent recycle. (*b*) Schematic diagram of a DAF system with effluent recycle.

required for effective DAF thickening can be estimated using knowledge of the sludge influent flow rate (i.e., gallons per minute) and Eq. (3.5):

$$\text{Minimum surface area (ft}^2) = \frac{\text{sludge flow rate (gal/min)}}{\text{hydraulic loading rate (gal/ft}^2 \cdot \text{min)}} \tag{3.5}$$

Equations (3.6) and (3.7) describe the relationship between the DAF design variables for thickening systems with and without pressurized

Figure 3.12 Bench scale system to estimate air-to-solids (A/S) ratio. (*Courtesy of EIMCO Process Equipment Co.*)

recycle. For systems without pressurized recycle (i.e., total influent flow is pressurized), use of the following equation is appropriate for design:

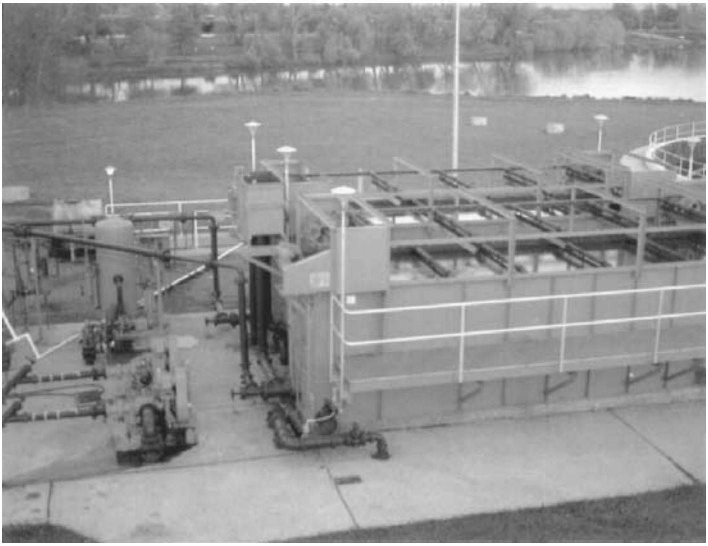
$$\text{A/S} = \frac{1.3s_a\,(fP - 1)}{S_a} \tag{3.6}$$

For systems with only pressurized recycle, the following design equation should be used:

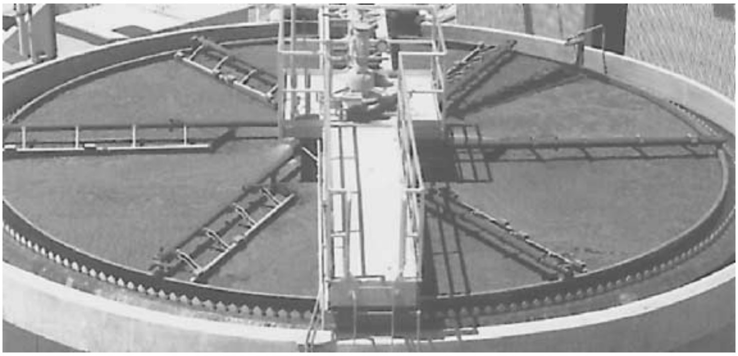
$$\text{A/S} = \frac{1.3s_a\,(fP - 1)\,R}{S_aQ} \tag{3.7}$$

where A/S = air-to-solids ratio, ml (air) /mg (solids)
s_a = air solubility, ml/liter
f = fraction of air dissolved at pressure P, usually 0.5
P = pressure, atm
S_a = influent solids concentration, mg/liter
R = pressurized recycle flow, MG/day
Q = influent flow, MG/day

Example 3.3 illustrates the process of using bench- and/or pilot-scale data for a full-scale DAF design.



(a)



(b)

Figure 3.13 (*a*) Rectangular dissolved-air flotation system. (*Courtesy of EIMCO Process Equipment Co.*) (*b*) Circular dissolved-air flotation system. (*Courtesy of WesTech Engineering Co.*)

Example 3.3 The Tubman County Water Reclamation Plant is presently considering the use of a dissolved-air flotation system at its facility. The combination of primary and secondary sludge results in a total sludge flow of 220,000 gal/day. If the combined sludge has an initial solids content of 0.6 percent and the facility desires to produce sludge with at least a 6.0 percent solids content to discharge to its dewatering system, determine the following design parameters for a DAF system with and without recycle:

1. DAF pressure (atm)
2. Surface area (ft^2)
3. Solids loading rate ($lb/ft^2 \cdot day$)

Assume that through a literature review and bench-scale tests, the following data were generated:

1. Optimum A/S ratio: 0.01 ml/mg
2. Water temperature: 68°F (20°C)
3. Air solubility: 18.7 ml/liter
4. Recycle system pressure: 50 psig
5. Fraction of water saturation: 0.40
6. Surface loading rate: $0.25 \text{ gal/ft}^2 \cdot \text{min}$

solution DAF system without recycle:

Step 1. Compute the required pressure (atm) using Eq. (3.6):

$$\frac{\text{A}}{\text{S}} = \frac{1.3 s_a (f P - 1)}{S_a}$$

$$0.01 = \frac{1.3 \text{ (18.7 ml/liter) } (0.4P - 1)}{6000 \text{ mg/liter}}$$

$$0.4P = 3.468$$

$$P = 8.76 \text{ atm}$$

Step 2. Determine the required surface area:

$$\text{Area} = \frac{\text{volumetric flow rate (gal/day)}}{\text{surface loading rate (gal/ft}^2 \cdot \text{day)}}$$

$$= \frac{220{,}000 \text{ gal/day}}{(0.25 \text{ gal/ft}^2 \cdot \text{day})\,(24 \text{ h/day})\,(60 \text{ min/h})}$$

$$= 611 \text{ ft}^2$$

Step 3. Determine the solids loading rate (SLR, $\text{lb/ft}^2 \cdot \text{day}$):

$$\text{SLR} = \frac{\text{mass flow rate (lb/day)}}{\text{area (ft}^2)}$$

$$= \frac{(0.22 \text{ MG/day})\,(6000 \text{ mg/liter})\,[8.34 \text{ lb/(mg/liter) MG}]}{611 \text{ ft}^2}$$

$$= 18.02 \text{ lb/ft}^2 \cdot \text{day}$$

DAF system with recycle:

Step 1. Determine pressure (atm) in recycle flow:

$$\text{Pressure (atm)} = \frac{\text{recycle pressure (psig)} + \text{atmospheric pressure (psia)}}{\text{atmospheric pressure (psia)}}$$

$$= \frac{50\text{ psig} + 14.7\text{ psia}}{14.7\text{ psia}} = 4.4\text{ atm}$$

Step 2. Determine the required recycle flow rate R using Eq. (3.7):

$$\frac{\text{A}}{\text{S}} = \frac{1.3 s_a\,(fP - 1)\,R}{S_a Q}$$

$$0.01 = \frac{1.3\,(18.7\text{ ml/liter})\,[0.4\,(4.4) - 1]\,R}{6000\text{ mg/liter} \cdot 0.22\text{ MG/day}}$$

$$R = 0.71\text{ MG }(713{,}945\text{ gal/day})$$

Step 3. Determine the required surface area:

$$\text{Area} = \frac{\text{volumetric flow rate (gal/day)}}{\text{surface loading rate (gal/ft}^2 \cdot \text{day)}}$$

$$= \frac{220{,}000\text{ gal/day} + 713{,}945\text{ gal/day}}{(0.25\text{ gal/ft}^2 \cdot \text{day})\,(24\text{ h/day})\,(60\text{ min/h})}$$

$$= 2594\text{ ft}^2$$

Step 4. Determine the solids loading rate (SLR, $\text{lb/ft}^2 \cdot \text{day}$):

$$\text{SLR} = \frac{\text{mass flow rate (lb/day)}}{\text{area (ft}^2)}$$

$$= \frac{(0.22\text{ MG/day})\,(6000\text{ mg/liter})\,[8.34\text{ lb/ (mg/liter) MG}]}{2594\text{ ft}^2}$$

$$= 4.24\text{ lb/ft}^2 \cdot \text{day}$$

3.1.3 Centrifugal thickening

The process of centrifugation achieves liquid-solids separation by the application of centrifugal forces. Although centrifugal thickeners have an advantage of achieving a high degree of thickening, the process is highly mechanized and therefore requires a sophisticated maintenance program. Table 3.4 lists some of the advantages and disadvantages of centrifugal thickeners.

Three basic types of centrifuges are available commercially: (1) the solid-bowl-decanter centrifuge, (2) the imperforate basket-type centrifuge, and (3) the disk-nozzle separator centrifuge. The basic difference between the three types of centrifuges is the method by which solids are collected and discharged from the bowl. Each of these designs is discussed briefly in the following sections.

TABLE 3.4 Advantages and Disadvantages of Centrifugal Thickeners*

Advantages
1. Centrifugal thickeners can treat sludge to varying degrees of thickness.
2. Centrifugal thickeners can concentrate sludge that cannot be treated by conventional means.
3. Centrifugal thickeners take up minimal space.

Disadvantages
1. Operational and maintenance costs are significant.
2. Operation of centrifugal thickeners require skilled operators.

*Adapted from ref. [38].

3.1.3.1 Solid-bowl-decanter centrifuge. The solid-bowl-decanter type of centrifuge consists of an imperforate cylindrical-conical bowl with an internal helical conveyor (Figs. 3.14 and 3.15). The influent sludge enters the cylindrical bowl through the conveyor discharge nozzles. Centrifugal forces concentrates the sludge against the bowl wall, and the internal scroll (i.e., conveyor) transports the thickened sludge along the bowl wall to the conical section, where it is discharged continuously. At the other end of the centrifuge, the centrate is discharged continuously and returned back to the headworks of the wastewater treatment plant.

3.1.3.2 Basket-type centrifuge. In the basket-type centrifuge, the sludge feed is introduced into the bottom of the basket. Solids settle out of the annular moving liquid layer to form a solids compression layer (Fig. 3.16). The compression layer accumulates on the bowl wall while the centrate overflows the lip ring at the top. When sludge solids have filled the basket, the influent sludge feed is stopped, the basket speed is reduced, and the compressed sludge is discharged from the bottom of the casing. The basket-type centrifuge operates at relatively low centrifugal forces and has a discontinuous discharge of both thickened sludge and centrate [39].

3.1.3.3 Disk-nozzle centrifuge. In the disk-nozzle centrifuge, incoming sludge is fed through the top of the device and passed down through a feed well in the center of the rotor (Fig. 3.17). An impeller within the rotor accelerates and distributes the feed slurry filling the rotor interior. The heavier solids settle toward the circumference of the rotor under an increasingly greater centrifugal force. The sludge flows inward through the cone-shaped disk stack.

The clarified liquid (i.e., centrate) passes through the disk stack into the overflow chamber and then is discharged through the effluent line. The centrate normally is returned to the headworks of the wastewater

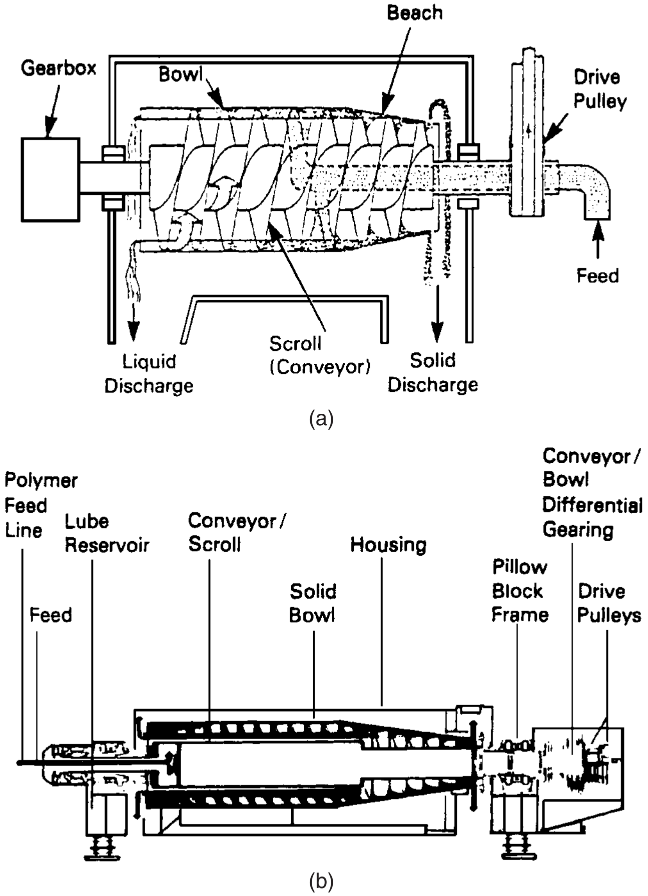
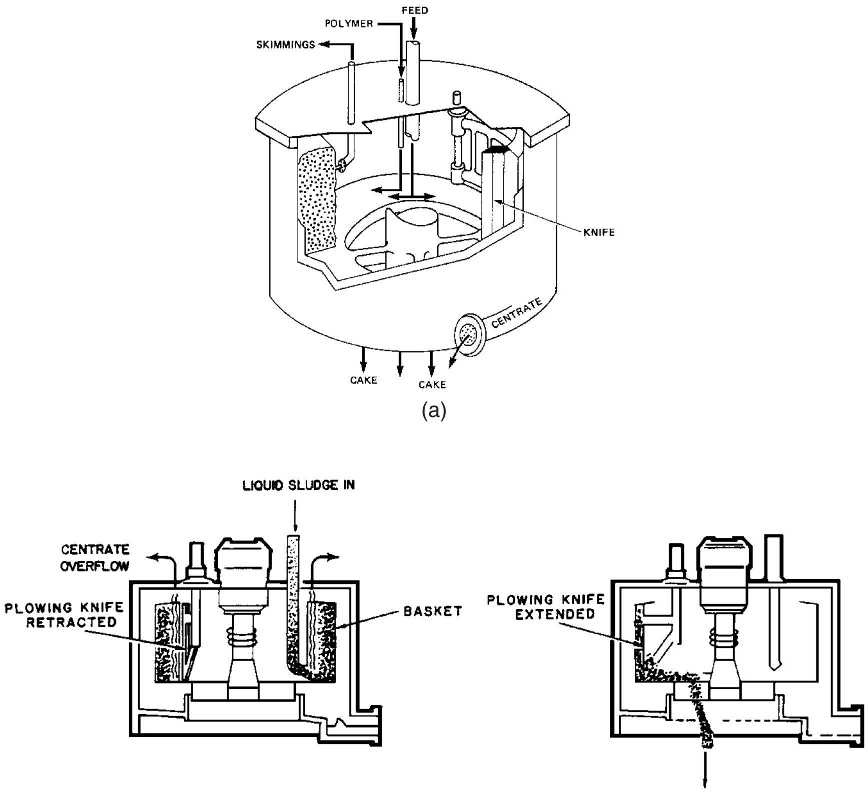


Figure 3.14 Schematic diagrams of solid-bowl-decanter-type centrifugal thickeners: (*a*) countercurrent flow; (*b*) concurrent flow.

Figure 3.15 A solid-bowl-decanter-type centrifuge.



(a)

(b)

(c)

Figure 3.16 Imperforate basket-type centrifugal thickener: (*a*) schematic diagram; (*b*) thickener operating in feed mode; (*c*) thickener operating in plow mode.

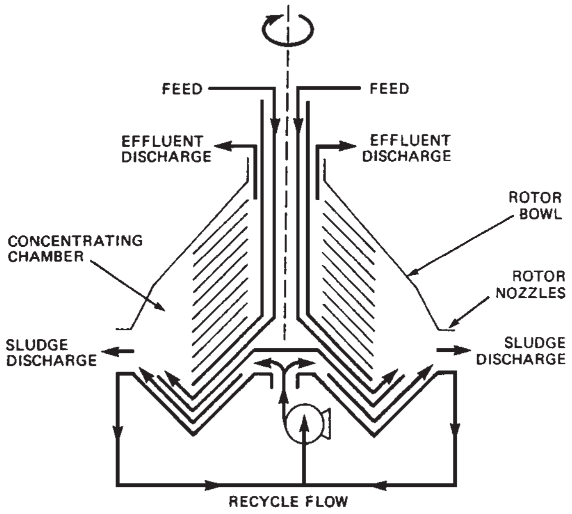


Figure 3.17 Schematic diagram of a disk-nozzle separator-type centrifugal thickener.

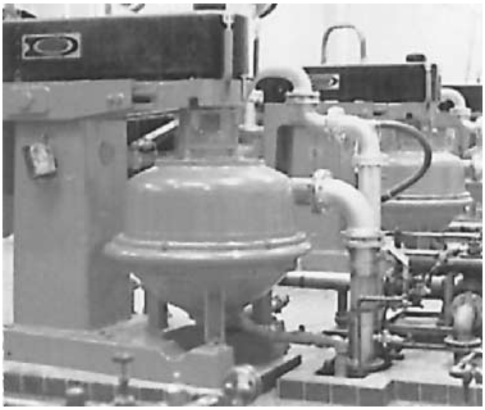


Figure 3.18 A disk-nozzle centrifuge. (*Courtesy of Dorr Oliver.*)

treatment plant for processing. A typical disk-nozzle centrifuge is depicted in Fig. 3.18.

Both general process variables and system design variables affect centrifuge performance. Process variables that have an impact on the performance of all three types of centrifuges include (1) the solids feed rate, (2) sludge characteristics, (3) chemical additions, and (4) temperature [27]. The critical system design variables for the solid-bowl-decanter type of centrifuge include (1) bowl speed, (2) pool volume, and (3) conveyor speed [39]. For the disk-nozzle type of centrifuge, the critical system design variables include (1) bowl speed, (2) recycle mode of operation, (3) disk spacing, and (4) nozzle configuration [39]. Finally, for the imperforate basket-type centrifuge, the critical system design variables include (1) bowl speed, (2) cycle feed time, (3) skimmer nozzle rate, and (4) skimmer nozzle dwell time [38]. Readers interested in the specific impact of system design variables on centrifuge operation should consult the following references or individual centrifuge manufactures [27,38].

In all centrifuge operations, when the sludge feed rate is increased, the retention time decreases, and the solids recovery is reduced. Sludge flow rates typically are limited to 0.5 to 20 gal/min · hp (3.65–14.6 m^3/day · kW) to obtain satisfactory solids recovery. Finally, polymers (e.g., polyelectrolytes) often are used to increase solids capture in the centrifugation process. Because of the increased capture of fine particles, the use of chemicals usually increases the moisture content of the thickened sludge [33].

3.1.3.4 Centrifuge operation. To evaluate the performance of a centrifuge thickener, a simple mass-balance approach can be employed. A control volume for analysis of any centrifuge operation is shown in Fig. 3.19.

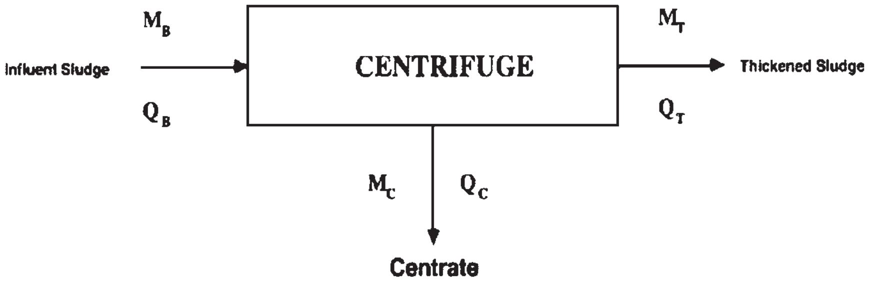


Figure 3.19 Control volume for analysis of centrifuge operation. (M_B = mass of sludge entering the centrifuge, lb/h; M_C = mass of sludge leaving centrifuge in the centrate, lb/h; M_T = mass of sludge leaving centrifuge in thickened sludge flow, lb/h; Q_B = volumetric flow rate of sludge entering centrifuge, gal/h; Q_C = volumetric flow rate of centrate, gal/h; Q_T = volumetric flow rate of thickened sludge flow, gal/h.)

Using the control volume in Fig. 3.19, the mass and volume balance can be described by Eqs. (3.8) and (3.9).

$$M_B = M_C + M_T \tag{3.8}$$

$$Q_B = Q_C + Q_T \tag{3.9}$$

where M_B = mass of sludge entering the centrifuge, lb/h
M_C = mass of sludge leaving centrifuge in the centrate, lb/h
M_T = mass of sludge leaving centrifuge in thickened sludge flow, lb/h
Q_B = volumetric flow rate of sludge entering centrifuge, gal/h
Q_C = volumetric flow rate of centrate, gal/h
Q_T = volumetric flow rate of thickened sludge flow, gal/h

Since the specific gravity of each process flow can be assumed to equal to 1.0 without introducing any significant error, the mass of sludge in each process flow can be estimated using Eq. (3.10):

$$\text{Mass flow rate of sludge} = \text{volumetric flow rate} \cdot \text{concentration}$$

$$M_X = Q_X \cdot C_X \tag{3.10}$$

where M_X = mass flow rate of sludge in process flow X, lb/h
Q_X = volumetric flow rate of sludge in process flow X, gal/h
C_X = concentration of sludge in process flow X, percentage, or mg/liter

Combining Eqs. (3.8), (3.9), and (3.10) results in Eqs. (3.11) and (3.12), which are the mass-balance equations that can be employed in analyzing the performance of any centrifuge system:

$$\frac{M_T}{M_B} = \frac{C_T\,(C_B - C_C)}{C_B\,(C_T - C_C)} \tag{3.11}$$

$$\frac{Q_C}{Q_B} = \frac{C_T - C_B}{C_T - C_C} \tag{3.12}$$

where M_T/M_B = solids capture efficiency, decimal
Q_C/Q_B = fraction of influent flow that appears as centrate (or return flow)
C_B = concentration of solids in influent sludge flow, % or mg/liter
C_T = concentration of solids in effluent sludge flow, % or mg/liter
C_C = concentration of solids in centrate flow, % or mg/liter
Q_c = volumetric flow rate of centrate, gal/h
Q_B = volumetric flow rate of influent sludge, gal/h

For a constant sludge feed concentration C_B, the solids capture efficiency (M_T/M_B) increases as the concentration of solids in the centrate C_C decreases. In thickening sludge, solids capture is critical in minimizing the amount of solids returned to the headworks of the wastewater treatment plant for reprocessing. Examples 3.4 and 3.5 illustrate the use of Eqs. (3.11) and (3.12) in the design and operation of centrifuges. It should be noted that in all cases, bench- and/or pilot-scale tests are recommended to establish the acceptable range for process variables.

Example 3.4 The Wallace County Water Reclamation Facility recently has purchased a disk-nozzle centrifuge to increase the solids content of its thickened sludge from 1 to 8 percent. If the centrifuge manufacturer claims that the solids capture efficiency of the device is 95 percent, what concentration of solids should be expected in the return centrate?

solution The concentration of solids in the return centrate C_C may be estimated by substituting the solids concentrations of the influent and thickened sludge into Eq. (3.11):

$$\frac{M_T}{M_B} = \frac{C_T\,(C_B - C_C)}{C_B\,(C_T - C_C)}$$

$$0.95 = \frac{8\,(1 - C_C)}{1\,(8 - C_C)}$$

or

$$C_C = 0.05674 \text{ percent} \quad (567.4 \text{ mg/liter})$$

Example 3.5 The Wallace County Water Reclamation Facility (see Example 3.4) is concerned that by increasing the solids content of its thickened sludge from 1 to 8 percent the return centrate will overwhelm the existing pump capacity of the centrate sump. If the flow rate of sludge to the new centrifuge system is 400,000 gal/day, estimate the return centrate flow rate (gal/day) if the solids capture of the device is maintained at 95 percent. If the wastewater treatment facility has centrate sump pumps with rated capacities of 100 gal/min, how many pumps must be operating continuously at the plant?

solution

Step 1. The first step is to estimate concentration of solids in the return centrate C_C using Eq. (3.11):

$$\frac{M_T}{M_B} = \frac{C_T\,(C_B - C_C)}{C_B\,(C_T - C_C)}$$

$$0.95 = \frac{8\,(1 - C_C)}{1\,(8 - C_C)}$$

or

$$C_C = 0.05674 \quad (567.4 \text{ mg/liter})$$

Step 2. The fraction of influent sludge flow that appears as return centrate may be estimated by substituting the appropriate solids concentration values into Eq. (3.12):

$$\frac{Q_C}{Q_B} = \frac{C_T - C_B}{C_T - C_C}$$

$$\frac{Q_C}{400{,}000 \text{ gal/day}} = \frac{8 - 1}{8 - 0.05674}$$

$$Q_C = 352{,}467 \text{ gal/day} \quad (244 \text{ gal/min})$$

Step 3. Since a continuous centrate flow of 244 gal/min is produced, three 100 gal/min pumps must be operating continuously. Under normal operations, four 100 gal/min pumps would be installed, with three operating continuously and one maintained in standby mode.

3.1.4 Gravity-belt thickeners

The design and operation of gravity-belt thickeners stem from the use of belt presses in sludge dewatering [38,39]. A gravity-belt thickener consists of a gravity belt that moves over rollers driven by a variable-speed drive unit. The hydraulic loading of sludge to a gravity-belt thickener is expressed in gallons of sludge per minute per meter of belt width (gal/min · m), whereas the solids loading rate is expressed

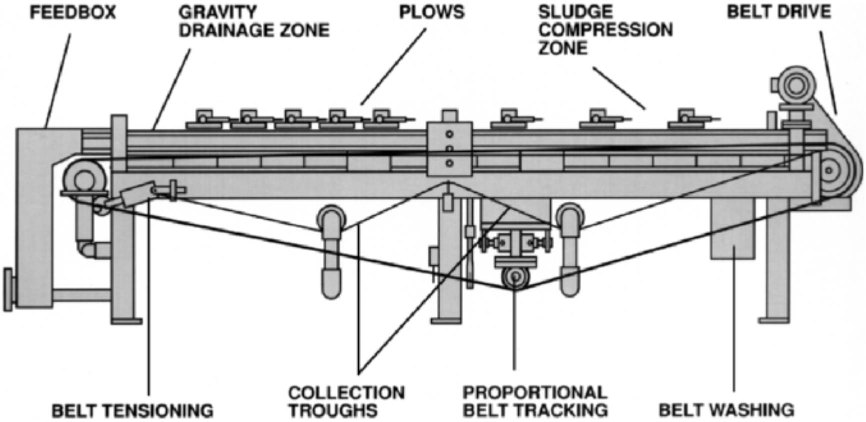


Figure 3.20 Schematic diagram of a gravity-belt sludge thickener. (*Courtesy of EIMCO Process Equipment Company.*)

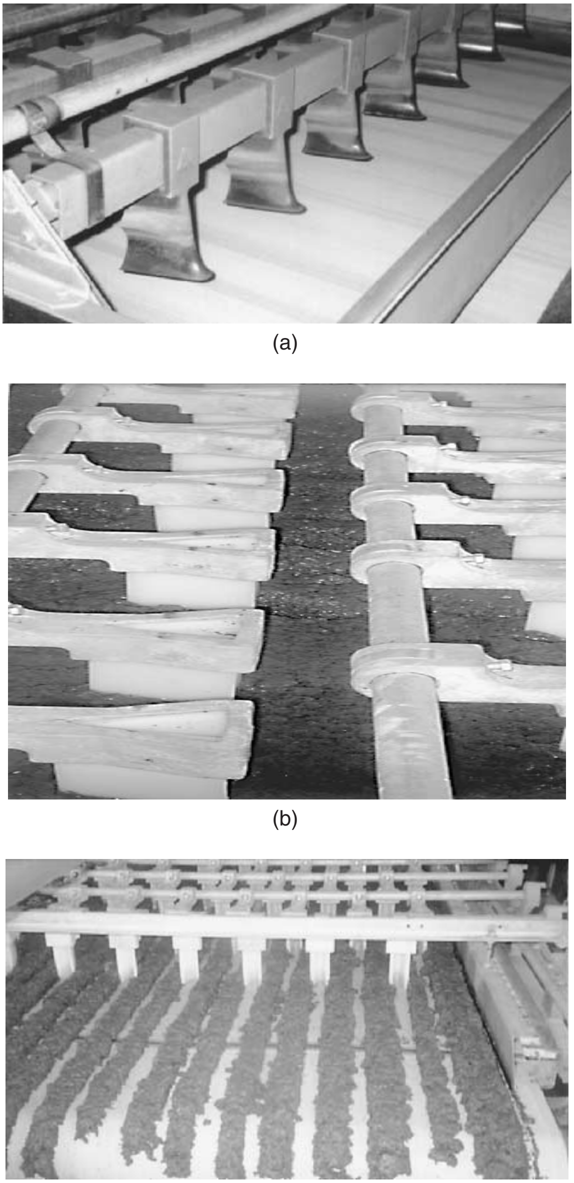
as pounds of sludge per hour per meter of belt width (lb/h · m). A schematic diagram of a gravity-belt thickener is given in Fig. 3.20. Effective thickening of sludge occurs in the gravity drainage section of the belt press. In this zone, water drains freely into a collection pan. Approximately 50 percent of the moisture is lost in the gravity drainage zone, resulting in a doubling of the solids content (Fig. 3.21). In some gravity-belt thickeners, there is the option of increasing the solids content of the resulting sludge by gradually compressing the sludge between two belts [33,38].

During normal gravity-belt thickener operation, the sludge is conditioned with a polymer before being transferred to a feed/distribution box. The feed/distribution box is used to distribute the sludge evenly across the width of the moving belt as the water drains through. As moisture is being lost, the sludge is conveyed toward the discharge end of the thickener. After the thickened sludge is removed from the belt through a series of plow blades, the belt travels through a wash cycle (Fig. 3.22).

Although there are many proprietary designs of gravity-belt thickeners, the most important variables in all operations include (1) sludge characteristics, (2) polymer conditioning, (3) sludge feed rate, (4) belt tension, and (5) belt speed. Additional information on gravity-belt thickeners may be found in the following references [38,39].

3.1.5 Rotary-drum thickening

Rotary-drum thickening consists of rotating cylindrical screens through which moisture is removed as the sludge is conveyed



(a)

(b)

(c)

Figure 3.21 Photographs of components of a gravity-belt thickener: (*a*) plow blades on thickener; (*b*) sludge moving through thickener; (*c*) thickened sludge leaving belt thickener.

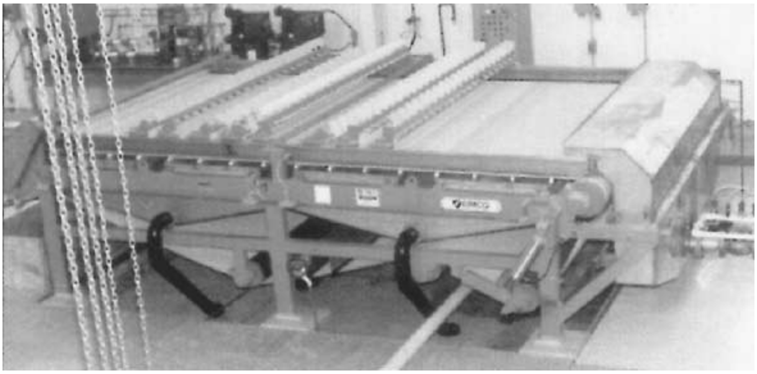


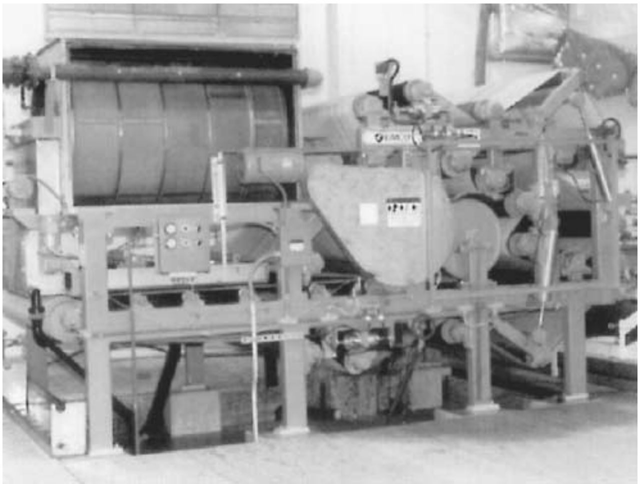
Figure 3.22 A gravity-belt thickener. (*Courtesy of EIMCO Process Equipment Co.*)

through the device. During normal operation, a polymer or lime is added to the influent sludge and mixed in a conditioning drum. The conditioned sludge is then passed to rotating screen drums that separate the flocculated solids from the water. Thickened sludge exits the ends of the drums, while decanted water passes through the screens. A photograph of a rotary drum thickener is shown in Fig. 3.23. Advantages of rotary-drum thickeners include low maintenance, low energy use, and small space requirements [71].

In dissolved-air flotation thickeners, centrifuges, gravity-belt thickeners, and rotary-drum thickeners, polymers often are used to increase particle aggregation and liquid-solids separation efficiency, particularly for waste-activated sludge. For example, when a polymer is used, waste-activated sludge can be thickened to a solids content of 3 to 4 percent in a rotary-drum thickener [71]. Typical doses of polymer used in the thickening of waste-activated sludge for the major types of thickening operations are given in Table 3.5. Actual polymer dosages used in full-scale operation always should be determined in laboratory tests using sludge from the wastewater treatment plant [38].

3.2 Stabilization

After thickening, liquid sludge may be transferred directly to conditioning and dewatering operations, after which it is stabilized, or it may be stabilized in liquid form. *Stabilization* refers to the treatment of sludge to achieve several process objectives, including (1) reduction or elimination of vector attraction, (2) reduction of pathogen concentrations, (3) elimination of offensive odors, and (4) reduction or elimination



(a)

(b)

Figure 3.23 (*a*) A rotary-drum thickener. (*Courtesy of EIMCO Process Equipment Co.*) (*b*) Another rotary-drum thickener. (*Courtesy of HYCOR Corporation.*)

TABLE 3.5 Typical Polymer Doses Used in WAS Thickening*

	Pounds of dry polymer per ton of dry solids		
Sludge type	DAF	Centrifuge	Gravity belt
Waste-activated sludge (WAS)	4–10	2–6	6–14

*Adapted from ref. [38].

of the potential for putrefaction [38,74]. Achievement of these objectives normally requires a reduction in the volatile (i.e., organic) fraction of sludge and/or an alteration of the physical/chemical environment to reduce or eliminate biological activity.

To meet the vector attraction reduction criteria as well as the pathogen concentrations specified in the 40 CFR Part 503 rule, sludge is required to be treated by a stabilization process prior to land application. (Note that once sludge is of a quality to be applied or disposed on land, it is called *biosolids.*) The principal processes used in sludge stabilization include (1) anaerobic digestion, (2) aerobic digestion, (3) lime treatment, (4) chlorine oxidation, and (5) composting. Other stabilization processes such as wet-air oxidation and heat treatment are considered both stabilization and conditioning processes and are covered under sludge conditioning operations later in this chapter.

Depending on the method used, stabilization processes can either reduce or increase the volume of biosolids or sludge [38,74]. The actual amount of stabilized sludge produced in a given stabilization process depends on operational parameters (e.g., chemical addition, temperature, aeration, mixing, pH, detention time, etc.). A description of the impact of the principal sludge stabilization processes on both the resulting sludge volume and beneficial use options will be presented in the following sections.

3.2.1 Sludge volume considerations

The volume of sludge or biosolids that result from any treatment process (including stabilization) will depend on both its moisture content and the characteristics of the solids fraction. A critical parameter affecting sludge volume estimates is the specific gravity. Since the solids portion of sludge consists of both a fixed (i.e., mineral) solids fraction and a volatile (i.e., organic) solids fraction, the specific gravity of the solids portion of sludge may be estimated using Eq. (3.13):

$$\frac{W_s}{S_s \rho_w} = \frac{W_f}{S_f \rho_w} + \frac{W_v}{S_v \rho_w} \tag{3.13}$$

where W_s = weight of solids
S_s = specific gravity of the solids fraction of sludge (or biosolids)
ρ_w = density of water
W_f = weight of fixed (i.e., mineral) solids
S_f = specific gravity of the fixed (i.e., mineral) solids fraction of sludge (or biosolids)
W_v = weight of volatile (i.e., organic) solids
S_v = specific gravity of volatile (i.e., organic) solids fraction of sludge (or biosolids)

Knowledge of the specific gravity of the solids portion of sludge together with the moisture content allows estimation of the specific gravity of the wet sludge using Eq. (3.14):

$$\frac{1}{S_{\text{bio}}} = \frac{P_s}{S_s} + \frac{P_w}{1.0} \tag{3.14}$$

where S_{bio} = specific gravity of wet sludge (or biosolids)
P_s = solids fraction of sludge, decimal
S_s = specific gravity of solid portion of sludge (or biosolids)
P_w = moisture fraction of sludge (or biosolids), decimal
1.0 = specific gravity of water

Example 3.6 illustrates the use of Eqs. (3.13) and (3.14) in estimating the specific gravity of the solids portion of biosolids.

Example 3.6 A 1-lb sample of wet sludge is brought to the laboratory and oven dried at 103°C (217°F). The resulting dry weight of the sludge is 0.09 lb. The oven-dried sludge is then put in a furnace set at 550°C (1022°F). Results indicate that the dry sludge is 70 percent volatile (organic) and 30 percent fixed (mineral) solids. If the specific gravities of the volatile and fixed solids fractions are 1.0 and 2.5, respectively, determine the following:

1. Specific gravity of the dry sludge
2. Specific gravity of the wet sludge

solution

Step 1. Use Eq. (3.13) to estimate the specific gravity of the dry sludge:

$$\frac{W_s}{S_s\rho_w} = \frac{W_f}{S_f\rho_w} + \frac{W_v}{S_v\rho_w}$$

$$\frac{0.09}{S_s} = \frac{(0.09)\,(0.3)}{2.5} + \frac{(0.09)\,(0.7)}{1.0}$$

$$S_s = 1.22$$

Step 2. Since 1 lb of wet sludge contains 0.09 lb of solids, the fraction of solids is 9 percent, and the water fraction is 91 percent. Equation (3.14) can then be used to estimate the specific gravity of the wet sludge:

$$\frac{1}{S_{\text{bio}}} = \frac{P_s}{S_s} + \frac{P_w}{1.0}$$

$$\frac{1}{S_{\text{bio}}} = \frac{0.09}{1.22} + \frac{0.91}{1.0}$$

$$S_{\text{bio}} \cong 1.02$$

Finally, knowledge of the specific gravity of the wet sludge together with its moisture content and dry weight allows estimation of the sludge volume using Eq. (3.15). The volume of sludge is a critical design parameter because it affects the sizing of downstream processing equipment (e.g., storage tanks, pump size, piping, etc.).

$$V = \frac{W_s}{\rho_w S_{\text{bio}} P_s} \tag{3.15}$$

where V = volume of sludge, ft^3
W_s = weight of dry solids, lb
ρ_w = density (specific weight) of water, 62.4 lb/ft^3
S_{bio} = specific gravity of wet biosolids or sludge
P_s = solids fraction of sludge, decimal

Example 3.7 illustrates the use of Eq. (3.15) in estimating the sludge flow rate generated from any treatment process.

Example 3.7 Addis County Wastewater Treatment Plant desires to size its sludge-handling equipment based on the volume of thickened sludge produced from combining the sludge generated in its primary and secondary wastewater treatment operations. From trial tests, it was estimated that a combined weight of approximately 2000 lb of dry sludge would be sent to the thickener from the two wastewater treatment operations. If the sludge thickener will be operated to generate a sludge flow that is 6 percent solids, what is the volume flow rate that downstream sludge-processing equipment must be designed to handle? Assume that the specific gravity of the thickened sludge is estimated to be 1.03.

solution Using Eq. (3.15), the volume flow rate of sludge from the thickener can be estimated:

$$\begin{aligned} V &= \frac{W_s}{\rho_w S_{\text{bio}} P_s} \\ &= \frac{2000 \text{ lb/day}}{(62.4 \text{ lb/ft}^3)\,(1.03)\,(0.06)} \\ &= \frac{518.6 \text{ ft}^3}{\text{day}} \left(\frac{3879 \text{ gal}}{\text{day}}\right) \end{aligned}$$

3.2.2 Anaerobic digestion

Anaerobic digestion is the term used to describe the microbial degradation of organic matter in the absence of molecular oxygen. As a result of the anaerobic microbial reactions, much of the organic matter is converted to methane, carbon dioxide, and water, as described in Eq. (3.16). In addition to the production of methane gas, human pathogens are reduced significantly during the anaerobic digestion process because the anaerobic environment is unsuitable for their survival [16,77].

$$\text{Organic matter} + H_2O \xrightarrow[\text{Anaerobic bacteria}]{} CH_4 + CO_2 + H_2O \quad (3.16)$$

where CH_4 = methane
CO_2 = carbon dioxide

Anaerobic digestion offers several advantages over other stabilization processes, including (1) production of methane (an energy source), (2) reduction in mass of solids, and (3) effective reduction in pathogens. The anaerobic digestion process is mediated by various groups of anaerobic bacteria [38]. However, because of their slow growth rate and sensitivity to environmental conditions, the design of anaerobic digestion processes must consider the growth-limiting characteristics of the methane-forming bacteria (i.e., methanogens).

Although most of the inherent energy associated with the influent organic matter is processed into methane and removed, a small percentage is directed to microbial cell growth. Typical microbial solids yields from the anaerobic treatment of secondary sludge (e.g., from waste-activated sludge systems) range from 0.04 to 0.1 lb volatile solids per pound BOD_5 removed [38]. Since little carbon and energy remain to sustain further microbial activity, the sludge generated in the anaerobic digestion process is considered stable.

The most important operating parameters affecting the performance of anaerobic digesters are the sludge retention time and digestion temperature. As the solid retention time increases, a greater portion of the influent sludge organic matter is converted to gaseous end products (CH_4, CO_2). The maximum reduction in organic matter content that can be achieved by increasing the sludge retention time is approximately 60 percent. Beyond this point, further reduction is minimal, even with substantial increases in the sludge retention time (Fig. 3.24).

3.2.2.1 Anaerobic digestion: Process fundamentals. The design of all microbial processes, including anaerobic digestion, should be based on the fundamentals of biological growth. Maintaining a suitable growth environment for microorganisms is critical for achieving adequate stabilization of thickened sludge. The theoretical approach to anaerobic digester design assumes that a portion of the growth-limiting substrate (i.e., organic matter) is directed toward the production of cell mass (i.e., anaerobic sludge solids) and a portion is directed toward meeting the

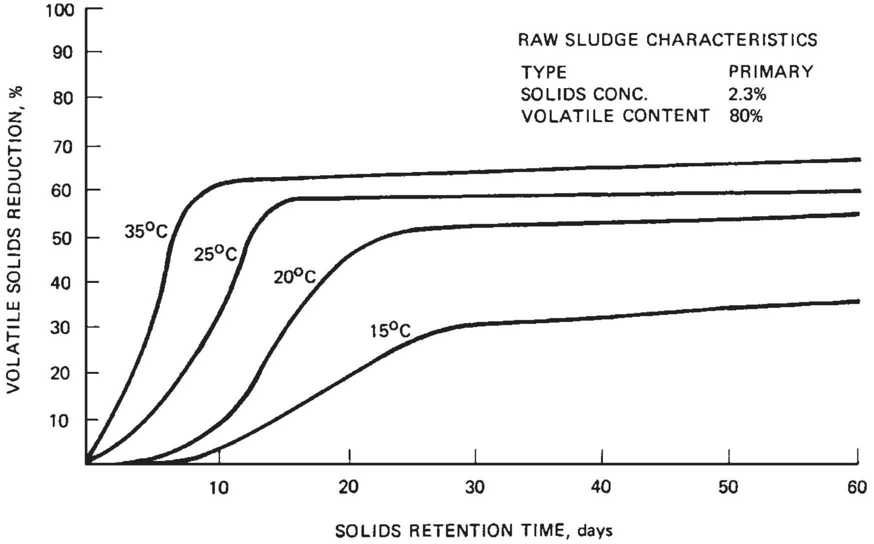


Figure 3.24 Effect of solid retention time and temperature on organic matter reduction.

energy and maintenance needs of the microbial population [33]. Using this principle, the steady-state concentration of microorganisms (X) in a suspended-growth (i.e., completely mixed) reactor system without sludge recycle can be developed [Eq. (3.17)]:

$$X = \frac{Y\,(S_o - S)}{1 + k_d\theta_c} \tag{3.17}$$

where X = concentration of sludge, normally measured as VSS/liter
Y = yield coefficient, mg VSS per mg BOD_L used
S_o = influent organic matter concentration, mg BOD_L/liter
S = influent organic matter concentration, mg BOD_L/liter
k_d = endogenous decay constant, day^{-1}
θ_c = mean cell residence time, days (same as sludge retention time)
VSS = volatile suspended solids (estimate of sludge concentration)
BOD_L = ultimate biochemical oxygen demand, mg/liter (*Note*: $\text{BOD}_L = 1.47\ \text{BOD}_5$ approximately)
BOD_5 = 5-day biochemical oxygen demand, mg/liter

The yield coefficient Y accounts of the portion of the influent ultimate biochemical oxygen demand (BOD_L) that is directed toward the production of cell mass, whereas the endogenous decay constant k_d

TABLE 3.6 Kinetic Coefficients Used in Estimating Sludge Generation*

	Activated-sludge process		Anaerobic digestion	
	Range	Typical	Range	Typical
Y (mg VSS/mg BOD_5)	0.4–0.8	0.6	0.04–0.10	0.06
k_d (day^{-1})	0.025–0.075	0.06	0.02–0.04	0.03

*Adapted from ref. [33].

accounts for the organic matter that is directed toward energy production and the fraction of cell mass that is lost through cell death and predation [22,36]. Typical values for Y and k_d for use in estimating sludge production from the activated-sludge and anaerobic-digestion processes are provided in Table 3.6. The mean cell residence time θ_c is the average time that sludge remains within the treatment system. It should be noted that this term is synonymous with *solids retention time*. For completely mixed systems without recycle, the mean cell residence time is equivalent to the hydraulic retention time (volume of treatment reactor divided by the volume flow rate).

Using Eq. (3.17), the net production of sludge (lb VSS/day, dry-weight basis) may be estimated for any biological treatment system by noting that in suspended-growth reactors without recycle, the effluent sludge concentration is equivalent to the sludge concentration within the reactor [9]. Therefore, the production rate of sludge from any suspended-growth biological treatment process without recycle may be estimated using Eq. (3.18):

$$\begin{aligned} P_x \text{ (lb/day)} &= QX \text{ [8.34 lb/(mg/liter) MG]} \\ &= \frac{QY(S_o - S)}{1 + k_d\theta_c} \, \frac{8.34 \text{ lb}}{\text{(mg/liter) MG}} \end{aligned} \tag{3.18}$$

where P_x = production rate of sludge, lb VSS/day, dry-weight basis
Q = steady-state volumetric flow rate, million gallons per day (MGD)
8.34 = conversion constant, 8.34 lb/[(mg/liter)MG]
MG = million gallons
VSS = volatile suspended solids, mg/liter

Example 3.8 illustrates the use of Eq. (3.18) to estimate the sludge generated from an anaerobic digestion process.

Example 3.8 Jamison County Wastewater Treatment Plant currently produces 6000 gal/day of thickened sludge having a steady-state ultimate biochemical oxygen demand (BOD_L) of 42,000 mg/liter. Effluent BOD_L from the anaerobic digestion process was estimated to be 4500 mg/liter. If the total volume of the anaerobic digester is 150,000 gal and the system is operated without sludge recycle, estimate the mass of sludge produced per day from the anaerobic treatment system. Assume a yield coefficient Y of 0.06 lb/lb and a k_d of 0.03 per day.

solution

Step 1. Estimate the solids retention or mean cell residence time θ_c from knowledge of the anaerobic digester volume and the daily sludge volumetric flow rate:

$$\theta_c = \frac{\text{volume of digester (gal)}}{\text{volumetric flow rate of sludge (gal/day)}}$$

$$= \frac{V}{Q} = \frac{150{,}000\text{ gal}}{6{,}000\text{ gal/day}} = 25\text{ days}$$

Step 2. Estimate the production rate of sludge (dry-weight basis) using Eq. (3.18):

$$P_x\text{ (lb/day)} = \frac{QY\,(S_o - S)}{1 + k_d\theta_c}\;\frac{8.34\text{ lb}}{\text{(mg/liter) MG}}$$

$$= \frac{(6000\text{ gal}/10^6\text{ gal})\,(0.06)\,(42{,}000\text{ mg/liter} - 4500\text{ mg/liter})}{1 + (0.03\text{ day}^{-1})\,(25\text{ days})}$$

$$\cdot\;\frac{8.34\text{ lb}}{\text{(mg/liter) MG}}$$

$$= 64.3\text{ lb VSS/day}$$

3.2.2.2 Biogas production and use. In addition to the low production rate of microbial solids, anaerobic digestion also has the advantage of producing a valuable energy source, i.e., methane (CH_4). Methane is generated as a component of the digester biogas mixture. The biogas mixture typically is 60 to 70 percent methane and 30 to 40 percent carbon dioxide (by volume). Trace amounts of N_2, H_2, H_2S, and water vapor also will be found in the generated biogas.

To estimate the amount of methane that can be generated from the anaerobic treatment of thickened sludge, it is necessary to first determine the mass of organic matter directed toward methane formation. In this approach, it is normally assumed that all the influent organic matter that is not directed to growth of anaerobic microorganisms (volatile suspended solids, VSS) is directed to methane formation. Therefore, the mass of organic matter that is converted to methane may be estimated by subtracting that portion of the influent organic matter that is directed toward anaerobic sludge solids production (i.e., VSS) from the overall organic matter removal [Eq. (3.19)].

Organic matter directed to methane production (lb/day)

= overall removal of organic matter (lb/day)

− organic matter directed to sludge solids production (lb VSS/day)

$$= Q\,(S_o - S)\,\frac{8.34\ \text{lb}}{(\text{mg/liter})\ \text{MG}} - 1.42P_x \qquad (3.19)$$

where Q = steady-state volumetric flow rate, in million gallons per day (MGD)

S_o = influent organic matter concentration, mg BOD_L/liter

S = influent organic matter concentration, mg BOD_L/liter

8.34 = conversion constant, 8.34 lb/(mg/liter)MG

MG = million gallons

P_x = production rate of sludge, lb VSS/day, dry-weight basis

1.42 = BOD_L equivalent of VSS

VSS = volatile suspended solids, mg/liter

Once the amount of organic matter consumed in methane formation is known, the methane equivalent of the organic matter must be estimated. This relationship may be estimated from the stoichiometric oxidation of methane [Eq. (3.20)].

$$\underset{(18)}{CH_4} + \underset{2(32)}{2O_2} \rightarrow CO_2 + 2H_2O \qquad (3.20)$$

From Eq. (3.20), 1 lb · mol of methane is equivalent to 64 lb · mol of oxygen or oxygen demand (BOD_L). Since 1 lb · mol of an ideal gas at standard temperature and pressure [i.e., 32°F (0°C), 1 atm of pressure] occupies 359 ft^3, 5.62 ft^3 of methane is equivalent to 1 lb of BOD_L (i.e., oxygen demand). Using the methane equivalent of BOD_L, the expression that can be employed to estimate the methane production rate from any anaerobic treatment process is given by Eq. (3.21). Example 3.9 illustrates the use of Eq. (3.21) in estimating the biogas production rate from anaerobic digestion.

Methane production rate (ft^3/day) =

$$\frac{5.62\ \text{ft}^3}{\text{lb BOD}_L}\left[Q\,(S_o - S)\,\frac{8.34\ \text{lb}}{(\text{mg/liter})\ \text{MG}} - 1.42P_x\right] \qquad (3.21)$$

where Q = steady-state volumetric flow rate, million gallons per day (MGD)

S_o = influent organic matter concentration, mg BOD_L/liter
S = influent organic matter concentration, mg BOD_L/liter
8.34 = conversion constant, 8.34 lb/(mg/liter)MG
MG = million gallons
1.42 = BOD_L equivalent of VSS
P_x = production rate of sludge, lb/day, dry-weight basis

Example 3.9 Washington County Water Reclamation Facility is considering using anaerobic digestion to stabilize its thickened biosolids. If the flow rate of thickened sludge is 5000 gal/day, estimate the volume of the digester and the volume of biogas produced if the influent sludge BOD_L is 50 g/liter and the effluent sludge BOD_L is 0.37 g/liter. Assume that the system will operate as a continuously mixed reactor without recycle with a mean cell residence time of 30 days and that the biogas is expected to be approximately 70 percent methane. In addition, assume that the following kinetic coefficients apply: $Y = 0.05$ lb/lb, $k_d = 0.02$ per day.

solution

Step 1. For a continuously mixed reactor without recycle, the mean cell residence time is equivalent to the hydraulic retention time. Using this fact, the reactor volume can be estimated as follows:

$$\begin{aligned}\text{Volume} &= \theta_c Q \\ &= (30 \text{ days})\,(5000 \text{ gal/day}) \\ &= 150{,}000 \text{ gal (or } 20{,}000 \text{ ft}^3)\end{aligned}$$

Step 2. Estimate the production of anaerobic biosolids P_x using Eq. (3.18):

$$\begin{aligned}P_x \text{ (lb/day)} &= \frac{QY(S_o - S)}{1 + k_d\theta_c}\;\frac{8.34 \text{ lb}}{(\text{mg/liter}) \text{ MG}} \\ &= \frac{(5000 \text{ gal}/10^6 \text{ gal})\,(0.05)\,(50{,}000 \text{ mg/liter} - 3700 \text{ mg/liter})}{1 + (0.02 \text{ day}^{-1})\,(30 \text{ days})} \\ &\quad \cdot \frac{8.34 \text{ lb}}{(\text{mg/liter}) \text{ MG}} \\ &= 60.3 \text{ lb VSS/day}\end{aligned}$$

Step 3. Estimate the volume of methane produced using Eq. (3.21):

$$\text{CH}_4 \text{ production rate (ft}^3\text{/day)} = \frac{5.62 \text{ ft}^3}{\text{lb BOD}_L}\left[Q\,(S_o - S)\,\frac{8.34 \text{ lb}}{(\text{mg/liter}) \text{ MG}} - 1.42P_x\right]$$

$$= \frac{5.62\ \text{ft}^3}{\text{lb BOD}_L}\left[\frac{5000\ \text{gal/day}}{10^6\ \text{gal}}(50{,}000\ \text{mg/liter} - 3700\ \text{mg/liter}) \cdot \frac{8.34\ \text{lb}}{(\text{mg/liter})\ \text{MG}} - 1.42\ (60.3\ \text{lb/day})\right]$$

$$= \frac{10{,}369\ \text{ft}^3}{\text{day}}$$

Step 4. Estimate the biogas production rate given the fact that the biogas mixture is 70 percent methane (volume basis):

$$\text{Biogas production rate (ft}^3\text{/day)} = \frac{10{,}369\ \text{ft}^3\ \text{methane/day}}{0.70\ \text{ft}^3\ \text{methane/1.0 ft}^3\ \text{biogas}}$$

$$= 14{,}813\ \text{ft}^3\ \text{biogas/day}$$

Biogas production rates may fluctuate over a wide range depending on the volatile solids (i.e., organic matter) content of the influent sludge and variations in digester operation. Total gas production may be estimated by noting that 12 to 18 ft^3 of biogas is produced per pound of volatile solids removed (0.75–1.12 m^3 gas per kilogram of volatile solids removed) [9].

In considering the use of biogas as a fuel source, it is important to note that methane has a net heating value of 960 British thermal units (Btu) per cubic foot (ft^3) at standard temperature and pressure (35,800 kJ/m^3). Since biogas typically is 65 percent methane, the heating value of digester gas is approximately 600 Btu/ft^3 (22,400 kJ/m^3). By comparison, natural gas has a heating value of approximately 1000 Btu/ft^3.

In many small wastewater treatment plants (i.e., less than 5 million gal/day wastewater flow) and in some larger plants with excess biogas production, biogas may be burned directly using a gas flare system with combustion products discharged directly to the atmosphere (Fig. 3.25). In most large wastewater treatment plants, the production rate of biogas may be significant, in which case it may be economically justifiable to use the generated biogas to fuel process boilers or internal combustion engines. It should be noted that biogas may contain significant quantities of hydrogen sulfide (H_2S) and other sulfur-containing compounds that could adversly affect its use as a fuel. These impurities are normally removed prior to use of the biogas in internal combustion engines.

The energy generated from using biogas as a fuel may be used for wastewater pumping, operating blowers, and generating electricity within the wastewater treatment plant. Figure 3.26 shows an internal combustion engine being operated to generate electricity from biogas and a process boiler used to generate hot water for use in a sludge heat exchanger.

Figure 3.25 A flare system used to burn excess digester biogas.

Although it has several distinct advantages over other stabilization processes, anaerobic digestion is also characterized by several disadvantages. The principal disadvantages of stabilizing sludge through anaerobic digestion include the following:

- High capital cost—Large closed tanks fitted with feeding, heating, and mixing systems are required.
- Susceptibility to upsets—Sophisticated process control may be required.
- Large solids retention times are required (15–40 days are typical).

At present, three anaerobic digestion design options are principally used in sludge-stabilization operations: (1) standard-rate digestion, (2) single-stage high-rate digestion, and (3) two-stage high-rate digestion. In addition to these systems, other anaerobic digestion design options include (1) anaerobic contact digestion and (2) phase separation, both of which have been demonstrated to be effective in both laboratory and full-scale tests [9]. However, since the standard-rate, single-stage high-rate, and two-stage high-rate

(a)

(b)

Figure 3.26 (*a*) An electric generator fueled by anaerobic digester biogas. (*b*) A biogas-fired process boiler used to generate hot water for use in heat exchangers.

digestion designs comprise the majority of anaerobic stabilization systems in use today, their design and operational procedures will be the focus of the following sections. Readers interested in other types of anaerobic stabilization systems should review the following references [9,22,36,38].

3.2.2.3 Standard-rate digestion. A standard-rate digester is depicted schematically in Fig. 3.27. In this design, unheated thickened sludge is fed into the digestion tank intermittently. Bubbles of biogas (i.e., CH_4 and CO_2) are generated within the digester, and their rise to the surface provides the only source of mixing [36,38].

As a result of only passive mixing, the contents of the standard-rate digestion tank will stratify, forming three distinct zones that include (1) a floating scum layer, (2) a middle supernatant layer, and (3) a lower layer or zone of sludge. Stabilized sludge, which accumulates and thickens at the bottom of the tank, is drawn off periodically and is stored or conveyed to downstream dewatering processes. The floating scum layer and supernatant are removed periodically through displacement when raw sludge is pumped into the tank. The floating scum layer and supernatant normally are recycled back to the headworks of the wastewater treatment plant. However, in some cases, these side streams must undergo some level of pretreatment prior to their reintroduction to the wastewater treatment system [38].

As a result of solids stratification and limited mixing, no more than 50 percent of the tank volume normally is available for sludge processing in standard-rate anaerobic digestion tanks [36,38]. Because of

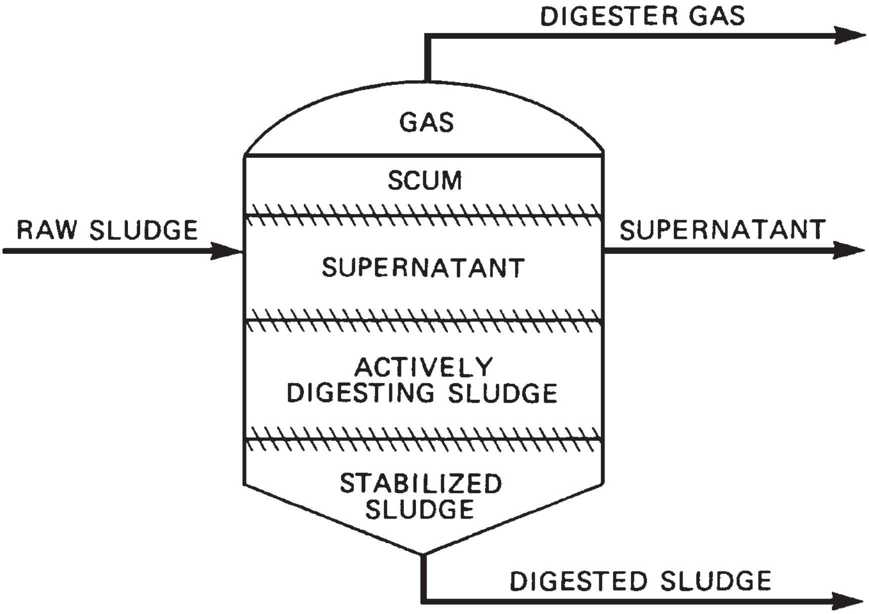


Figure 3.27 Schematic diagram of a standard-rate anaerobic digester.

these limitations, the standard-rate anaerobic digestion process is used primarily in small wastewater treatment plants (i.e., facilities that process less than 2 million gal/day of wastewater).

3.2.2.4 Single-stage high-rate digestion. In the single-stage high-rate anaerobic digestion process, the incoming sludge is heated, actively mixed, and fed uniformly to the digester tank (Fig. 3.28). The net result of these pretreatment steps is a lower required tank volume for a given level of stabilization and increased process stability compared with the standard-rate digestion process [33].

Heating of the incoming sludge is beneficial because the rate of microbial growth and, therefore, the rate of organic matter removal increases with increasing temperature. High-rate digesters normally operate at between 30 and 38°C (86 and 100°F), which is termed *mesophilic digestion.* Some single-stage high-rate systems also are designed to operate at temperatures of 50 to 60°C (122–144°F), which is defined as *thermophilic digestion* [16,24,36].

Although heating to thermophilic conditions requires a greater energy input, some wastewater treatment facilities find that thermophilic conditions are necessary to meet the specific level of pathogen destruction required in the new biosolids regulations (40 CFR Part 503) [59]). In general, the advantages claimed for thermophilic digestion over mesophilic digestion include

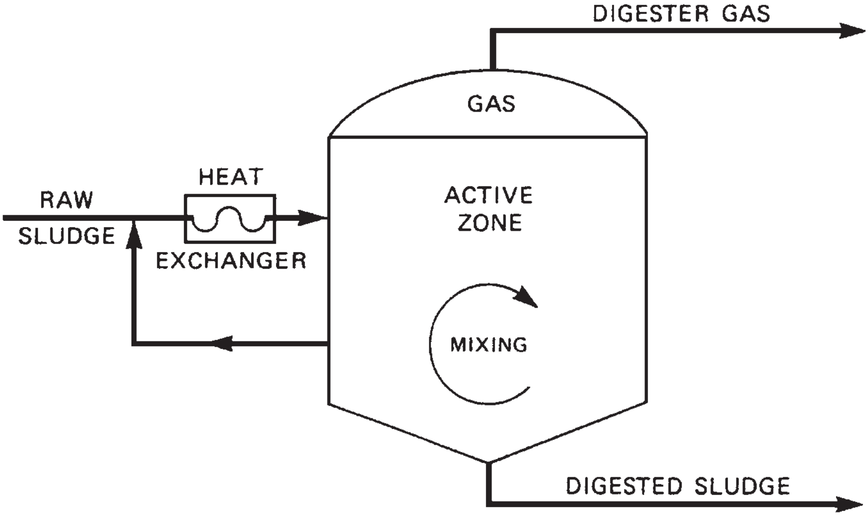


Figure 3.28 Schematic diagram of a high-rate anaerobic digester.

- Enhanced organic matter removal rates
- Improved sludge dewaterability
- Increased destruction of pathogens
- Lower solids detention times

The disadvantages cited for thermophilic anaerobic digestion include (1) increased energy costs and (2) relatively poorer quality side streams requiring treatment [38].

The energy requirements of single-stage high-rate anaerobic digesters consist of the amount of heat needed to (1) raise the temperature of the incoming thickened sludge to the digester tank temperature, (2) compensate for the heat lost to the environment through the digester roof, walls, and floor, and (3) make up for any heat losses that occur within the heat-transfer equipment including piping. Typical methods used to transfer heat to sludge include

- Heat-exchanger coils placed inside the digester tank
- Direct steam injection
- External heat exchangers
- Direct flame heating

Of all the approaches identified, external heat exchangers are the most commonly used sludge heating method (Fig. 3.29). The principal reason for using external heat exchanges is that any repairs or maintenance on the heating system can be performed without taking the digester out of service. Second, since the entire system is readily accessible from outside the tank, the costly and time-consuming process of emptying (or dewatering) the digester is eliminated. And finally, external heat exchangers allow the raw influent sludge to be thoroughly blended with the active digester sludge and preheated outside the

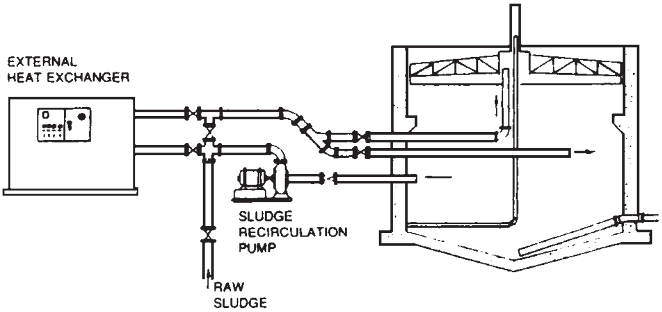
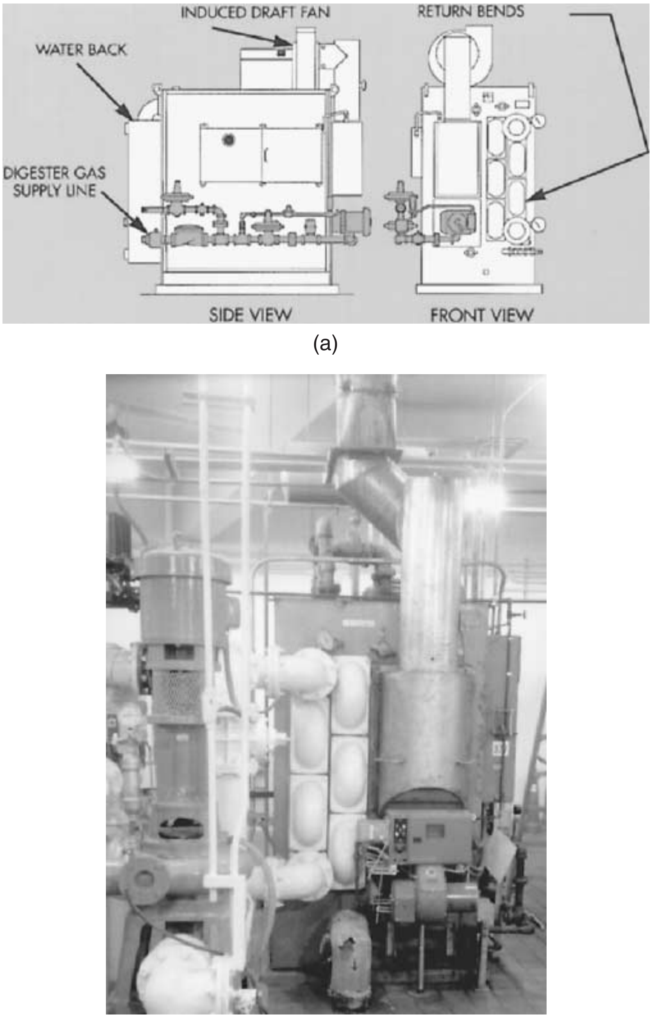


Figure 3.29 Schematic diagram of a typical external heat-exchanger arrangement. (*Courtesy of US Filter/Envirex.*)

digester. This design characteristic ensures against the formation of pockets of cold inactive sludge.

The three principal types of external heater exchangers include (1) water bath, (2) jacketed pipe, and (3) spiral [36,38]. A typical water-bath system (e.g., heater/heat-exchanger system) is illustrated in Fig. 3.30. In this combination system, one side of the unit is used to generate hot water (heat system), which is discharged to the heat-exchanger side, in which pipes carrying digester sludge are immersed.



(a)

(b)

Figure 3.30 (*a*) Schematic diagram of heater/heat-exchange system. (*Courtesy of US Filter/Envirex.*) (*b*) Photograph of heater/heat exchanger used in an anaerobic digestion system.

In a jacketed-pipe heat exchanger, hot water is pumped countercurrent to the sludge flow through a concentric pipe surrounded by the sludge pipe. This type of system (often referred to as a *tube-in-tube heat exchanger*) requires a separate boiler system to supply the hot water used for heating. A typical tube-in-tube heat exchanger used for anaerobic digestion systems is illustrated in Fig. 3.31.

Like the tube-in-tube heat exchanger, the spiral heat exchanger is also a countercurrent flow design. However, the sludge and hot water passageways are cast in a spiral (Fig. 3.32). Some of the benefits claimed for the spiral heat exchanger design over the tube-in-tube heat exchanger include (1) reduced pressure drop, (2) higher heat-transfer rates, (3) lower wall stress (resulting in lower fluid viscosity), and (4) reduced fouling. Moreover, since opening a bolted or hinged door readily exposes the heat-exchange surfaces, the time necessary for cleaning and maintenance are significantly reduced.

Heat-transfer coefficients for the design of external heat exchangers typically range between 150 and 275 $\text{Btu/ft}^2 \cdot \text{h} \cdot {}^\circ\text{F}$ (740 and 1350 $\text{kcal/m}^2 \cdot \text{h} \cdot {}^\circ\text{C}$) depending on heat-exchanger construction and fluid turbulence [38]. A piping arrangement to control the hot water supply to an external heat exchanger is depicted in Fig. 3.33.

In the heating system depicted in Fig. 3.33, hot water is pumped though the heat exchanger and circulated through a secondary heat loop. When the temperature of the sludge leaving the heat exchanger falls below the temperature setpoint, hot water from the primary heat loop is introduced through a modulating valve into the secondary heat loop. The hot water displaces an equal volume of the cooler water back into the primary heat loop. Balancing valves are required in this type of heating system to ensure that the secondary loop will not be bypassed and to allow adjustment of circulating pump capacity.

The sludge pumping rate can be controlled automatically or it can be set for continuous operation. Under automatic control, an adjustable repeating cycle timer periodically starts the recirculation pump, moving sludge from the digester through the heat exchanger. A thermostat located at the inlet to the heat exchanger senses whether the influent sludge requires heat. If the sludge requires heat, the hot water circulation pump operates, thereby applying heat to the sludge. If at the end of the cycle timer period the sludge entering the heat exchanger has not attained the desired temperature, the sludge recirculation pump and the hot water circulation pump continue to operate until the temperature is reached.

If the temperature of the sludge entering is at the desired level, the sludge merely passes through the unit, and the hot water circulation pump does not apply heat. The sludge recirculation pump continues to

Figure 3.31 A tube-in-tube heat exchanger. (*Courtesy of US Filter/Envirex.*)

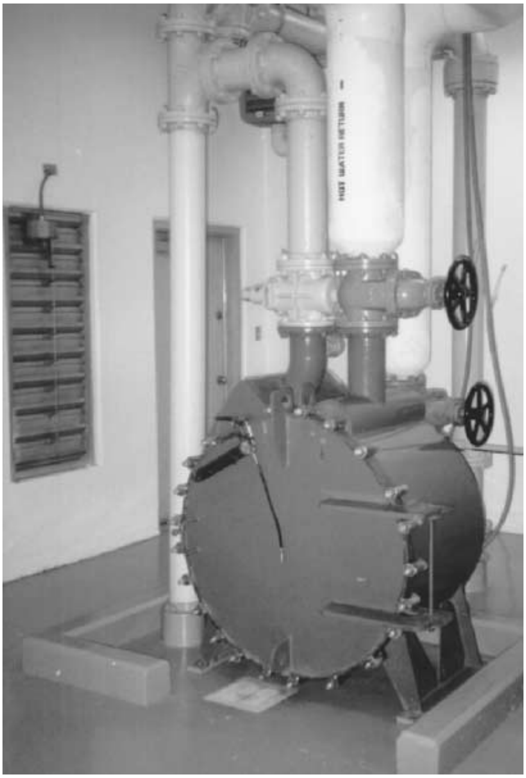


Figure 3.32 Counterflow spiral anaerobic sludge heat exchanger.

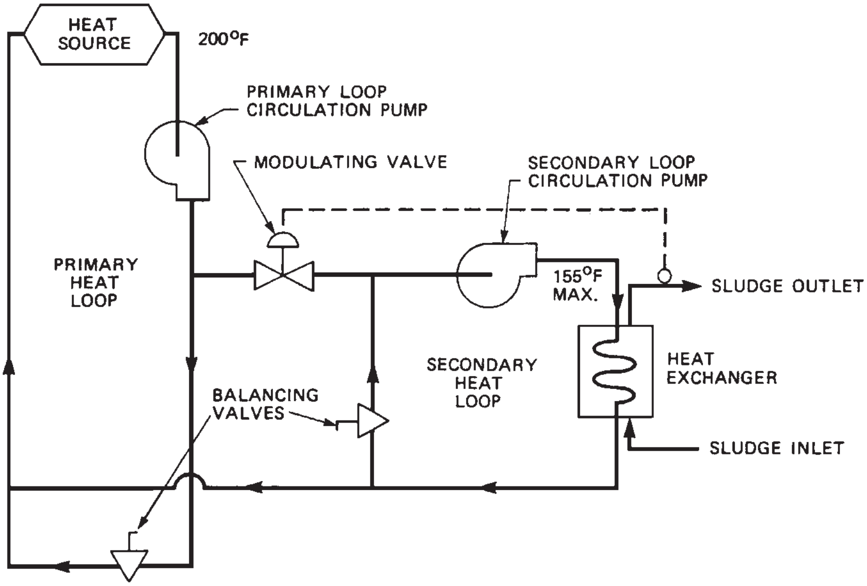


Figure 3.33 Heat-exchanger system design for sludge heating [38].

operate through the time period (typically 15–30 min) to ensure that a representative temperature sample is obtained from the digester. In the continuous pumping mode, the sludge circulation pump operates independent of the sludge thermostat and the repeating cycle time. The thermostat, however, continues to control the operation of the hot water circulation pump to ensure that heat is applied to the sludge only when necessary.

In determining the sludge pumping rate within the heat exchanger, it is important to maintain a sludge velocity that will prevent the accumulation of solids within the heat exchanger. A velocity of 4 ft/s, which is typically used in design, corresponds to a sludge flow of 350 gal/min for a 6-in-diameter pipe.

The hot water used to heat sludge may be generated in a boiler fueled by digester biogas and/or an alternative fuel. Up to 80 percent of the heating value of the digester gas can be recovered in a process boiler [36]. Provisions for burning an alternative fuel (e.g., natural gas) must be included in the boiler design to ensure sufficient heating during periods of low digester gas production or high heating demand. Under normal conditions, the biogas pressure triggers the switching of fuels from digester biogas to an alternative fuel. When the digester gas pressure falls below approximately 4 to 6 in of H_2O, an alternative fuel will be burned within the boiler.

It is critical that the water temperature of the return loop entering the boiler never drops below the dew point of the combustion products within the combustion chambers of the heat exchanger. At or below the dew-point temperature (*ca.* 140°F), moisture forms in the chambers. The hydrogen sulfide present in the digester gas can combine with this moisture to form an acidic solution that greatly accelerates the deterioration of the boiler fire tubes. For this reason, the reclaimed heat water loop should never drop below 160°F (70°C).

To estimate the heating requirement (Btu/h) to raise the temperature of the incoming sludge to the temperature of the digester tank, Eq. (3.22) may be used:

$$Q_{\text{heat}} = \left(\frac{\text{gal of biosolids}}{\text{h}}\right)(C_p)\,(T_2 - T_1)\,(8.34 \text{ lb/gal}) \qquad (3.22)$$

where Q_{heat} = heat required to raise temperature of sludge, Btu/h
C_p = specific heat of sludge, *ca.* 1.0 Btu/lb · °F
T_1 = temperature of raw sludge, °F
T_2 = temperature of digestion tank, °F
Btu = British thermal unit (energy to raise 1 lb of water 1°F)
8.34 = approximate density of sludge (lb/gal)

It should be noted that the solids content of sludge will have a significant impact on the heating requirements. Figure 3.34 illustrates the effect of the sludge solids content on the digester heating requirement.

In addition to the heat required to raise the temperature of the incoming sludge, heat must be added continuously to account for heat losses through the digester surface to the surrounding environment. To

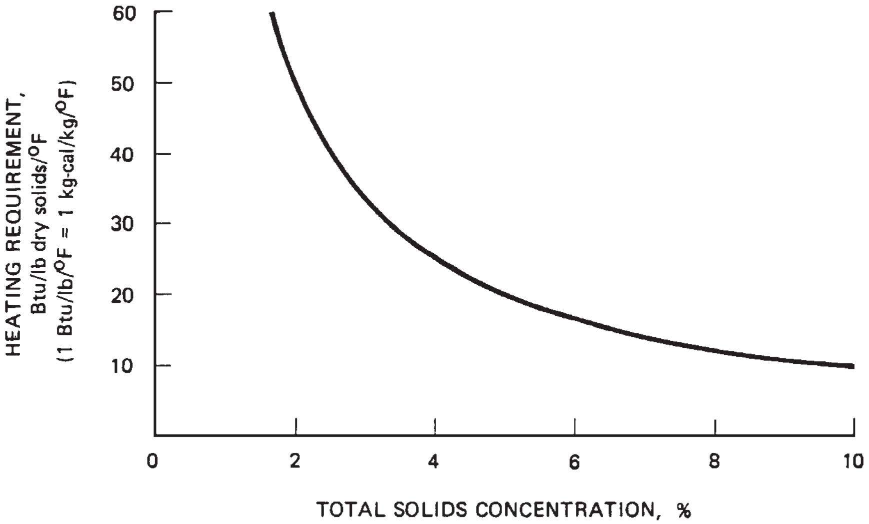


Figure 3.34 Effect of solids content on sludge heating requirement.

TABLE 3.7 Heat-Transfer Coefficients*

	Btu/ft^2 · h · °F
Concrete walls (above ground)	
1 ft thick (uninsulated)	0.83–0.90
1 ft thick with airspace plus brick facing	0.32–0.42
1 ft thick with insulation	0.11–0.14
Concrete walls (below ground)	
Surrounded by dry earth	0.10–0.12
Surrounded by moist earth	0.19–0.25
Concrete floors	
1 ft thick in contact with moist earth	0.10–0.12
1 ft thick in contact with dry earth	0.05–0.07
Floating covers	
With 1.5-in wood deck, built-up roofing, uninsulated	0.32–0.35
With 1.0-in insulating board installed under roofing	0.16–0.18
Fixed concrete covers	
4 in thick and covered with built-up roofing, uninsulated	0.70–0.88
4 in thick and covered, insulated with 1-in insulting board	0.21–0.28
9 in thick, uninsulated	0.53–0.63
Fixed steel covers	
0.25 in thick	0.70–0.95

*Adapted from ref. [38].

estimate the overall heat loss from the anaerobic digester, Eq. (3.23) may be used:

$$Q_{\text{loss}} = UA\,(T_2 - T_3) \tag{3.23}$$

where Q_{loss} = heat loss rate, Btu/h
A = digester surface area normal to direction of heat flow, ft^2
T_2 = temperature within digester tank, °F
T_3 = temperature outside of digester tank, °F
U = heat transfer coefficient, Btu/ft^2 · h · °F

Values of the heat-transfer coefficients for digester wall construction, floor, and covers are provided in Table 3.7. Example 3.10 illustrates the use of Eqs. (3.22) and (3.23) in estimating the overall heating requirements of a high-rate mesophilic digester system.

Example 3.10 Carver County Water Reclamation Plant is considering the purchase of a high-rate anaerobic digester system for stabilization of its thickened sludge. As the sludge engineer at the facility, you have been assigned to estimate the average heating costs to maintain mesophilic temperatures [98°F (36.7°C)] within the digester tank. If the influent sludge has an average temperature of 63°F (17.2°C) and the ambient air temperature at the facility is 67°F (19.4°C), what is the average annual heating cost for the digestion system? Assume that the sludge flow rate is 22,000 gal/day and that your average energy purchase price is $12.50 per decatherm ($10 \cdot 10^6$ Btus). Assume that the digester walls and floor are in contact with the ground, which has an average temperature of 42°F (5.6°C). Specifications for the digester are given as follows:

1. Digester dimensions:
   - Diameter: 80 ft
   - Side depth: 25 ft
   - Middle depth: 30 ft
   - Digester material: Concrete
   - Digester floor: Conical shape
   - Digester cover: Flat circular fixed roof
2. Heat-transfer coefficients:
   - Dry earth embankment for the entire depth and floor: 0.10 $\text{Btu/ft}^2 \cdot \text{h} \cdot {}^\circ\text{F}$
   - Roof exposed to air: 0.15 $\text{Btu/ft}^2 \cdot \text{h} \cdot {}^\circ\text{F}$
3. Specific heat of biosolids: 1 Btu/lb · °F (4200 J/kg · °C)
4. Temperatures:
   - Incoming sludge: 63°F (17.2°C)
   - Average air temperature: 67°F (19.4°C)
   - Average earth temperature: 42°F (5.6°C)

solution

Step 1. Compute the heat required to raise the incoming sludge to the temperature of the digestion tank. NOTE: It is assumed that the density of the sludge is equal to that of water (i.e., 8.34 lb/gal).

$$\text{Heat required (Btu/day)} = \frac{22{,}000 \text{ gal}}{\text{day}} \, \frac{8.34 \text{ lb}}{\text{gal}} \, (98^\circ\text{F} - 63^\circ\text{F}) \left(\frac{1 \text{ Btu}}{\text{lb} \cdot {}^\circ\text{F}} \right)$$

$$= \frac{6{,}421{,}800 \text{ Btu}}{\text{day}}$$

Step 2. Estimate the area of the digester floor, walls, and roof.

Floor—surface area of a cone:

$$\text{Surface area of a cone} = \pi r \, (r^2 + h^2)^{1/2}$$

where r = radius of base (40 ft)
h = depth of cone (5 ft)

$$\text{Surface area of digester floor} = \pi r \, (r^2 + h^2)^{1/2}$$

$$= \pi \, (40) \, (40^2 + 5^2)^{1/2}$$

$$= 5066 \text{ ft}^2$$

Walls—surface area of cylinder:

$$\begin{aligned}\text{Surface area of walls} &= \pi\,(\text{diameter})\,(\text{height})\\ &= \pi\,(80\text{ ft})\,(25\text{ ft})\\ &= 6283\text{ ft}^2\end{aligned}$$

Roof—surface area of circle:

$$\begin{aligned}\text{Surface area of roof} &= \frac{\pi\,(\text{diameter})^2}{4}\\ &= \frac{\pi\,(80)^2}{4}\\ &= 5027\text{ ft}^2\end{aligned}$$

Step 3. Compute the conductive heat losses from the digester.

From floor and wall:

$$\begin{aligned}q &= UA\Delta T\\ &= \left(\frac{0.10\text{ Btu}}{\text{ft}^2\cdot{}^\circ\text{F}\cdot\text{h}}\right)(6283\text{ ft}^2 + 5066\text{ ft}^2)\,(98^\circ\text{F} - 42^\circ\text{F})\\ &= 63{,}544\text{ Btu/h (or }1{,}525{,}306\text{ Btu/day)}\end{aligned}$$

From roof:

$$\begin{aligned}q &= UA\Delta T\\ &= \left(\frac{0.15\text{ Btu}}{\text{ft}^2\cdot{}^\circ\text{F}\cdot\text{h}}\right)(5027\text{ ft}^2)\cdot(98^\circ\text{F} - 67^\circ\text{F})\\ &= 23{,}376\text{ Btu/h (or }561{,}013\text{ Btu/day)}\end{aligned}$$

Total heat losses:

$$\begin{aligned}q &= \frac{6{,}421{,}800\text{ Btu}}{\text{day}} + \frac{1{,}525{,}306\text{ Btu}}{\text{day}} + \frac{561{,}013\text{ Btu}}{\text{day}}\\ &= 8{,}508{,}119\text{ Btu/day (or }3.105\cdot 10^9\text{ Btu/year)}\end{aligned}$$

Step 4. Compute annual sludge heating costs:

$$\begin{aligned}\text{Annual heating costs (\$/year)} &= \frac{3.105\cdot 10^9\text{ Btu}}{\text{year}}\;\frac{\$12.50}{10\cdot 10^6\text{ Btu}}\\ &= \$3881.83\text{/year}\end{aligned}$$

3.2.2.5 Two-stage high-rate digestion. In the two-stage high-rate anaerobic digestion process, a high-rate digester is coupled in series with a second digester tank. Traditionally, the second digester is similar in

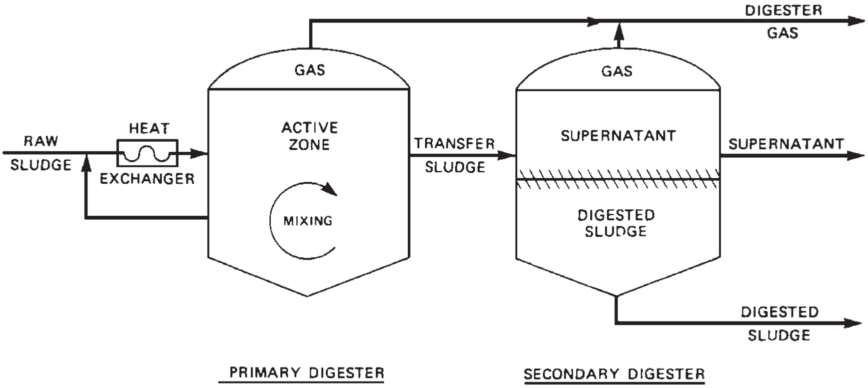


Figure 3.35 Schematic diagram of a two-stage high-rate anaerobic digester.

design to the primary digester except that it is neither heated nor mixed (Fig. 3.35). The primary function of the secondary digester is to allow gravity thickening of the digested sludge solids and decanting of the supernatant liquor.

By employing the secondary digester, the volume of sludge requiring further processing is reduced. However, little solids reduction or gas production occurs within the secondary digester tank.

Unfortunately, many secondary digesters have performed poorly as thickeners and result in the production of a dilute sludge together with a high-strength supernatant (see Chap. 5). The basic cause of the poor performance is that anaerobically digested sludge does not settle readily. The two major reasons for the poor settling of anaerobically digested sludge are (1) flotation of solids due to the attachment of bubbles of biogas and (2) high proportion of fine solids (particularly when waste-activated sludge is digested). Figure 3.36 illustrates the impact of recycling secondary digester supernatant on the solids loadings to the wastewater treatment system.

Although approaches for improving liquid-solids separation efficiency of secondary digesters have been developed, when digesting waste-activated sludge (WAS), it is normally more effective to eliminate the secondary digester altogether or convert it to a completely mixed system. Digested WAS then can be transferred to either a facultative sludge lagoon or mechanical dewatering systems [38]. Since the solids capture is better in these systems than in the secondary digester, their side streams are of higher and more predictable quality compared with the secondary digester supernatant [38].

Despite their operational difficulties in thickening digested WAS, secondary digesters may effectively serve the following functions: (1) thickening digested primary sludge, (2) providing standby digestion capacity (if equipped with the appropriate heating, mixing, and

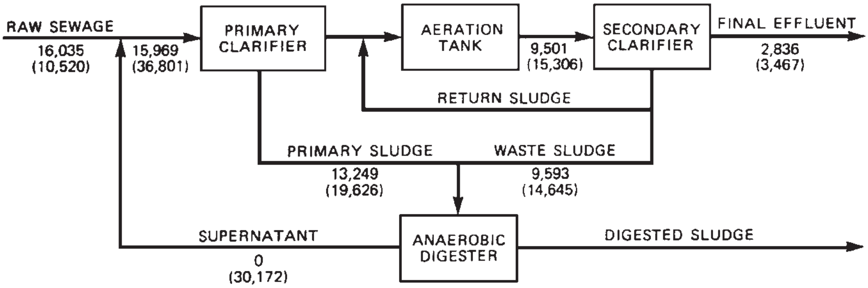


Figure 3.36 Impact of recycling supernatant from secondary digester on solids loadings. NOTE: Data in parentheses were obtained when untreated supernatant was returned to the plant headworks. Data not in parentheses were obtained when no supernatant was recycled. All values are expressed as pounds of suspended solids per day (lb SS/day).

intake piping), (3) providing digested sludge storage (if fitted with a floating cover), (4) ensuring against short circuiting of raw sludge through digestion system (this is particularly important for odor control if digested sludge is to be discharged to a drying bed or lagoon), and (5) providing an extra margin of safety for pathogen destruction (Fig. 3.37).

Recent advances in anaerobic digester technology have led to the development of the temperature-phased anaerobic digestion (TPAD) process. The TPAD process consists of treating thickened sludge at thermophilic [55–65°C (130–150°F)] temperatures in one reactor followed by treatment at mesophilic [30–35°C (86–95°)] temperatures in a second reactor (Fig. 3.38). Advantages claimed in employing the TPAD process over the use of either mesophilic or thermophilic digestion alone for sludge stabilization include (1) increase in overall volatile solids destruction, (2) increased gas production, (3) reduced detention time, (4) reduced sludge disposal costs, (5) reduced odors, and (6) reduced need for biosolids storage [77].

3.2.2.6 Anaerobic digester operational considerations. Fundamentally, the anaerobic digestion process consists of two phases, an organic acid formation phase and methane-producing phase (Fig. 3.39). When an anaerobic digester is working properly, the two phases are in dynamic equilibrium. In other words, the volatile organic acids are converted to methane at the same rate they are formed from the more complex organic molecules (e.g., carbohydrates, fats, and proteins). As a result, volatile acid levels are relatively low in a well-operating system [36,77].

In establishing proper digester operating conditions, it must be recognized that the methane-forming bacteria (i.e., methanogens) are inherently slow growing, with doubling times measure in days [38]. In addition to their slow growth rates, methanogenic bacteria are extremely sensitive to environmental conditions, with small fluctua-

Figure 3.37 A two-stage anaerobic digester system. (*Courtesy of Dorr Oliver.*) (Note the additional gas storage capacity in the digester tank fitted with floating gas-holder cover.)

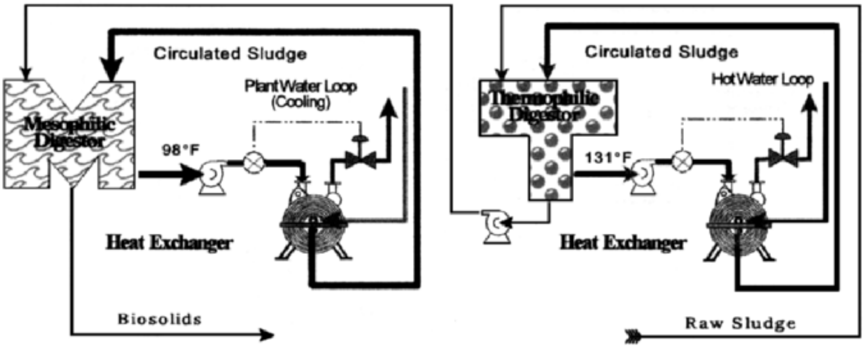


Figure 3.38 Schematic diagram of a temperature-phased anaerobic digestion (TPAD) process. (*Courtesy of Alfa Laval.*)

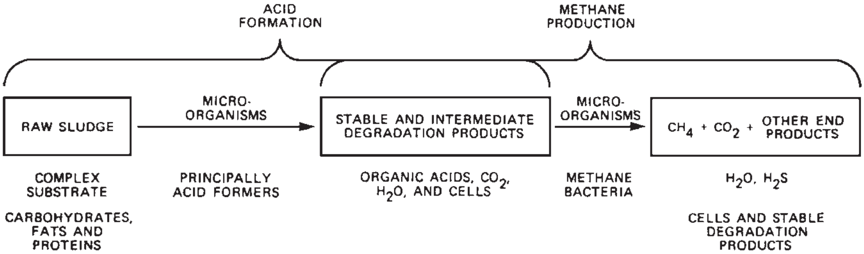


Figure 3.39 Schematic diagram of the two-phase anaerobic digestion process [38].

tions in pH, volatile acid concentrations, or temperature causing severe microbial inhibition [77].

In contrast to methanogens, the acid-forming bacteria can function over a wide range of environmental conditions and have doubling times measured in hours [9]. As a result, when the anaerobic digester is environmentally stressed (e.g., shock organic loads, temperature fluctuations, presence of toxic material, etc.), methane formation lags behind volatile acid formation. When this occurs, organic acids cannot be metabolized by methane bacteria as rapidly as they are formed, leading to organic acid accumulation and potential pH reduction. If the pH is reduced significantly, the methanogens are further inhibited, and the process eventually fails unless corrective action is taken.

To ensure uninterrupted digestion operation, close monitoring of digester pH is necessary for process control. The optimal range of pH for proper anaerobic digester operation is 6.4 to 7.5 [9,38]. Within this range, the carbon dioxide (CO_2)–bicarbonate (HCO_3^-) equilibrium chemistry is responsible for buffering the digester liquor against significant variations in pH [2,38]. As illustrated in Fig. 3.40, the carbon dioxide (CO_2) concentration in the gas phase and the bicarbonate alkalinity concentration in the liquid phase establish the digester pH under equilibrium conditions.

With regard to daily digester operation, the pH variation is normally less important than maintaining a sufficient level of bicarbonate alkalinity. The bicarbonate alkalinity in a well-operating anaerobic digester should be maintained at approximately 2000 to 3000 mg/liter

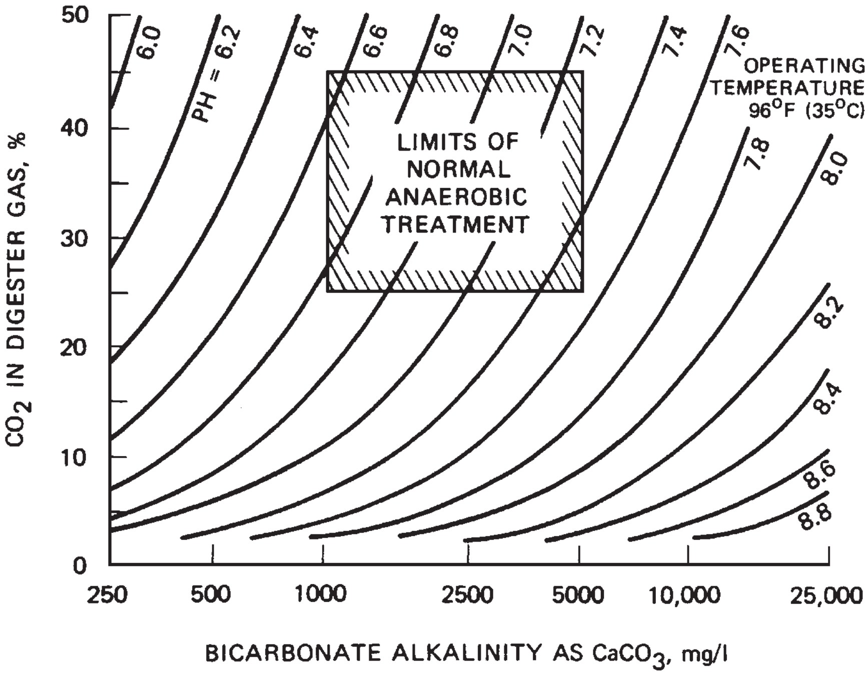


Figure 3.40 Impact of gaseous CO_2 content and alkalinity on digester pH.

(measured as $CaCO_3$). When the digester becomes environmentally stressed, it is the bicarbonate alkalinity (i.e., buffering capacity) that protects the methanogenic bacteria against the organic acid accumulations that can cause severe upset and process cessation. In other words, by maintaining an adequate buffering capacity, the digester pH remains stable and the system becomes less susceptible to failure.

The most appropriate approach for maintaining digester pH and increasing buffering capacity is by the addition of sodium bicarbonate ($NaHCO_3$). Other materials, such as caustic soda, soda ash, and lime [CaO or $Ca(OH)_2$], cannot increase bicarbonate alkalinity without reacting with soluble CO_2, which causes a partial vacuum to occur within the digester. Moreover, above a pH of 6.3, lime will react with bicarbonate alkalinity to form insoluble calcium carbonate ($CaCO_3$), which promotes scale formation and encrustation [36,38]. For any fixed gas phase CO_2 composition, the amount of sodium bicarbonate required to achieve the desired pH change is given by Eq. (3.24). Example 3.11 illustrates the use of Eq. (3.24) in estimating chemical buffering dosages to anaerobic digesters.

$$\text{Dose (mg/liter)} = 0.6\,(\text{bicarbonate alkalinity at final pH} - \text{bicarbonate alkalinity at initial pH}) \qquad (3.24)$$

where dose = dose of sodium bicarbonate, mg/liter

0.6 = ratio of bicarbonate (HCO_3) to $CaCO_3$ molecular weights

bicarbonate alkalinity = measured as mg/liter $CaCO_3$

Example 3.11 The Tororo City Wastewater Treatment Plant currently operates a 200,000-gal mesophilic anaerobic digester to recover energy from its primary and secondary sludge. Due to the industrial discharges of food-processing waste, the wastewater plant supervisor desires to increase both the pH and the buffering capacity of the digester system to protect it from possible upset. If it is desired that the biogas from the digester have an average CO_2 concentration of no more than 35 percent, estimate the amount of sodium bicarbonate that must be added to increase the pH from 6.8 to 7.3.

solution

Step 1. From Fig. 3.40, estimate the bicarbonate alkalinity for the digester mixed liquor if it is in equilibrium with a biogas having 35 percent CO_2. The bicarbonate alkalinity at the two pH levels in question is given in the following table:

pH	Bicarbonate alkalinity (mg/liter $CaCO_3$)
6.8	1600
7.3	4200

Step 2. Using Eq. (3.24), estimate the concentration of sodium bicarbonate necessary to increase the pH from 6.8 to 7.3:

$$\text{Dose (mg/liter)} = 0.6\,(4200\text{ mg/liter} - 1600\text{ mg/liter})$$

$$= 1560\text{ mg NaHCO}_3\text{/liter}$$

Step 3. Estimate the mass of sodium bicarbonate that must be added to the digester:

$$\text{Mass of NaHCO}_3\text{ added, lb} = \frac{200{,}000\text{ gal}}{1 \cdot 10^6\text{ gal}} \cdot \frac{1560\text{ mg NaHCO}_3}{\text{liter}} \cdot \frac{8.34\text{ lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= 2602\text{ lb NaHCO}_3$$

3.2.2.7 Heavy metal toxicity in anaerobic digestion. Heavy metal toxicity frequently has been cited as the cause of anaerobic digester failures. Heavy metals that cannot be detected in the influent wastewater can be concentrated to measurable levels in the raw sludge discharged to digesters. In evaluating the potential adverse impacts of heavy metals on digester operation, it is important to note that it is only the soluble fraction of the heavy metals that is toxic to anaerobic microorganisms. Total and soluble heavy metal concentrations are significantly different within anaerobic digesters because anions such as carbonate and sulfide can effectively remove heavy metals from solution by precipitation and sequestering [23,36,38]. Table 3.8 illustrates the difference between total and soluble concentrations of heavy metals found in anaerobic digesters.

Except for chromium, heavy metal toxicity in anaerobic digesters can be prevented or eliminated by sulfide precipitation [23,38]. Under normal conditions, sulfate contained in the raw sludge is reduced to sulfide within the digester, resulting in heavy metal precipitation. If sufficient

TABLE 3.8 Typical Ranges of Total and Soluble Heavy Metal Concentrations Found in Anaerobic Digesters*

Metal	Total concentration (mg/liter)	Soluble concentration (mg/liter)
Chromium(VI)	88–386	0.03–3.0
Copper	27–196	0.10–1.0
Nickel	2–97	0.00–5.0
Zinc	11–390	0.10–0.7

*Adapted from ref. [38].

sulfate is unavailable within the raw sludge influent, it can be added in the form of metal sulfates. A potential drawback of adding sulfate is the production of excess dissolved sulfide or hydrogen sulfide in the digester biogas [23,24]. Because of the adverse impacts of excessive sulfide, it is recommended that ferrous sulfate be used as the supplementary sulfate source. Any excess sulfide produced in the reduction of sulfate will be precipitated from solution as iron sulfide. Although hexavalent chromium [chromium(VI)] cannot be removed effectively through sulfide precipitation, within the reducing environment of the anaerobic digester, hexavalent chromium is converted to trivalent chromium, which is relatively insoluble and nontoxic [77].

3.2.2.8 Anaerobic digester mixing. Proper mixing is one of the most important considerations in achieving optimal anaerobic digester performance. When stratification is prevented, the entire digester volume is available for stabilization. Methods used for mixing the contents of anaerobic digesters include (1) gas mixing systems, (2) mechanical mixing systems, and (3) mechanical pumping systems. Schematic diagrams of nine types of digester mixing systems are depicted in Fig. 3.41. The advantages and disadvantages of each type of mixing design are summarized in Table 3.9.

Anaerobic digester gas mixing systems involve the recirculation of biogas and are classified as *unconfined* or *confined.* Unconfined gas mixing systems are designed to collect biogas at the top of the digesters, compress the biogas, and then discharge it through a series of bottom diffusers or top-mounted lances.

In confined gas mixing systems, biogas is collected at the top of the anaerobic digesters, compressed, and discharged through confined tubes (see Fig. 3.41). The two major types of confined gas mixing systems are the biogas lifter system and biogas piston system [38,46]. The biogas lifter system consists of submerged biogas pipes or lances inserted into an eductor tube or biogas lifter. Compressed biogas is released from the tubes, and as the biogas bubbles rise, they mix the contents of the anaerobic digester. In the biogas piston system, biogas bubbles are released intermittently at the bottom of a cylindrical tube. The bubbles rise and act like a piston, pushing sludge solids to the surface.

Mechanical mixing systems use low-speed turbine impellers to achieve sludge mixing (see Fig. 3.41). Mechanical mixers can be installed through the digester cover or through the walls of the tank. Wall installations restrict maintenance and repair to the time when the digester has been emptied. Low-speed turbine systems normally have one cover-mounted motor with two turbine impellers located at different depths along the turbine shaft [38,46]. Turbine mixing systems are suitable for anaerobic digesters with either fixed or floating covers.

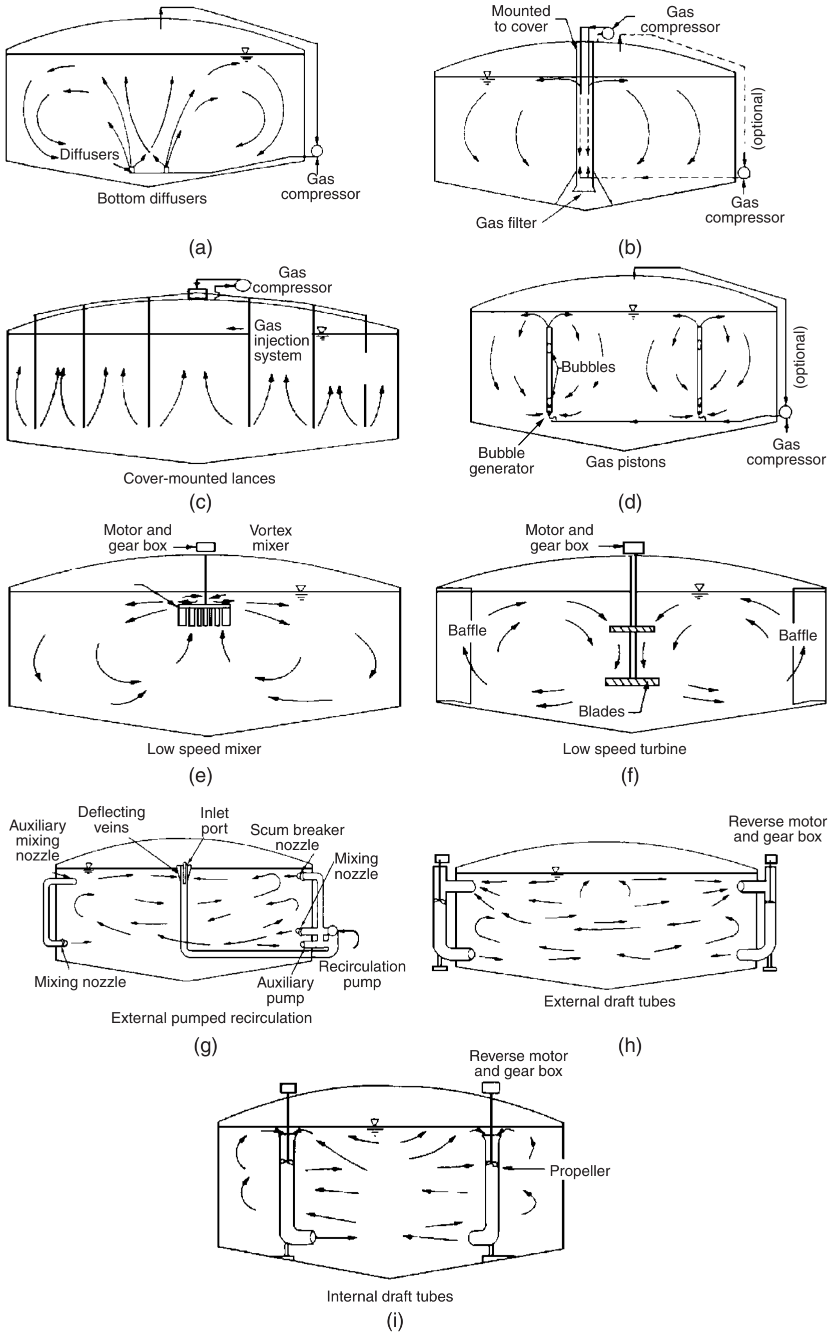


Figure 3.41 Schematic diagrams of digester mixing systems: (*a*) unconfined gas injection; (*b*) confined gas injection; (*c*) unconfined gas injection; (*d*) confined gas injection; (*e*) mechanical stirring, low-speed mixer; (*f*) mechanical stirring, low-speed turbine; (*g*) mechanical pumping, external pumped recirculation; (*h*) mechanical pumping, external draft tubes; (*i*) mechanical pumping, internal draft tubes [38,46].

TABLE 3.9 Advantages and Disadvantages of the Principal Digester Mixing Systems*

Mixer type	Advantages	Disadvantages
All systems	Increased rate of sludge stabilization	Corrosion and wear of ferrous metal piping and supports
	Gas injection	
Unconfined		
Cover-mounted lances	Lower maintenance and less hindrance to cleaning	Corrosion of gas piping and equipment
Bottom-mounted lances	Better movement of bottom deposits than cover-mounted system	Corrosion of gas piping and equipment
Confined		
Gas lifters	Better mixing than cover-mounted lances; lower power requirement	Corrosion of gas piping and equipment, high maintenance for compressors
Gas pistons	Good mixing efficiency; better movement of bottom deposits than cover-mounted system	Corrosion of gas piping and equipment, high maintenance for compressors
	Mechanical stirring	
Low-speed turbines	Good mixing efficiency	Wear of impellers and shafts, bearing failures
Bottom-mounted lances	Breaks up scum layers	Not designed to mix entire tank
	Mechanical pumping	
Internal draft tubes	Good top-to-bottom mixing	Sensitive to liquid level, wear of impellers and shafts, bearing failures
External draft tubes	Good top-to-bottom mixing	Sensitive to liquid level, wear of impellers and shafts, bearing failures
Pumps	Pumps easier to maintain than compressors	Wear of impellers and shafts, bearing failures

*Adapted from ref. [46].

Mechanical pumping systems, which are suitable for anaerobic digesters equipped with fixed covers, consist of either a propeller-type pump mounted in internal or external draft tubes or an axial or centrifugal pump installed externally (see Fig. 3.41). Mixing is achieved by circulation of the anaerobic liquor [46,77]. The general design parameters for all three mixing type systems, gas flow, mechanical mixing, and mechanical pumping systems, are summarized in Table 3.10.

TABLE 3.10 Design Parameters for Anaerobic Digester Mixing Systems*

Parameter	Description	Typical values
Unit power	Motor power of mixing in hp (or watts) divided by digester volume	$0.2–0.3\ \text{hp}/10^3\ \text{ft}^3$ ($5–8\ \text{W/m}^3$)
Unit gas flow	Quantity of gas delivered by gas injection system in ft^3/min (m^3/min) divided by digester volume	Free lift systems $4.5–5.0\ \text{ft}^3/10^3\ \text{ft} \cdot \text{min}$ ($4.5–5.0\ \text{m}^3/\text{km}^3 \cdot \text{min}$)
		Draft tube systems $5–7\ \text{ft}^3/10^3\ \text{ft} \cdot \text{min}$ ($5–7\ \text{m}^3/\text{km}^3 \cdot \text{min}$)
Velocity gradient	Square root of the ratio of power used per unit volume divided by sludge viscosity	All mixing systems $50–80\ \text{s}^{-1}$

*Adapted from ref. [46].

3.2.2.9 Digester mixing efficiency. The three basic design criteria used to size gas mixing systems are: (1) the unit power (power per unit volume), (2) the velocity gradient (G-value), and (3) the unit gas flow (gas flow per unit volume). The size of the unit power mixing criterion stems from the observation that the relative mixing effectiveness is a function of the total power expended. This unit power criterion is expressed in terms of the motor horsepower used to drive the gas compressor. Less power is actually delivered to the liquid because of losses in the mixing system (e.g., frictional losses, compressor inefficiency, etc.).

When gas is discharged into a digester, liquid flow results from the transfer of energy from the gas to the liquid as the gas expands isothermally and rises to the surface [46]. Ignoring the kinetic energy of the gas, the rate of power transferred from the gas to the liquid may be expressed by Eq. (3.25):

$$E = 2.40 P_1 Q \ln\left(\frac{P_2}{P_1}\right) \tag{3.25}$$

where E = rate of power (energy) transferred, $\text{ft} \cdot \text{lb}_f/\text{s}$
Q = gas flow rate, cubic foot per minute, ft^3/min
P_1 = absolute pressure at liquid surface, psia
P_2 = absolute pressure at the depth of gas injection, psia

Once the power transferred to the digester liquid E is estimated through Eq. (3.25), the power dissipated per unit volume W can be estimated by dividing the rate of power transferred by the digester volume V. Example 3.12 illustrates the use of Eq. (3.25) in evaluating the power transferred to the contents of an anaerobic digester from a gas mixing system..

Example 3.12 The anaerobic digester operated by Tororo City (see Example 3.11) is currently using biogas recirculation to mix the digester contents. Estimate the rate of power transferred to the liquid if the gas recirculation rate is 200 actual cubic feet per minute (acfm) at a pressure of 10 psi (gauge). Assume that the liquid surface of the digester is under 8 in of water pressure (8 in H_2O) and that atmospheric pressure is 14.7 psia.

solution

Step 1. Convert both the liquid surface and the incoming biogas pressures to psia.

Liquid surface pressure:

$$\begin{aligned}\text{Pressure (psia)} &= 8\ \text{in}H_2O\ \frac{14.7\text{psi}}{32.0\ \text{ft}\ H_2O\ (12\ \text{in/ft})} \\ &\quad + 14.7\ \text{psi (atmospheric conditions)} \\ &= 15.0\ \text{psia}\end{aligned}$$

Incoming biogas pressure:

$$\begin{aligned}\text{Pressure (psia)} &= 10\ \text{psi} + 14.7\ \text{psi (atmospheric conditions)} \\ &= 24.7\ \text{psia}\end{aligned}$$

Step 2. Using Eq. (3.25), estimate the energy input to the digester through gas recirculation:

$$\begin{aligned}E &= 2.40 P_1 Q \ln (P_2/P_1) \\ &= 2.40 \cdot 15.0\ \text{psia} \cdot 200\ \text{ft}^3/\text{min} \ln (24.7\ \text{psia}/15.0\ \text{psia}) \\ &= 3591\ \text{ft} \cdot \text{lb}_f/\text{s}\end{aligned}$$

Step 3. Estimate the rate of power transferred by gas recirculation per unit volume:

$$\begin{aligned}\text{Work per unit volume}\left(\frac{\text{ft} \cdot \text{lb}_f}{\text{gal} \cdot \text{s}}\right) &= \frac{E}{V} = \frac{3591\ \text{ft} \cdot \text{lb}_f/\text{s}}{200{,}000\ \text{gal}} \\ &= 0.018\ \text{ft} \cdot \text{lb}_f/(\text{gal} \cdot \text{s})\end{aligned}$$

Another estimate of gas mixing effectiveness is the measurement of the velocity gradient G that is created by the gas flow. The velocity gradient is given by Eq. (3.26):

$$G = \sqrt{\frac{W}{\mu}} \tag{3.26}$$

where G = velocity gradient, (ft/s)/s (s^{-1})

W = power dissipated per unit volume, $E/V = \dfrac{(\text{ft} \cdot \text{lb}_f)/\text{s}}{\text{ft}^3}$

E = rate of power (energy) transferred, $(\text{ft} \cdot \text{lb}_f)/\text{s}$

V = volume of digester, ft^3

μ = absolute viscosity of the liquid, $(\text{lb}_f \cdot \text{s})/\text{ft}^2$

The velocity gradient is a more refined design criterion for mixing than the unit-power criterion in that it takes into account the power actually transferred to the liquid as well as the liquid viscosity. The difficulty in using Eq. (3.26) arises from the fact that temperature, solids concentration, and volatile matter content all affect sludge viscosity. Although digester temperature is relatively constant, the solids content and volatile matter content change continuously during digester operation. Figure 3.42 illustrates the general influence of total solids concentration and volatile matter content on sludge viscosity measured using a bench-scale viscometer [38].

Although assigning a minimum velocity gradient necessary to achieve adequate mixing in anaerobic digesters is difficult, the general range used in practice is 50 to 80 s^{-1} [38]. A design value at the high end should be selected for a large digester with only a single mixer or in the case where grit or scum problems appear likely. A lower velocity gradient is appropriate for design in cases where several mixers are distributed throughout the tank or where sufficient detention time has been provided to allow a slower rate of digestion. The use of a two-speed compressor provides the capability to match mixing intensity with variations in operating conditions [46].

Since the gas flow through a mixing system can be related to the mixing energy delivered to the liquid, a simple approach to sizing a gas mixing system is to specify a unit gas flow. For a draft-tube mixer system, 5 to 7 $\text{ft}^3/1000\ \text{ft}^3 \cdot \text{min}$ (5–7 $\text{m}^3/\text{km}^3 \cdot \text{min}$) at about 6 psig (41.4 kN/m^2) is sufficient to produce adequate mixing. Less gas is required for a free-lift system (e.g., 4.5–5.0 $\text{ft}^3/1000\ \text{ft}^3 \cdot \text{min}$ or 4.5–5.0 $\text{m}^3/\text{km}^3 \cdot \text{min}$). However, the pressure must be higher in the free-lift system because the gas is discharged at the bottom of the tank (Fig. 3.43).

The unit gas flow can be related to the velocity gradient G by combining Eqs. (3.25) and (3.26) to produce Eq. (3.27). Example 3.13 illustrates the use of Eq. (3.27) in estimating the required unit gas flow rate to produce a desired velocity gradient.

$$\frac{Q}{V} = \frac{G^2 \mu}{P_1 \ln (P_2/P_1)} \tag{3.27}$$

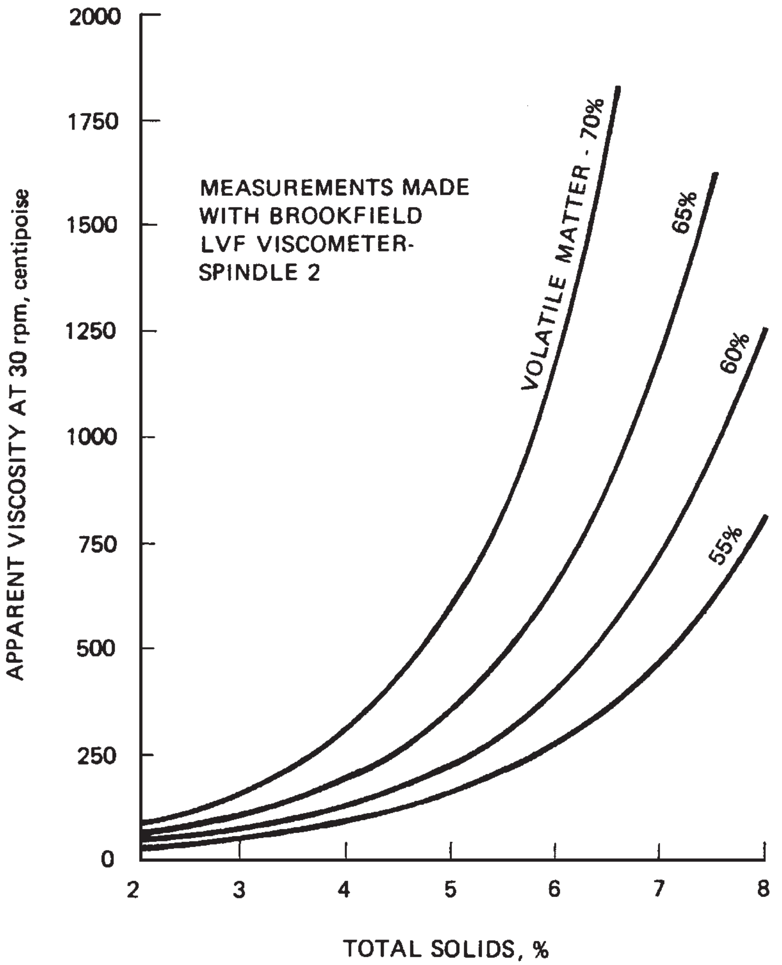


Figure 3.42 Effect of total solids and volatile matter on sludge viscosity. NOTE: Centipoise = $2.08 \cdot 10^{-5}$ $\mathrm{lb_f \cdot s/ft^2}$

where Q = gas flow rate, $\mathrm{ft^3/min}$
V = volume of digester, $\mathrm{ft^3}$
P_1 = absolute pressure at liquid surface, psia
P_2 = absolute pressure at the depth of gas injection, psia
G = velocity gradient, (ft/s)/s ($\mathrm{s^{-1}}$)
μ = absolute viscosity of the liquid, $(\mathrm{lb_f \cdot s})/\mathrm{ft^2}$

Example 3.13 The anaerobic digester mixing system presently operated by Tororo City (see Example 3.12) has been found to be inadequate. The plant supervisor estimates that the gas recirculation rate for the digester should

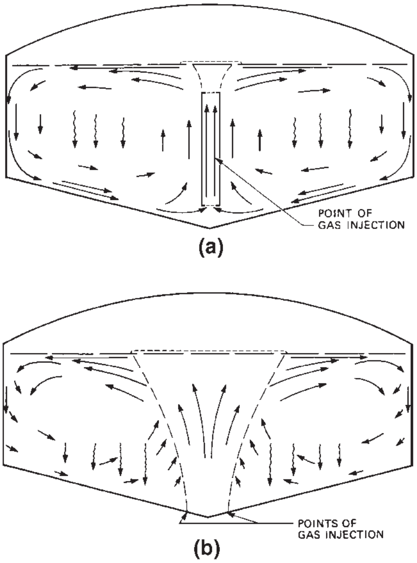


Figure 3.43 Mixing patterns in (*a*) a draft-tube gas mixer and (*b*) a free gas lift system [46].

be redesigned to achieve a velocity gradient of at least 60 s^{-1} to ensure adequate mixing. Assuming that the surface and inlet gas pressures do not change, estimate the minimum gas flow rate in cubic feet per minute per cubic foot of digester (cfm/ft^3) and the total volumetric flow rate of gas (cfm) needed to achieve the desired velocity gradient. Assume that the following conditions apply:

1. G: 60 s^{-1}
2. P_1: 15.0 psia
3. P_2: 24.7 psia
4. μ: $1.9 \cdot 10^{-5}$ lb · s/ft^2

solution

Step 1. Using Eq. (3.27), estimate the minimum gas recirculation rate:

$$\frac{Q}{V} = \frac{G^2\mu}{P_1 \ln (P_2/P_1)}$$

$$= \frac{(60\ \text{s}^{-1})^2 \cdot (1.9 \cdot 10^{-5}\ \text{lb} \cdot \text{s/ft}^2)}{(15\ \text{psia}) \cdot (24.7\ \text{psia}/15.0\ \text{psia})}$$

$$= 0.0091\ \text{ft}^3/\text{min}/\text{ft}^3$$

Step 2. Estimate the gas recirculation rate for the digester:

Gas recirculation rate (ft^3/min)

$$= 200{,}000 \text{ gal} \cdot \text{ft}^3/7.48 \text{ gal} \cdot \frac{0.0091 \text{ ft}^3 \text{ gas/min}}{\text{ft}^3 \text{ of digester}}$$

$$= 243.3 \text{ ft}^3/\text{min} \left(\frac{9.1 \text{ ft}^3/\text{min}}{10^3 \text{ ft}^3 \text{ digester volume}} \right)$$

Like gas mixing systems, the power input per unit digester volume may be used to estimate the mixing effectiveness for mechanical mixing systems. Various types of mixing impellers are illustrated in Fig. 3.44.

For a desired velocity gradient *G,* Eq. (3.27) can be rearranged to estimate the power requirement for the mixer motor [Eq. (3.28)]:

$$G = \sqrt{\frac{W}{\mu}} = \sqrt{\frac{E/V}{\mu}} \qquad \text{or} \qquad E = \mu G^2 V \tag{3.28}$$

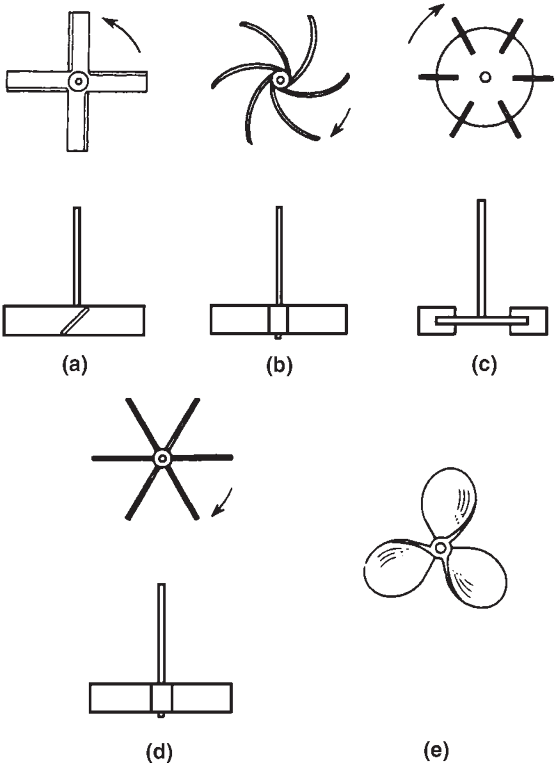


Figure 3.44 Various types of mechanical mixing systems: (*a*) pitched-blade turbine; (*b*) vertical curved-blade turbine; (*c*) bladed disk turbine; (*d*) open straight-blade turbine; (*e*) three-blade marine impeller.

where E = rate of power (energy) transferred, $(\text{ft} \cdot \text{lb}_f)/\text{s}$
G = velocity gradient, (ft/s) /s (s^{-1})
W = power dissipated per unit volume, $E/V = \dfrac{(\text{ft} \cdot \text{lb}_f)/\text{s}}{\text{ft}^3}$
V = volume of digester, ft^3
μ = dynamic viscosity of the liquid, $(\text{lb}_f \cdot \text{s})/\text{ft}^2$

The power requirement can be expressed as a function of rotational speed, size, and type of mixer using either Eq. (3.29*a*) or Eq. (3.29*b*). It should be noted that the choice of using Eq. (3.29*a*) or (3.29*b*) depends on knowledge of the fluid mixing characteristics in the digester, which is defined by the Reynolds (Re) number [Eq. (3.30)]. For laminar flow mixing, the Reynolds number should be 10 or less, whereas for turbulent flow conditions, the Reynolds number is 10,000 or more. For situations where the mixing conditions are between 10 and 10,000, an alternate approach should be employed [38,46].

$$E = k\mu n^2 D^3 \qquad \text{(laminar flow)} \tag{3.29a}$$

$$E = k\rho n^3 D^5 \qquad \text{(turbulent flow)} \tag{3.29b}$$

where E = rate of power (energy) transferred, $(\text{ft} \cdot \text{lb}_f)/\text{s}$
μ = dynamic viscosity of the liquid, $(\text{lb}_f \cdot \text{s})/\text{ft}^2$
n = rotational speed, rev/s
D = diameter of impeller, ft (m)
ρ = density of sludge, slug/ft^3 (kg/m^3)
k = constant (see below)

k* Values as a Function of Impeller Type and Mixing Characteristics

Impeller type	Laminar mixing	Turbulent mixing
Propeller, square pitch, three blades	41.0	0.32
Propeller, pitch of two, three blades	43.5	1.00
Turbine, six flat blades	71.0	6.30
Turbine, six curved blades	70.0	4.80
Fan turbine, six blades	70.0	1.65

*Adapted from ref. [38].

$$\text{Reynolds number (Re)} = \frac{D^2 n\rho}{\mu} \tag{3.30}$$

Pumped circulation, while relatively simple, requires large fluid flow rates to achieve adequate digester mixing. This mixing method is effective if sufficient energy (0.2–0.3 hp/1000 ft^3, 5–8 W/m^3) is dissipated within the digester tank [38,46]. Greater pumping power will be required if piping friction losses are significant. Pumped circulation is used most advantageously in combination with other mixing systems. In addition to mixing, pumped circulation allows external heat exchangers to be used for heating the digester and for uniform blending of raw sludge with heated circulating sludge prior to the raw sludge entering the digester.

Design problems associated with mixing systems include unbalanced gas flow to lances, misapplication of cover-mounted mixers, poor sludge compaction, inadequate number of low-speed mixers, lack of mixing system in secondary digesters, lack of access space for cranes, and improper pump selection [46]. Each of these problems is addressed in the following section.

3.2.2.10 Mixing design concerns. It is difficult to balance gas flow to cover-mounted lances when gas is discharged simultaneously to every lance. Pressure losses to each lance must be equal or gas will flow to the lance(s) with the least pressure drop. Plugged lances will receive no gas flow, whereas broken lances will receive high gas flows. As a result of the imbalance of gas flow, mixing will be inefficient. Design of these types of mixing systems should include isolation valves for each lance to facilitate inspection and hydraulic balancing to evenly distribute the gas flow. Sequentially fed lance systems do not experience the problems of simultaneously fed lances. In these mixing systems, each lance receives full gas flow for a timed increment. Gas flow can only enter the lance being fed so that each lance receives full flow regardless of pressure drop [27,46].

The decision to install cover-mounted mixers (e.g., gas lifters, lances, or mechanical stirrers) on floating or gas-holder covers depends on digestion function and the expected sludge level variation [38,46]. As the covered-mounted system rises, bottom mixing may be reduced. Design of cover-mounted mixing systems should be based on the maximum sludge depth. Installation of mixing equipment on gas-holder covers is not advisable due to the large distance traveled by these systems [36,46].

In mixing systems with bottom-mounted diffusers, it may be difficult to obtain well-compacted sludge. Sludge will be kept in suspension by the gas flow and will not thicken if the diffusers are located too close to the sludge-withdrawal lines. As the sludge accumulates, the heavier solids can plug the diffusers. These problems can be eliminated by relocation of either the diffusers or the sludge-withdrawal lines [46]. Shutting off the mixing system 4 to 12 hours before sludge withdrawal can enhance thickening.

A single low-speed mixer is not adequate to mix the contents of a digester but may be adequate to break up the floating scum layer. To provide proper mixing in existing installations, either more mixers should be added or the system should be replaced. In plants where the two-stage digester process is conducted using two separate tanks, both digesters should be equipped with a mixing system so that either one can function as a single-stage digester if the other tank is out of service.

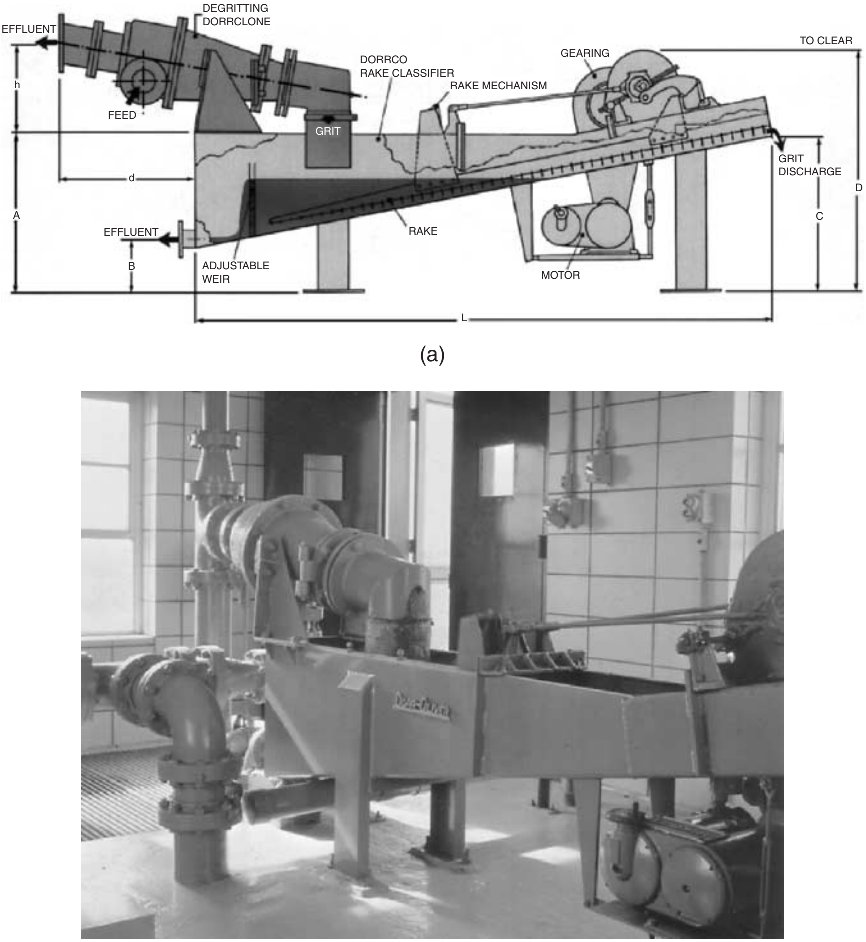
Finally, improper selection of recirculation pumps can cause ineffective mixing because of inadequate pump capacity. Selection of pumps should be based on the anticipated range of sludge characteristics, as well as estimated pumping friction losses [27,38].

In addition to improperly designed mixing systems, operational problems can occur due to the accumulation of grit and screenings within the digester [36]. Grit can cause excessive wear on all ferrous metal equipment in anaerobic digesters and can accumulate within the digestion tank, reducing its capacity. To minimize these operation problems, grit and screenings removal should be critically evaluated when designing anaerobic digestion systems. Although employing a grit chamber at the plant headworks is effective in reducing the overall grit content in downstream processes, smaller grit particles (particularly those having a significant organic matter content) may escape removal in these units and cause problems in downstream sludge-treatment operations. Where the grit content of primary or secondary sludge is a concern for subsequent sludge processing, consideration should be given to the use of a hydroclone system for grit removal. Hydroclones use centrifugal force to separate the lighter, grit-free organic matter from the heavier grit particles. The lighter fraction typically is sent to a thickener and then to digestion, whereas the heavier grit is rinsed and drained before being sent to a landfill. A typical hydroclone system is illustrated in Fig. 3.45.

Other equipment problems common in digester mixing systems include compressor failures, gas/mechanical seal failures, bearing failures in pumps and motors, and breakage of gas piping. Most of these equipment problems can be eliminated by minimizing grit in the digester, removing moisture from the recycled digester gas, and implementation of a regularly scheduled maintenance program (e.g,. oil changing, lubrication of bearings, etc.).

3.2.2.11 Sizing of anaerobic digesters. Determination of tank volume is a critical step in the design of anaerobic digesters. The size of the digester must be sufficient to prevent the process from failing under all expected operational conditions. Under normal conditions, *process failure* is defined as the accumulation of volatile acids to the point where the volatile acid–alkalinity ratio is greater than 0.5 and/or the cessation of methane production has occurred [33,38].

The most important consideration in sizing an anaerobic digester is that the methanogenic bacteria must be given sufficient time to reproduce. Therefore, the ideal design parameter for anaerobic digestion is the *mean cell residence time* (MCRT) (note that in some older texts this



(a)

(b)

Figure 3.45 (*a*) Schematic diagram of a hydroclone system used for grit removal from sludge (DorrClone cyclone). (*Courtesy of Dorr-Oliver.*) (*b*) Photograph of a DorrClone cyclone. (*Courtesy of Dorr-Oliver.*)

is referred to as the *solids retention time,* SRT). Since the MCRT can be defined operationally as the total solids mass in the treatment system divided by the quantity of solids withdrawn daily, it reflects the average time that microorganisms remain in the system. For an anaerobic digester system operated without solids recycle, the MCRT is equivalent to the hydraulic retention time (i.e., volume of digester/sludge flow rate). Figure 3.46 illustrates the relationship between the MCRT and the performance of a bench-scale anaerobic digester fed primary sludge.

As illustrated in Fig. 3.46, volatile solids removal, organic matter removal [as measured by the chemical oxygen demand (COD)], and

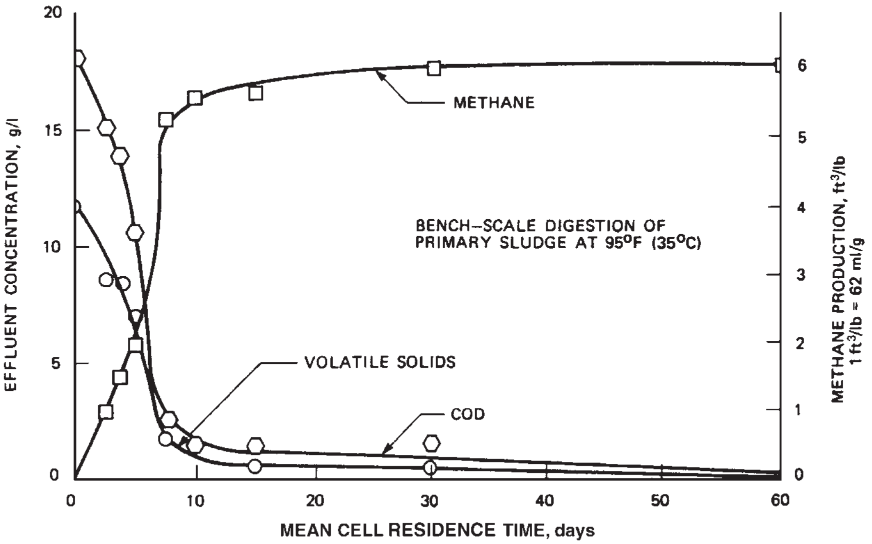


Figure 3.46 Performance of anaerobic digestion systems as a function of mean cell residence time (MCRT) [38].

methane production increase with longer MCRTs. As the MCRT is reduced, more types of bacteria are washed out of the digester, and system performance becomes inhibited. The MCRT can be lowered to a critical MCRT beyond which the digester will fail. A generally accepted minimum MCRT of 10 days has been recommended for digesters operated at mesophilic temperatures (95°F, 35°C) [38].

Although the 10-day minimum MCRT is a reasonable operational value that is based on the replication times of the slower-growing methanogenic bacteria, under field conditions, this minimum MCRT must be maintained during all expected situations, including (1) peak hydraulic loading, (2) maximum grit and scum accumulations in the digester, and (3) liquid-level variability due to differences in the rates of sludge feed and withdrawal from the digester. Recommended minimum MCRTs for digester systems operated at various temperatures are given in Table 3.11. It should be noted that since microbial cells grow and reproduce faster at higher temperatures, the minimum required MCRT will decrease as the temperature of digestion increases (see Table 3.11). Example 3.14 illustrates the use of MCRT in anaerobic digester tank design.

Example 3.14 The Northern Nakuru Sewer Improvement District desires to construct an anaerobic digestion system at the local wastewater treatment plant to reduce sludge volume and to recover energy. If the estimated sludge flow rate through the digester is anticipated to be 18,000 gal/day, what is the minimum digester volume needed if the system is operated without recycle?

TABLE 3.11 Suggested Mean Cell Residence Times for Anaerobic Digester Design*

Digestion temperature		
°C	°F	MCRT (days)
18	64	28
24	75	20
30	86	14
35	95	10
40	104	10

*Adapted from ref. [38].

Assume that the digester will be operated at mesophilic temperatures (32–38°C) with a mean cell residence time of 15 days.

solution For a digester operating without recycle, the mean cell residence time is equivalent to the hydraulic retention time θ. The definition of the hydraulic retention time can be employed to estimate reactor volume:

$$\text{Hydraulic retention time } \theta \text{ (days)} = \frac{\text{volume of digester (gal)}}{\text{volumetric flow rate (gal/day)}}$$

or

$$\text{Volume of digester (gal)} = \theta \text{ (days)} \cdot \text{volumetric flow rate (gal/day)}$$

$$= 15 \text{ days} \cdot 18{,}000 \text{ gal/day} = 270{,}000 \text{ gal}$$

Although the concept of MCRT is based on microbial growth fundamentals, not all digesters use this principle for estimating tank size. Empirical methods based on (1) volumetric loading factors, (2) observed sludge volume reduction, and (3) population loading factors are still in use today [38,77]. Each of these methods of sizing anaerobic digestion tanks will be reviewed briefly.

3.2.2.12 Sizing anaerobic digester tanks based on loading factors. Another common approach to sizing anaerobic digester tanks is to estimate the volume required based on a loading factor. The most common loading factor is pounds (or kilograms) of volatile solids (VS) added per day per cubic foot (or cubic meter). The loading factor is chosen based on the digestion temperature and anticipated detention time. Typical design loading factors for high-rate (mesophilic) digestion and standard-rate digestion are given in Table 3.12. Example 3.15 illustrates the use of loading factors in sizing anaerobic digester tanks.

Example 3.15 Instead of designing the anaerobic digester based on hydraulic retention time, the Northern Nakuru Sewer Improvement District (see

TABLE 3.12 Loading Factors for Sizing Anaerobic Digesters*

	Standard-rate digestion	High-rate digestion
Solids loading rate, lb VSS/ft^3 · day	0.04–0.10	0.15–0.40
kg/m^3 · day	0.63–1.60	2.40–6.40
Mean cell residence time (days)	30–60	10–20

*Adapted from ref. [38].

Example 3.14) desires to base the digester volume on solids loading. Assuming that the digester receives 18,000 gal/day of sludge having an average volatile suspended solids content of 3.5 percent, estimate the minimum digester volume assuming a maximum design solids loading rate of 0.20 lb of volatile suspended solids per cubic foot of digester per day (i.e., 0.2 lb VSS/ft^3 · day).

solution

Step 1. Estimate the pounds of volatile suspended solids loaded to the digester per day. (NOTE: 3.5 percent volatile suspended solids content is equivalent to 35,000 mg/liter VSS.)

Volatile suspended solids loading rate (lb VSS/day)

$$= 35{,}000 \text{ mg VSS/liter} \cdot \frac{18{,}000 \text{ gal}}{1 \cdot 10^6 \text{ gal/MG}} \cdot \frac{8.34 \text{ lb}}{\text{MG} \cdot (\text{mg/liter})}$$

$$= 5254.2 \text{ lb VSS/day}$$

Step 2. Estimate the volume of the digester based on a maximum design solids loading rate of 0.2 lb VSS/ft^3 · day:

$$\text{Digester volume (ft}^3) = \frac{\text{daily VSS loading (lb VSS/day)}}{\text{design VSS loading rate (lb VSS/ft}^3 \cdot \text{day)}}$$

$$= \frac{5254.2 \text{ lb VSS/day}}{0.2 \text{ lb VSS/ft}^3 \cdot \text{day}} = 26{,}271 \text{ ft}^3 \text{ (196,507 gal)}$$

3.2.2.13 Sizing anaerobic digester tanks based on observed volume reduction. Another empirical approach for sizing anaerobic digestion tanks is based on the observed reduction in sludge volume as treatment proceeds. The relationship between digester volume and the reduction in sludge volume is described by Eq. (3.31). It should be noted that Eq. (3.31) assumes that when digester mixing is terminated and the supernatant is removed, the sludge volume decreases in a consistent and predictable manner [38]. Example 3.16 illustrates the sizing of a digester tank based on the empirical volume-reduction approach.

$$V = \left[Q_f - \tfrac{2}{3}\,(Q_f - Q_d)\right] t \qquad (3.31)$$

where V = volume of digester tank, ft^3 (m^3)
Q_f = influent flow rate sludge, ft^3/day (m^3/day)
Q_d = effluent flow rate of sludge, ft^3/day (m^3/day)
t = digestion time, days

Example 3.16 An engineering consultant to the Northern Nakuru Sewer Improvement District (see Example 3.14) has recommended that the minimum anaerobic digester volume be based on the solids volume reduction typically observed in mesophilic digestion systems. Assuming that the digester receives 18,000 gal/day (2,406.42 ft^3/day) of thickened sludge and that there is a 40 percent reduction in sludge volume during mesophilic digestion, estimate the volume of the digester if the mean cell residence time is maintained at 15 days.

solution Using Eq. (3.31), estimate the volume of the anaerobic digester based on a 10 percent sludge volume reduction over a 15-day digestion period:

$$V = \left[Q_f - \tfrac{2}{3} \cdot (Q_f - Q_d)\right] t$$

$$= \left[2406.42\ \text{ft}^3/\text{day} - \tfrac{2}{3}\,(2406.42\ \text{ft}^3/\text{day} - 0.40 \cdot 2406.42\ \text{ft}^3/\text{day})\right] 15\ \text{days}$$

$$= 21{,}658\ \text{ft}^3\ (162{,}000\ \text{gal})$$

3.2.2.14 Sizing anaerobic digester tanks based on population. Anaerobic digestion tanks also may be sized on the basis of allocating a certain number of cubic feet per capita. Typical design criteria for anaerobic digesters are given in Table 3.13. Example 3.17 illustrates the approach for sizing anaerobic digesters based on population.

TABLE 3.13 Typical Design Criteria for Sizing Anaerobic Digesters*

Sludge type	Standard-rate digestion (ft^3/capita)	High-rate digestion (ft^3/capita)
Primary sludge	2–3	1.3–2.0
Primary sludge plus trickling-filter sludge	4–5	2.7–3.3
Primary sludge plus waste-activate sludge	4–6	2.6–4.0

*Adapted from ref. [38].

Example 3.17 To ensure that the anaerobic digestion system employed by the Northern Nakuru Sewer Improvement District (see Example 3.14) properly accounts for population growth, the plant superintendent desires to estimate the minimum digester size necessary to treat the waste. Assuming that the local sewer system will collect and transfer a wastewater loading equivalent to 12,500 persons to the treatment plant during peak operating conditions, estimate the size of the anaerobic digester if the per capita (i.e., person-equivalent) digestion design capacity for mesophilic digestion is 3.0 ft^3.

solution To estimate anaerobic digester size, multiply the person-equivalents digestion design criteria by the person-equivalents estimated from the wastewater volume and strength:

$$\begin{aligned}\text{Digester volume } (\mathrm{ft}^3) \\ &= \text{person-equivalents} \cdot \text{digester design criteria } (\mathrm{ft}^3/\text{person-equivalent}) \\ &= 12{,}500 \text{ person-equivalents} \cdot 3.0\ \mathrm{ft}^3/\text{person-equivalent} \\ &= 37{,}500\ \mathrm{ft}^3\ (280{,}500\ \mathrm{gal})\end{aligned}$$

3.2.2.15 Anaerobic digester roof and tank configuration. Anaerobic sludge digestion tanks are covered to contain odors, maintain operating temperature, serve as a barrier to oxygen transfer, and collect digester biogas. Digester covers can be classified as either fixed or floating (Fig. 3.47).

Fixed digestion covers are fabricated from steel, reinforced concrete, or corrosion-proof fiberglass-reinforced polyester (FRP). In most cases, fixed covers are domed-shaped, although conical and flat covers have been built. Digesters with fixed covers are operated so as to maintain

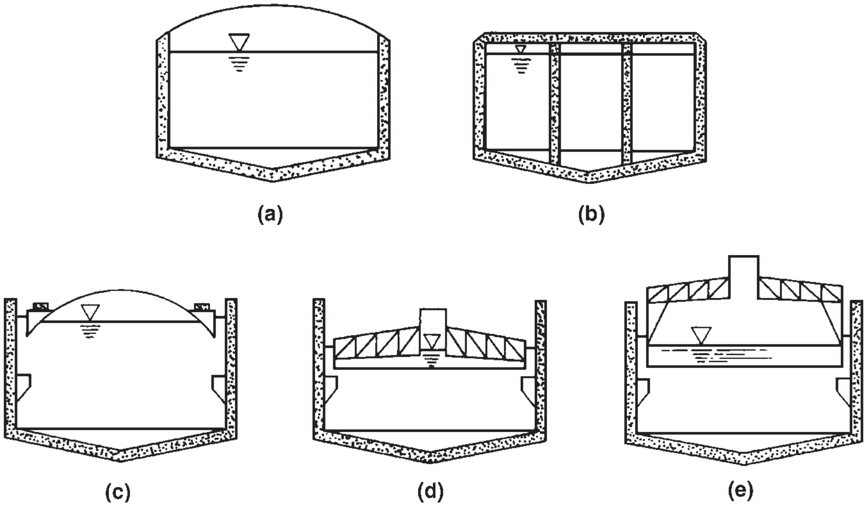


Figure 3.47 Schematic diagrams of fixed digester covers: (*a*) domed roof; (*b*) flat roof. Floating digester covers: (*c*) Wiggens type; (*d*) Downes type; (*e*) floating gas cover.

a constant liquid surface level in the tank. Rapid withdrawals of digested sludge (i.e., without a corresponding addition of raw sludge) can draw air into the tank, producing an explosive mixture of biogas and oxygen. In addition, there have been reported cases in which the liquid level under the fixed cover has been allowed to increase sufficiently to damage the cover structurally [36]. This type of problem usually involves a tightly clogged overflow system and/or a feed valve that has been inadvertently left open.

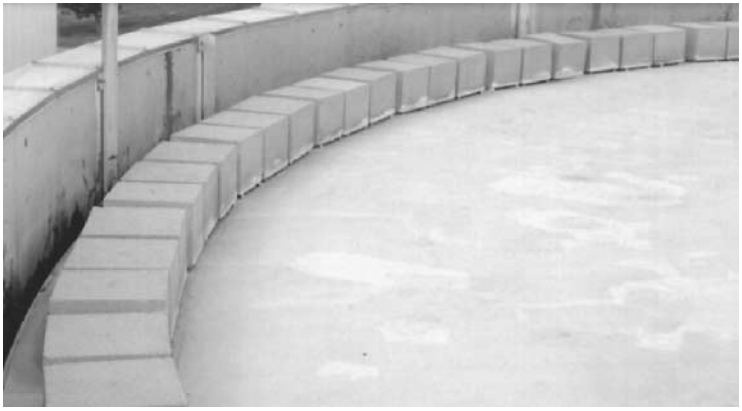
Floating covers are more expensive than fixed covers, but they have the advantage of permitting independent additions and withdrawals of sludge; they also reduce gas hazards and can be designed to control formation of scum [36]. Traditionally, floating covers have followed one of two designs: (1) the pontoon or Wiggins-type and (2) the Downes-type [38]. Both types of covers float directly on the liquid and typically have a maximum vertical travel of 6 to 8 ft (2–3 m). These floating cover designs differ primarily in the method used to maintain buoyancy, which, in turn, determines the degree of cover submergence [36].

In the Wiggins-type cover design, the bottom of the cover slopes steeply along the outer edge. This outer portion of the cover forms an annular pontoon or float that results in a large liquid displacement for a small degree of cover plate submergence. Therefore, Wiggins covers have only a portion of the annular submerged, with the largest portion of the cover remaining exposed to the biogas above the liquid surface.

In the Downes-type cover design, the bottom of the cover slopes gradually throughout the entire radius, resulting in only a small liquid displacement for each degree of ceiling plate submergence. Typically, the lower third of the radius of the Downes cover is in contact with the liquid. However, it is normally desirable to increase the degree of cover submergence by adding ballast. The proximity of the cover to the liquid keeps floating matter submerged and subjected to mixing action, reduces the area exposed to the corrosive sludge biogas, and adds to cover stability. Ballast can be added as concrete blocks or as a layer of concrete spread across the upper surface of the cover (Fig. 3.48).

A variation of the floating cover is the floating gas-holder, which may be fabricated from steel or polymer membrane (Fig. 3.49). A gas-holder is a floating cover with an extended skirt [up to 10 ft (3 m) high] to allow storage of biogas during periods when gas production exceeds demand. Floating gas covers provide an effective means of adjusting for the difference between the rate of gas production and the rate of gas consumption by storing the gas produced at peak periods for use during times of low production. When used to provide fuel for digester heating, gas covers generally are designed to store from 6 to 10 hours of average gas production. Although the storage volume in gas-holders is significantly larger than other floating covers, storage pressure in a gas-holder is low (maximum pressure is approximately 15 inH_2O, 3.7 kN/m^2).

(a)



(b)

Figure 3.48 (*a*) Concrete ballast used to support a Downes-type floating digester cover. (*b*) Close-up of preformed concrete ballast.

Greater gas storage is achieved by compressing the biogas for high-pressure storage in cylinders or by providing a separate low-pressure displacement storage tank [36,77]. Figure 3.50 depicts the floating cover guide mechanism that is typically employed.

Gas-holding covers are less stable than conventional floating covers because they are supported entirely by a cushion of compressible gas rather than by an incompressible liquid. Moreover, gas-holders also expose a large side area to lateral wind loads [38]. To prevent tipping or binding, ballast at the bottom of the extended skirt and spiral guides must be provided [38,77]. Typical appurtenances for all types of digester covers include (1) sampling ports, (2) manholes, (3) a ventila-

(a)

(b)

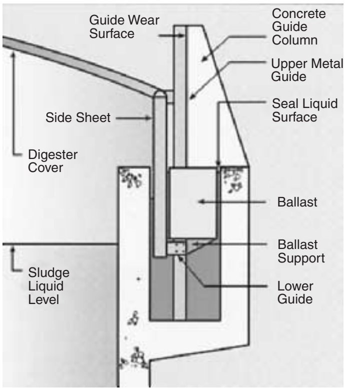
Figure 3.49 (*a*) Diagram of floating biogas holder system. (*Courtesy of Dorr Oliver.*) (*b*) Photograph of membrane-type biogas holder system. (*Courtesy of US Filter/Envirex.*)

tion system, (4) a liquid overflow system, and (5) a vacuum-pressure relief system equipped with a flame trap [38,77].

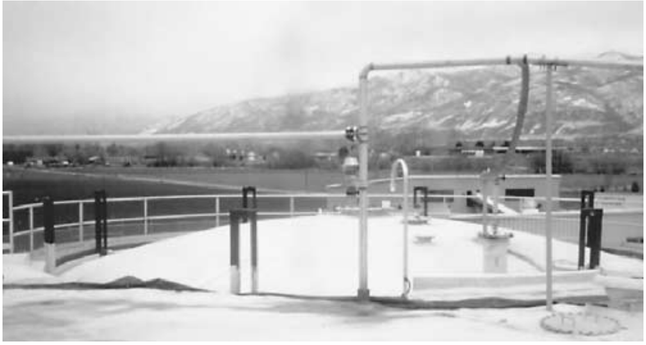
In addition to cover design, the types of cross-sectional tank areas used in the design of anaerobic digesters varies in practice. Anaerobic digestion tanks may be rectangular, egg-shaped, or cylindrical. Figure 3.51 illustrates typical rectangular and cylindrical digestion tank designs. Although simple to construct, rectangular tanks are not used very often because of their poor mixing characteristics.

Egg-shaped digestion tanks have improved mixing characteristics compared with rectangular tanks. Moreover, the egg-shaped design essentially eliminates the need for digester cleaning. Although popular in Europe, only a limited number of these units are found in the United States (Fig. 3.52).

The most common digestion tank design is a low-depth, vertical cylinder. Cylindrical anaerobic digester tanks are normally 20 ft (6 m)



(a)



(b)

Figure 3.50 (*a*) Schematic diagram of a gas-holder cover system with guide system. (*Courtesy of EIMCO Process Equipment Company.*) (*b*) Photograph of a floating gas cover system. Note the metal guide system.

to 125 ft (38 m) in diameter and have a water depth of at least 25 ft (7.5 m) at the side wall and may be as deep as 45 ft (14 m) in the center. The floor of the digester is usually conical, with the bottom sloping to the sludge draw-off in the center.

For cylindrical tanks, alternative bottom designs using a “waffle” shape have been employed to minimize grit accumulation and reduce the need for digester cleaning (Fig. 3.53).

3.2.2.16 Piping in anaerobic digestion systems. The piping system for an anaerobic digester is a critical component of the overall design. Since many activities occur during the operation of a digester, including (1) feeding of raw sludge, (2) circulation of sludge through heat exchangers, (3) withdrawal of digested sludge and supernatant, and (4)

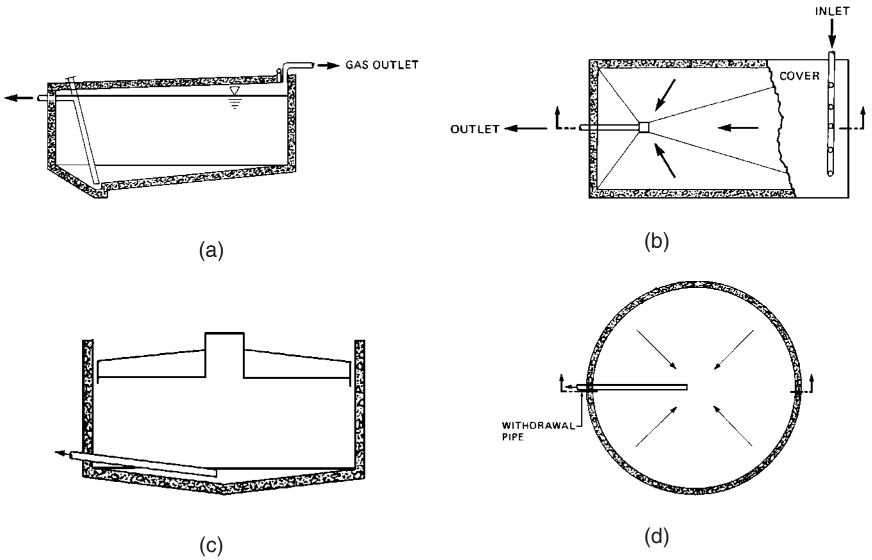


Figure 3.51 Schematic diagrams of rectangular and cylindrical-shaped digestion tanks: (*a*) rectangular tank, side view; (*b*) rectangular tank, plan view; (*c*) cylindrical tank, side view; (*d*) cylindrical tank, plan view [36].

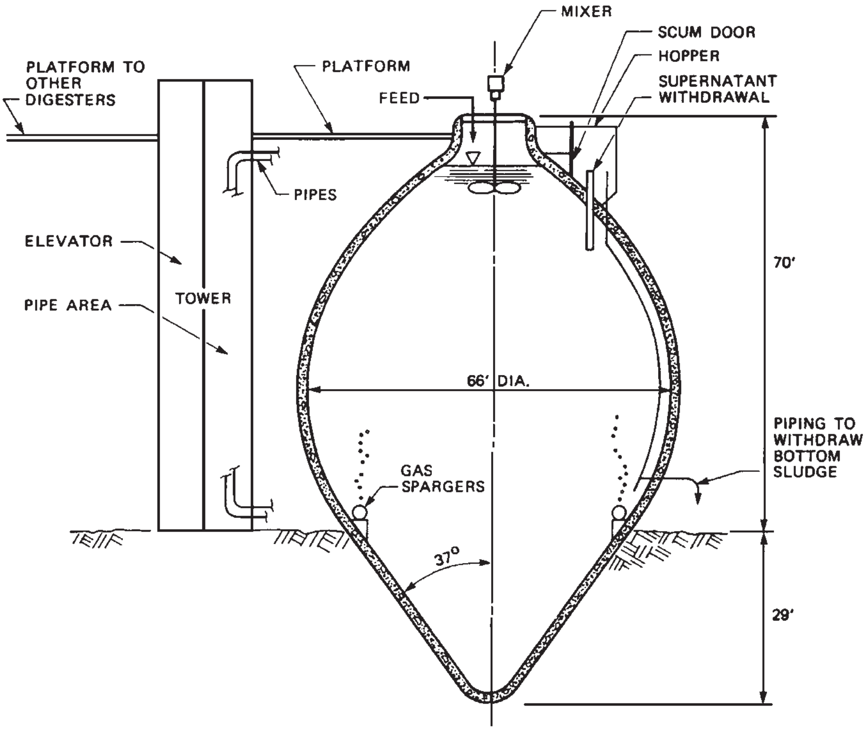


Figure 3.52 Schematic diagram of an egg-shaped digestion tank.

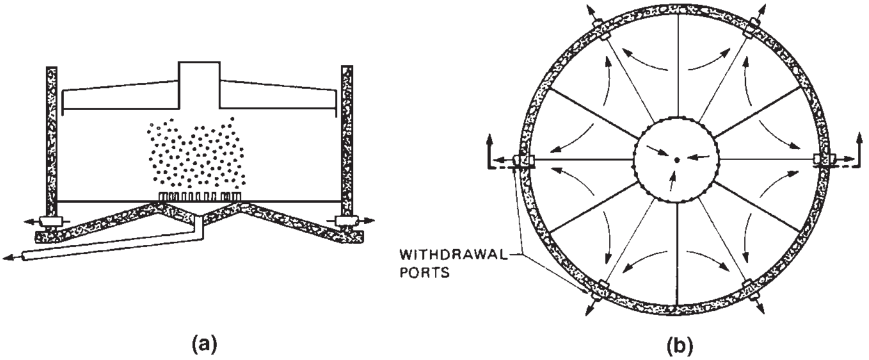


Figure 3.53 Schematic diagrams of a waffle-bottomed digestion tank: (*a*) section view; (*b*) plan view.

collection of biogas, flexibility should be built into the system to allow operation in a variety of modes and to ensure that digestion can continue in the event of equipment breakdown or pipe clogging [38].

Raw sludge should enter the digester in a zone of intense mixing to rapidly disperse the undigested organics. Before entering the digester, raw sludge should be mixed with warm circulating sludge to seed the incoming sludge with active anaerobic microorganisms and to avoid thermal shock. The introduction of cold feed sludge into regions where there is no local mixing results in the feed sludge sinking to the digester bottom and becoming an isolated mass [38,77]. Feeding of incoming sludge into anaerobic digesters can be automated to load the tanks frequently and uniformly. Switching the raw sludge feed between several tanks can be controlled based on time, hydraulic flow, or solids flow [38].

A time-controlled feed system uses a repeat cycle timer to sequentially open and close the feed valve for each digester. A flow-controlled feed system uses a flowmeter on the raw sludge pipeline (typically in combination with a totalizer) to load preset volumes to each digester. A feed-control system based on solids flow requires the measurement of both raw sludge flow and density. Since density is a function of the concentration of sludge solids, these two signals can be combined to yield a measure of the mass of solids fed to the digesters.

Digested sludge is usually drawn off the bottom of the tank, although means to withdraw sludge from at least one other point should be provided in case the main line becomes plugged. A supernatant collection system, when required, should have draw-off points at three or more elevations to allow the operator to remove the clearest supernatant [36]. An illustration of a supernatant collection system is provided in Fig. 3.54. The telescopic valve in the supernatant

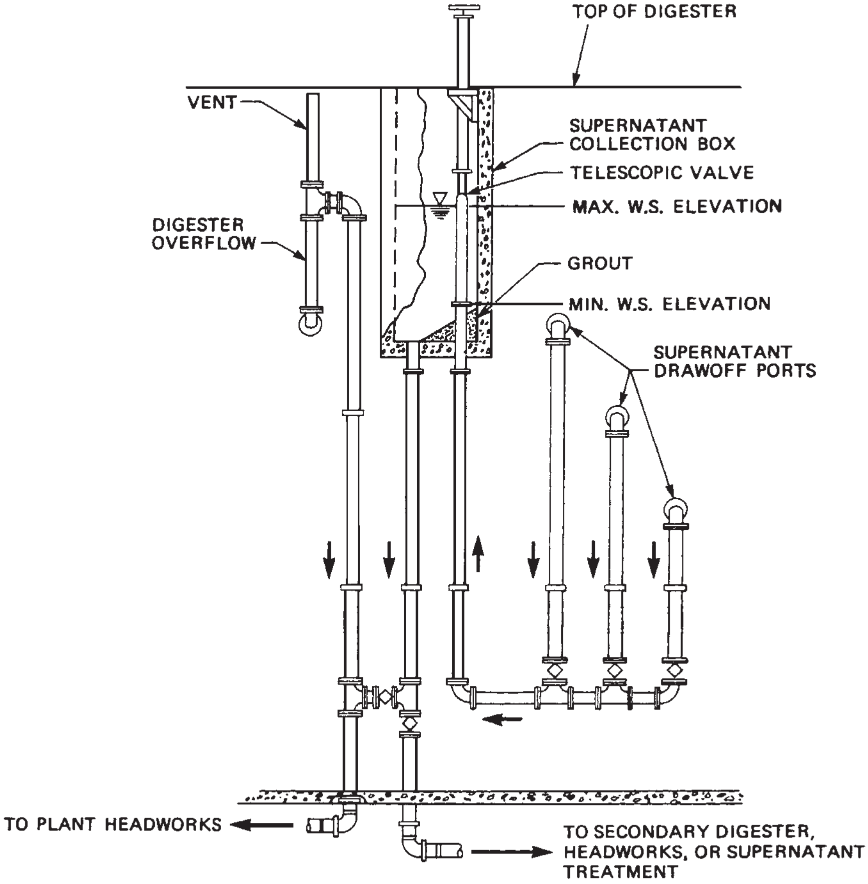


Figure 3.54 Schematic diagram of a digester supernatant withdrawal system [38].

draw-off system is used to adjust the fluid surface level in the digester. An unvalved overflow with a vent as a siphon breaker is provided to ensure that the tank cannot be overfilled.

To minimize the potential for pipe clogging, sludge piping generally has a minimum diameter of 6 in (150 mm), and fluid velocities should be maintained above 4 ft/s (1.2 m/s) to keep sludge solids in suspension [38]. Sludge piping should be kept as short as practicable with a minimum number of bends. To facilitate changes in flow direction, the piping network should employ long-radius elbows and sweep tees. Provisions are commonly made for cleaning sludge pipelines with steam, high-pressure water, or mechanical devices [38].

A problem unique to anaerobic digestion systems is the accumulation of magnesium ammonium phosphate ($MgNH_4PO_4 \cdot 6H_2O$, commonly know as *struvite*) on the interior walls of the digester tank and downstream piping. This scale will increase pipeline friction, displace volume in the digestion tank, and foul downstream mechanical equipment [38]. Methods used successfully to prevent the accumulation of this material include (1) substitution of polyvinyl chloride (PVC) pipe for cast iron pipe to reduce roughness, (2) limiting magnesium ion concentration in the influent wastewater stream, and (3) dilution of digested sludge flows to prevent supersaturation and to raise pipeline velocities.

3.2.2.17 Anaerobic digester cleaning. Anaerobic digestion tanks can become partially filled with a bottom layer of settled grit and a top layer of floating scum. These accumulations reduce the volume available for active digestion and thereby degrade the performance of the digesters. Periodically, the digestion tank must be drained and these deposits removed.

Not only is the cleaning process expensive and unpleasant, it can disrupt normal processing of sludge for as long as several months. Therefore, attention should be given during anaerobic digester design to (1) reduce the rate at which grit and scum accumulate and (2) facilitate the cleaning of the digester when it becomes necessary.

3.2.3 Aerobic digestion

Aerobic digestion is the oxidative microbial stabilization of sludge. Aerobic digestion is based on the principle that when there is an inadequate external substrate available, microorganisms will metabolize their own cellular mass, resulting in an overall loss of volatile solids. In actual operation, aerobic digestion involves the oxidation of any biodegradable matter and microbial cellular material by aerobic microorganisms, as described by Eqs. (3.32) and (3.33):

$$\text{Available organic matter} + O_2 \xrightarrow[\text{Aerobic microorganisms}]{} CO_2 + H_2O + \text{heat} \qquad (3.32)$$

$$\text{Cellular material} + O_2 \xrightarrow[\text{Aerobic microorganisms}]{} CO_2 + H_2O + \text{heat} \qquad (3.33)$$

Equation (3.33) normally is referred to as *endogenous respiration* and sometimes is written as Eq. (3.34). It should be noted that as a result of microbial degradation of both available organic matter and cellular material, the overall mass of sludge from this stabilization process is reduced. Although there is a net loss of sludge solids, a finite amount of stabilized cell mass is generated as well.

$$\underset{\text{Microbial cell (m.w. 113)}}{C_5H_7O_2N} + \underset{7\,(32)}{7O_2} \rightarrow \underset{5\,(44)}{5CO_2} + \underset{3\,(18)}{3H_2O} + H^+ + \underset{(62)}{NO_3^-} \qquad (3.34)$$

where $C_5H_7O_2N$ = typical composition of the organic fraction of microbial cells.

3.2.3.1 Aerobic digestion process design. Aerobic digestion systems can be designed to operate as semibatch or continuous operating systems (Fig. 3.55). In the semibatch mode, sludge is pumped directly from thickening operations to the aerobic digester. The time required to fill the digester tank will be a function of the thickened sludge flow rate, available tank volume, precipitation, and evaporation [9]. During the filling operations, the sludge is aerated continually. When the tank is full, aeration is operated continuously for 2 to 3 weeks to ensure that the sludge is thoroughly stabilized. Aeration is then discontinued, and the stabilized sludge is permitted to settle within the tank. Clarified liquor (supernatant) can then be

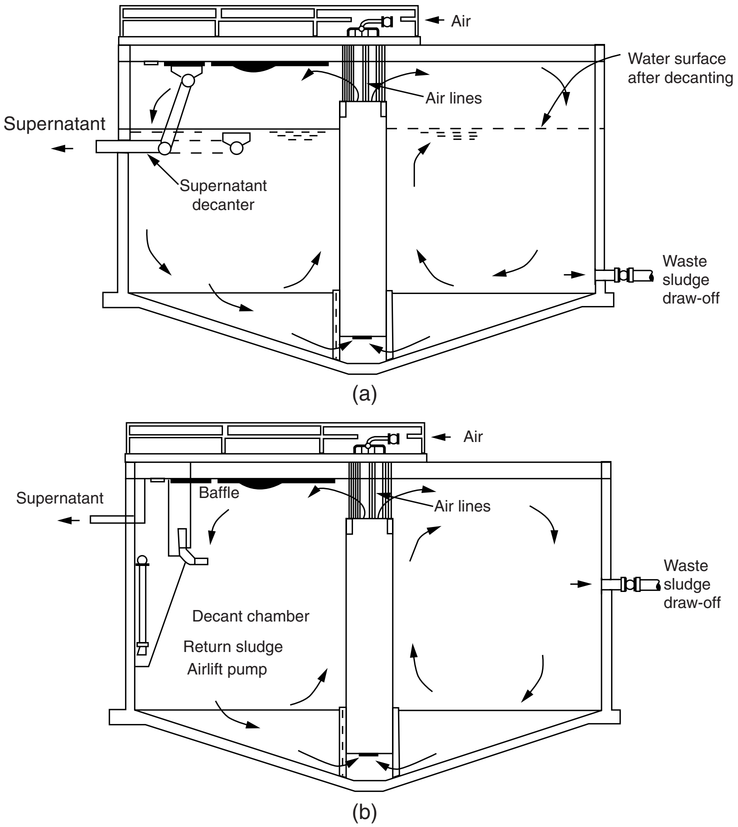


Figure 3.55 Schematic diagrams of aerobic digester systems: (*a*) batch operation; (*b*) continuous operation.

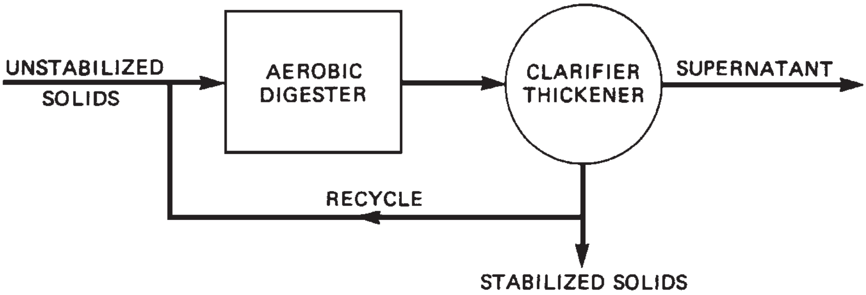


Figure 3.56 Schematic diagram of an aerobic digester system.

decanted, and the concentrated sludge is removed. The solids content of thickened sludge from this type of operation normally ranges from 2 to 4 percent.

When a sufficient amount of stabilized sludge and/or supernatant has been removed, the cycle is repeated. Between cycles, it is customary to leave some stabilized sludge in the aeration tank to provide a microbial seed of organisms that are acclimated to degrading organic matter. It should be noted that if the aerobic digester is operated so that the incoming sludge is used to displace supernatant and the sludge is allowed to accumulate, the solids retention time (i.e., MCRT) is not equal to the hydraulic residence time.

The continuous aerobic digestion process closely resembles the activated-sludge process (Fig. 3.56). In this operation, sludge is pumped directly from the thickening operation to the aerobic digester.

The aeration tank operates at a fixed level, with the overflow normally going to a solids-liquid separation unit (e.g., secondary thickener), where stabilized sludge is separated from a clarified effluent. A portion of the thickened and stabilized sludge from the solids-liquid separator is recycled back to the digestion tank, while the remainder is removed for further processing.

In both aerobic digester operational modes, mixing is required to keep sludge solids in suspension and to maintain aerobic conditions. Air may be introduced into the system through a set of subsurface diffusers or is entrained using a turbine-type mixer.

3.2.3.2 Aerobic digestion temperature. Since the majority of aerobic digesters are open tanks, digester temperatures depend on weather conditions. Minimization of heat losses always should be considered in aerobic digester system design. Some typical design options that will reduce heat loss include (1) use of concrete rather than steel tanks, (2) placing tanks below rather than above ground, and (3) using subsurface rather than surface aeration [38].

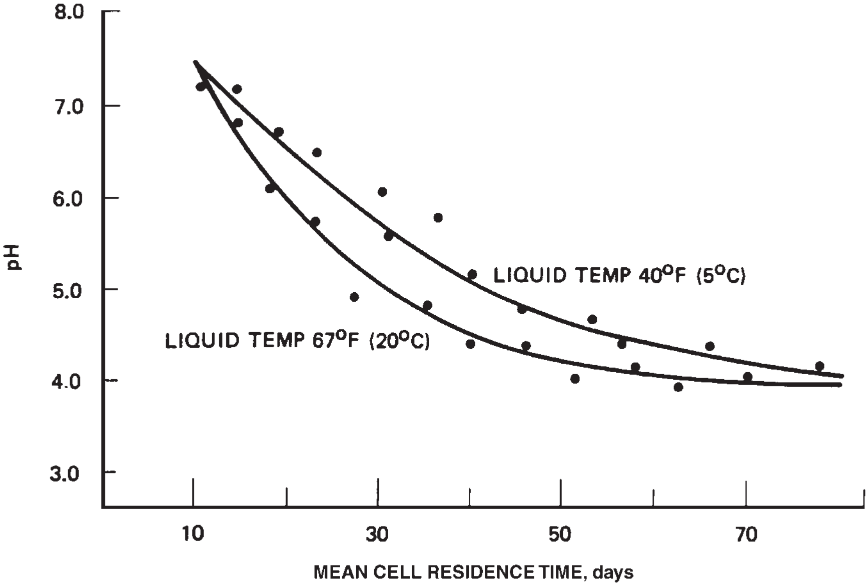


Figure 3.57 Effect of detention time on sludge pH.

3.2.3.3 Aerobic digester pH. The effect of increasing the mean cell residence time on aerobic digester pH is illustrated in Fig. 3.57. The reduction in pH in aerobic digester operations is due primarily to the acid formation that occurs during nitrification (i.e., conversion of ammonia nitrogen to nitrate).

The energy and microbial synthesis reactions that characterize the nitrification process are given by Eqs. (3.35) and (3.36). These equations indicate that the nitrification process results in both acid production (H^+) and the consumption of bicarbonate alkalinity (HCO_3^-), both of which result in the lowering of digester pH.

Energy reaction:

$$NH_4^+ + 2O_2 \xrightarrow[\text{Microorganisms}]{} NO_3^- + \underbrace{2H^+}_{\text{Acid formation}} + H_2O \tag{3.35}$$

Synthesis reaction:

$$NH_4^+ + 4CO_2 + \underbrace{HCO_3^-}_{\text{Bicarbonate alkalinity}} + H_2O \xrightarrow[\text{Microorganisms}]{} \underbrace{C_5H_7O_2N}_{\text{Cell tissue}} + 5O_2 \tag{3.36}$$

Lower mean cell residence times and/or higher operating temperatures (above 45°C, or 113°F) have been found to be effective in reducing nitrate formation [38]. Although limiting nitrification is desirable, it has been found that the reduction in digester pH does not significantly affect the volatile solids removal efficiency [33].

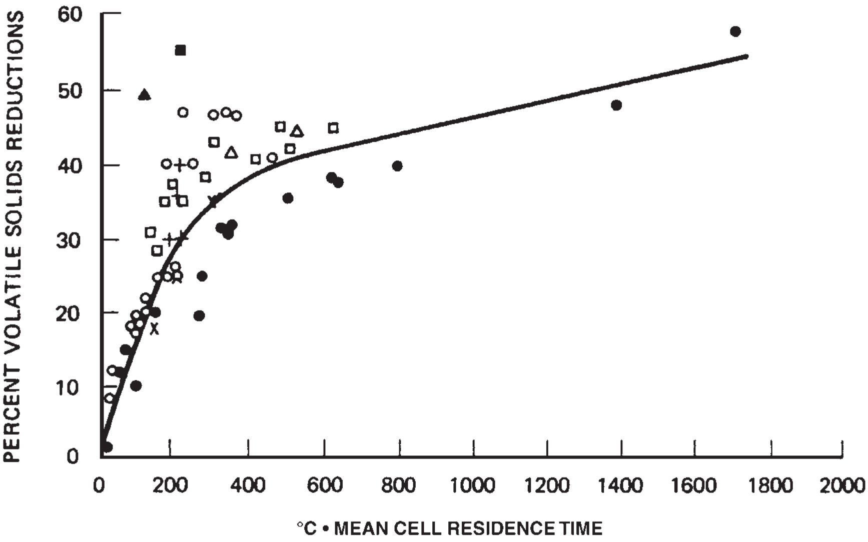


Figure 3.58 Effect of aerobic digester temperature and detention time on volatile solids removal.

3.2.3.4 Aerobic digestion volatile solids removal. Like anaerobic digestion, volatile solids (i.e., organic matter) removal in the aerobic digestion process is a function of digester temperature and the mean cell residence (or solids retention) time. The behavior of the volatile solids removal efficiency as a function of both digester temperature and the mean cell residence time is illustrated in Fig. 3.58.

Under normal conditions, the oxidation of organic matter generates sufficient heat within the system that an external heat supply is unnecessary. The reduction in organic matter content that occurs during the aerobic digestion process can be represented by a first-order reaction [Eq. (3.37)]. The reaction rate constant K_d is a function of sludge type, temperature, and solids concentration. The normal range for this parameter is 0.05 day^{-1} [at 15°C (59°F)] to 0.14 day^{-1} [at 25°C (77°F)] [33,38].

$$\frac{dM}{dt} = r_d = -K_d M \tag{3.37}$$

where M = concentration of organic fraction of sludge, mg VS /liter
t = mean cell retention time, days
r_d = rate of volatile solids (i.e., organic matter) removal, mg VS/liter · day
K_d = reaction rate constant, day^{-1}

TABLE 3.14 Operational Parameters for Aerobic Digestion of Sludge*

Parameter	Value
Mean cell retention time (days)	
Waste-activated sludge (WAS) only	10–15
Primary plus WAS or trickling filter sludge	15–20
Solids loading (lb VS/ft^3 · day)	0.1–0.3
Oxygen requirements (lb O_2/lb solids into digester)	
Cell tissue (endogenous respiration)	2.3
BOD_5 (primary sludge)	1.6–1.9
Energy requirements for mixing	
Mechanical aerators, hp/10^3 ft^3	0.75–1.50
Diffused-air mixing, ft^3 /10^3 ft^3 · min	20–40
Dissolved oxygen residual, mg/liter	1–2
Reduction in volatile solids (VS), %	40–50

*Adapted from ref. [38].

The operation of aerobic digesters varies among wastewater treatment plants. Table 3.14 provides the typical range of operational parameters found in properly operating aerobic digester facilities.

3.2.3.5 Aerobic digester tank size. The volume of an aerobic digester tank may be estimated by knowledge of the required mean cell retention time (i.e., solids retention time) to achieve a desired reduction in volatile solids. Equation (3.38) provides the fundamental relationship between solids retention time and sludge solids concentrations.

$$\text{Solids retention time} = \frac{\text{mass of solids in digester}}{\begin{array}{c}\text{mass of solids removed from digester}\\ \text{(in supernatant and in}\\ \text{thickened underflow)}\end{array}} \tag{3.38}$$

The concentration of suspended solids in the aerobic digester at any time will range from the influent suspended solids concentration to a maximum value of the thickened and stabilized sludge. On average, the suspended solids concentration within the aerobic digester is equal to approximately 70 percent of the thickened effluent suspended solids concentration [33]. The mass of sludge contained in the aerobic digester together with the mass of sludge removed in the stabilized/thickened sludge flow and the decanted supernatant flow may be estimated using Eqs. (3.39), (3.40), and (3.41), respectively:

$$\text{Mass of solids in digester} = \text{volume} \cdot \text{SS}_{\text{digester}} \cdot \frac{8.34 \text{ lb}}{(\text{mg/liter})\text{ MG}} \tag{3.39}$$

Mass of solids removed in stabilized sludge flow (lb/day)

$$= Q_{\text{influent}} \cdot \text{SS}_{\text{stabilized effluent}} \cdot f \cdot \frac{8.34 \text{ lb}}{(\text{mg/liter}) \text{ MG}} \tag{3.40}$$

Mass of solids removed in supernatant (lb/day)

$$= Q_{\text{influent}} \cdot \text{SS}_{\text{supernatant}} \cdot (1 - f) \cdot \frac{8.34 \text{ lb}}{(\text{mg/liter}) \text{ MG}} \tag{3.41}$$

where volume = volume of aerobic digester, in million gallons, MG
Q_{influent} = influent flow rate of thickened sludge, MG/day
$\text{SS}_{\text{stabilized effluent}}$ = suspended solids concentration of thickened and stabilized sludge, mg/liter
$\text{SS}_{\text{supernatant}}$ = suspended solids concentration of supernatant, mg/liter
$\text{SS}_{\text{digester}}$ = average suspended solids concentration of digester, mg/liter
f = fraction of influent flow that is retained in the aerobic digester, decimal
$1 - f$ = fraction of influent flow that leaves from the aerobic digester as supernatant, decimal
8.34 = conversion factor, 8.34 lb/(mg/liter)MG
MG = million gallons

Substituting Eqs. (3.39), (3.40), and (3.41) into Eq. (3.38), provides a general relationship that can be used to estimate tank volume as a function of mean cell residence time [Eq. (3.42)]:

Volume of tank (MG)

$$= \frac{\theta_c Q_{\text{influent}} \left[\text{SS}_{\text{stabilized effluent}} f + \text{SS}_{\text{supernatant}} (1 - f)\right]}{\text{SS}_{\text{digester}}} \tag{3.42}$$

where θ_c = solids retention time (days).

It should be noted that the term f may be estimated using Eq. (3.43):

$$f = \frac{\text{influent SS concentration (mg/liter)}}{\text{SS}_{\text{stabilized effluent}} \text{ (mg/liter)}} \cdot \text{fraction of solids not destroyed} \tag{3.43}$$

Example 3.18 illustrates the use of Eqs. (3.38) through (3.43) in designing an aerobic digestion stabilization process.

Example 3.18 Cushite County Wastewater Treatment Plant is currently generating 2500 lb/day of primary sludge (dry-mass basis) and 1200 lb/day of secondary sludge (dry-mass basis). The sludge is combined in a gravity thickener where it is concentrated to a single sludge flow of 3 percent solids with a specific gravity of 1.02. If the thickened sludge is to be stabilized in a semicontinuous aerobic digester, estimate the following:

1. Volume flow rate of thickened sludge to the digester
2. Solids retention time
3. Mass of volatile solids removed per day
4. Oxygen and air requirement for stabilization
5. Volume of digester
6. Air requirement per 1000 ft^3 of digester volume

Assume that the following data apply:

1. Digester design will be based on a minimum winter temperature of 10°C (50°F).
2. Digester must achieve at least 40 percent volatile solid removal at 10°C (50°F).
3. Suspended solids in digester are 70 percent of thickened sludge concentration.
4. Volatile fraction of the influent sludge solids is 0.9.
5. Stabilized and thickened sludge will have a solids content of 5 percent.
6. Supernatant will have a solids concentration of 300 mg/liter.
7. Density of air is 0.075 lb/ft^3.
8. Air contains 23.2 percent oxygen by weight.
9. Diffused-air oxygen transfer efficiency is 12 percent.

Step 1. Compute the volume flow rate of sludge from the thickener:

$$Q_{\text{influent}} = \frac{\text{mass of solids/day}}{\rho_w S_s P_s}$$

$$= \frac{2500 \text{ lb/day} + 1200 \text{ lb/day}}{(62.4 \text{ lb/ft}^3)(1.02)(0.03)}$$

$$= 1938 \text{ ft}^3\text{/day} \quad \text{(or 14,496 gal/day, 0.014496 MG/day)}$$

Step 2. Compute the solids retention time required to achieve 40 percent volatile solids reduction at a minimum temperature of 10°C. From Fig. 3.50, 450 degree-days are required to achieve 40 percent volatile solids removal. The solids retention time is then computed as follows:

$$\text{Solids retention time (days)} = \frac{450°\text{C} \cdot \text{days}}{10°\text{C}} = 45 \text{ days}$$

Step 3. Compute the volatile solids removed per day:

Mass of volatile solids added to system (lb/day)

$$= 0.9\ (2500\ \text{lb/day} + 1200\ \text{lb/day})$$

$$= 3330\ \text{lb/day}$$

$$\text{Mass of volatile solids removed (lb/day)} = 0.4\ (3330\ \text{lb/day})$$

$$= 1332\ \text{lb/day}$$

Step 4. Determine the oxygen requirements based on guidelines in Table 3.14.

$$\text{lb } O_2\text{/day} = \frac{2.3\ \text{lb } O_2}{\text{lb VS removed}}\ 1332\ \text{lb VS/day}$$

$$= 3064\ \text{lb } O_2\text{/day}$$

Step 5. Estimate the volume of air required assuming the oxygen transfer efficiency is 12 percent:

$$\text{Air requirement (ft}^3\text{/day)} = \frac{3064\ \text{lb } O_2\text{/day}}{(0.12)\ (0.075\ \text{lb air/ft}^3\text{air})\ (0.232\ \text{lb } O_2\text{/lb air})}$$

$$= 1{,}467{,}241\ \text{ft}^3\text{/day (or 1019 ft}^3\text{/min)}$$

Step 6. Determine the fraction of influent flow that is retained within the digester *f* using Eq. (3.43):

$$f = \frac{\text{influent suspended solids concentration (mg/liter)}}{\text{suspended solids concentration of thickened/stabilized biosolids (mg/liter)}} \cdot \text{fraction of solids not destroyed}$$

The fraction of solids not destroyed is estimated as follows:

Fraction of solids not destroyed

$$= \frac{\text{total solids in influent} - \text{volatile solids removed}}{\text{total solids in influent}}$$

$$= \frac{(2500\ \text{lb/day} + 1200\ \text{lb/day}) - (1332\ \text{lb/day})}{2500\ \text{lb/day} + 1200\ \text{lb/day}}$$

$$= 0.64$$

$$f = \frac{30{,}000\ \text{mg/liter}}{50{,}000\ \text{mg/liter}}\ 0.64$$

$$= 0.38$$

Step 7. Estimate the digester tank volume using Eq. (3.42). Note that the digester suspended solids content is assumed to be 70 percent of the stabilized effluent solids content.

$$\text{Volume of tank (MG)} = \frac{\theta_c Q_{\text{influent}}\,[\text{SS}_{\text{stabilized effluent}} f + \text{SS}_{\text{supernatant}}\,(1 - f)]}{\text{SS}_{\text{digester}}}$$

$$= \{45 \text{ days} \cdot 0.014996 \text{ MG/day } [50{,}000 \text{ mg/liter} \cdot 0.38 + 300 \text{ mg/liter } (1 - 0.38)\,]\}/[(0.7) \cdot 50{,}000 \text{ mg/liter}]$$

$$= 0.37 \text{ MG } (370{,}000 \text{ gal or } 49{,}500 \text{ ft}^3)$$

Step 8. Estimate the cubic feet of air per 1000 ft^3 of digester volume to verify adequate mixing:

$$\text{Air requirement} = \frac{1019 \text{ ft}^3\text{/min of air}}{49.5 \text{ ft}^3/10^3 \text{ ft}^3} = \frac{20.6 \text{ ft}^3 \text{ of air}}{10^3 \text{ ft}^3 \cdot \text{min}}$$

Since the air requirement is within the range specified in Table 3.14, we can assume that the system will be mixed sufficiently.

Another approach for estimating aerobic digester tank volume involves the use of Eq. (3.44). Equation (3.44) has been found to be effective in estimating the digester tank size when nitrification is not significant [38]. Example 3.19 illustrates the use of Eq. (3.44) in estimating the required aerobic digester tank volume.

$$\text{Digester tank volume (ft}^3) = \frac{Q_i\,(X_i + yS_i)}{X\,(K_d P_v + 1/\theta_c)} \tag{3.44}$$

where Q_i = influent sludge flow rate to digester, ft^3/day
X_i = influent suspended solids concentration, mg/liter
y = fraction of influent BOD_5 consisting of raw primary sludge, decimal
S_i = influent BOD_5, mg/liter
X = digester suspended solids concentration, mg/liter
K_d = reaction rate constant, day^{-1}
P_v = volatile fraction of digester suspended solids concentration, decimal
θ_c = solids retention time, days

Example 3.19 The Garissa County Wastewater Treatment Plant has decided to employ mesophilic aerobic digestion to reduce its sludge volume. Given that nitrification is sufficiently inhibited at mesophilic temperatures (35°C), estimate the minimum digester volume if the average thickened sludge flow rate is 22,000 gal/day (2941.2 ft^3/day). Assume the following values for other system parameters:

1. Q_i (influent sludge flow rate to digester): 2941.2 ft^3/day
2. X_i (influent suspended solids concentration): 40,000 mg/liter (4%)
3. y (fraction of influent BOD_5 consisting of raw primary sludge): 0.60 (60%)
4. S_i (influent BOD_5): 55,000 mg/liter
5. X (digester suspended solids concentration): 27,000 mg/liter
6. K_d (reaction rate constant): 0.26 day^{-1}
7. P_v (volatile fraction of digester suspended solids concentration): 0.8 (80%)
8. θ_c (mean cell residence time): 10 days

solution To estimate anaerobic digester size, substitute the appropriate values into Eq. (3.44):

Digester tank volume (ft^3)

$$= \frac{Q_i\,(X_i + yS_i)}{X\,[K_d P_v + (1/\theta_c)\,]}$$

$$= \frac{(2941.2\ \text{ft}^3/\text{day})\,(40{,}000\ \text{mg/liter} + 0.60 \cdot 55{,}000\ \text{mg/liter})}{27{,}000\ \text{mg/liter})\,[0.26\ \text{day}^{-1} \cdot 0.8 + (1/10\ \text{days})\,]}$$

$$= 25{,}818.4\ \text{ft}^3\ (193{,}121.7\ \text{gal})$$

Many advantages have been cited for aerobic digestion over anaerobic digestion in the stabilization of sludge [9,33]. The principal operational advantages of aerobic digestion include

- Lower capital costs
- No nuisance odors generated
- Operation is less susceptible to upsets

Despite its advantages, aerobic digestion also has several disadvantages relative to anaerobic digestion, including (1) high power costs and (2) the resulting sludge has poor dewatering characteristics.

3.2.4 Autothermal thermophilic aerobic digestion (ATAD) process

A variation on the standard aerobic digestion process is the autothermal thermophilic aerobic digestion (ATAD) process. There is growing interest in this technology following increased scientific and regulation concern about sludge-borne pathogens [14,50]. Studies on the ATAD sludge process indicate that it can reduce fecal coliforms, *Salmonella,* and *Ascaris* ova to nondetectable levels. Therefore, the process does meet the current U.S. criteria for processes to further reduce pathogens (PFRPs). However, since the data on fecal streptococci reduction has been variable, the ability of the ATAD process to

Figure 3.59 An autothermal aerobic digestion system.

meet the criteria established for processes that significantly reduce pathogens (PSRPs) is less certain.

The ATAD system normally is a two-stage aerobic process that operates under thermophilic conditions (130–150°F, or 55–65°C) without supplemental heat (Fig. 3.59). The system relies on the microbial heat released during the aerobic digestion process to achieve and sustain the desired operating temperature [15,50]. The ATAD process has many benefits relative to other biological stabilization processes, including (1) high disinfection potential, (2) low space and volume requirements, (3) minimum nitrification rates, (4) significant loss of organic matter (i.e., volatile solids), and (5) treated sludge may be applied to land with minimal or no management restrictions for pathogen control [50].

ATAD reactors usually are cylindrical in shape and constructed of carbon steel with a coal tar–based interior. ATAD tanks are insulated with mineral wool (*ca.* 6 in thick) around the tank walls and styrene foam material on the cover, resulting in a typical heat-transfer coefficient of 0.3 to 0.4 $W/m^2 \cdot {}^\circ C$. The entire tank is covered with a metallic skin (corrugated steel or aluminum) to protect the insulation and enhance the appearance of the tank.

3.2.4.1 Fundamentals of autothermal thermophilic aerobic digestion.

In the ATAD process, the heat released by the digestion process is the major heat source used to achieve the desired operating temperature. Figure 3.60 illustrates the various energy and mass inputs and outputs for the ATAD system.

Autothermal conditions result from an adequately thickened sludge feed, suitably insulated digestion tanks, good mixing, and an efficient aeration device that minimizes latent heat loss. Typical design parameters for the ATAD system are provided in Table 3.15.

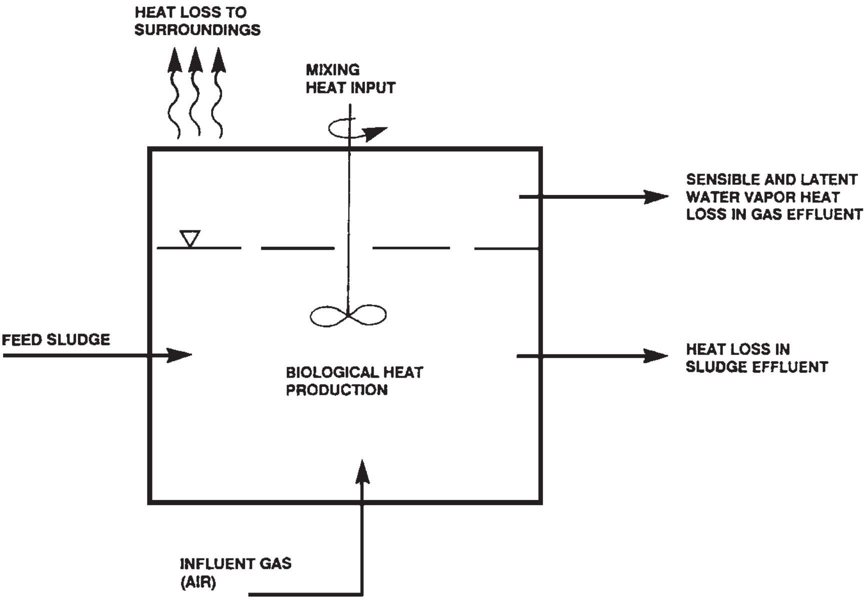


Figure 3.60 Heat balance for the autothermal aerobic digestion system [50].

TABLE 3.15 Design Parameters for Autothermal Thermophilic Aerobic Digestion*

Number of reactors	Two or more; tanks should be of equal size, operating in series.
Reactor type	Cylindrical; height/diameter ratio of 0.5 to 1.0
Normal sludge type	Primary Secondary: waste-activated sludge Secondary: trickling filter Mixture of primary and secondary sludge Gravity or dissolved-air flotation thickened
Feed total solids range	4–6% (40,000–60,000 mg/liter)
Detention time	5–6 days
Temperature and pH	Reactor I: 35–50°C, pH 7.2 Reactor II: 50–65°C, pH ~8.0
Air input	4 $m^3/m^3 \cdot$ reactor per hour
Specific power	85–105 $W/m^3 \cdot$ reactor
Energy requirement	9–15 $kWh/m^3 \cdot$ reactor
Potential heat recovery	20–30 kWh/m^3 of sludge treated

*Adapted from ref. [50].

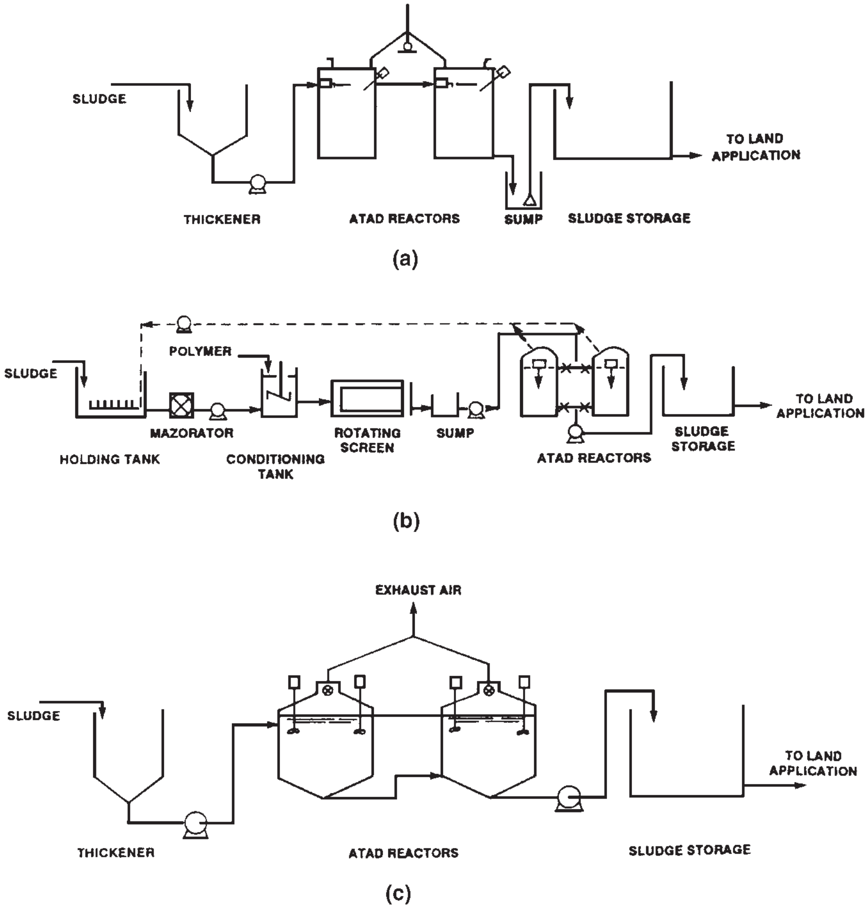


Figure 3.61 Schematic designs of three ATAD systems: (*a*) Fuchs system; (*b*) Thieme system; (*c*) Limus system.

Figure 3.61 illustrates three typical ATAD process configurations. Typical design features include a prethickener, two enclosed and insulated reactors configured in series, a mixing/aeration system, foam control equipment, and a final storage/postthickening tank.

ATAD systems are intended to reduce the total mass of sludge requiring disposal and to produce a stabilized final product suitable for disposal or reuse. Data from European applications of this technology indicate a minimum volatile suspended solids removal efficiency of 35 to 45 percent [50].

Like other digestion processes, ATAD systems exist both as single- and multiple-stage processes. Single-stage systems can achieve similar volatile solids reduction to multiple-stage systems but cannot reduce pathogen concentrations to the same extent. Therefore, single-stage configurations are more suitable for restricted disposal options that do not require enhanced pathogen reduction.

3.2.4.2 Sludge feed storage and thickening requirements. Raw sludge is thickened prior to discharge into an ATAD system to minimize the size of the digestion tanks and to limit the energy requirements for heating and mixing. Like composting, ATAD systems rely on conserving the heat released during microbial destruction of organic compounds to attain and sustain an operating temperature of 55 to 65°C (130–150°F). Therefore, the sludge is both the material being treated and the fuel that drives the process.

Typical influent sludge solids concentrations are 4 to 6 percent, with at least 65 percent of the solids being volatile [33,50]. In some plants, waste-activated sludge is cothickened with primary solids, a process variation that eliminates the need for a separate sludge thickener [50]. Typical hydraulic retention times for ATAD systems will be in the range of 5 to 6 days. At this level, pathogen destruction and volatile solids reduction are significant [50].

3.2.4.3 Feed cycle and reaction time. ATAD reactor tanks normally are fed and operate in a batch mode. The feed sludge pumping system should be sized to deliver the daily thickened sludge volume to the reactor tank in less than 1 hour. At the end of ATAD treatment, aeration and mixing are stopped, and stabilized sludge is discharged from the second reactor into the sludge storage tank (Fig. 3.61). Sludge from the first tank is then allowed to flow into the second tank until both tanks reach equilibrium. Raw sludge is then added to the first tank to raise the level and to displace sludge into the second tank. When both tanks reach full operating level, aeration and mixing are resumed.

3.2.4.4 Aeration and mixing. Aeration and mixing efficiency are two of the most important factors to consider in the design of an ATAD system. A highly efficient aeration system is needed to (1) supply sufficient oxygen to meet the high oxygen demand and (2) minimize the latent heat loss that occurs when process air is exhausted. Both the airflow rate and oxygen transfer efficiency depend on system geometry, sludge characteristics (viscosity, solids content), and turbulence conditions. The most widely established ATAD aeration and mixing system is the Fuchs system, which is characterized by two side-mounted aspirator aerators (Fig. 3.62).

If needed, a third aerator can be mounted in the center of the reactor. Typical design ranges for aeration/mixing requirements for ATAD systems are provided in Table 3.16.

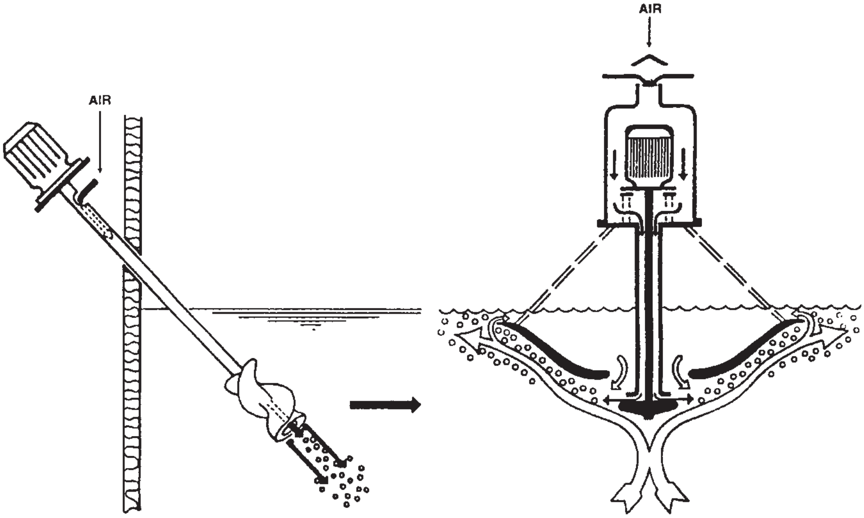


Figure 3.62 Fuchs aerator/mixer used in autothermal thermophilic aerobic digester systems [50].

TABLE 3.16 Design Requirements for Mixing and Aerating ATAD Systems*

Parameter	Typical value
Specific power	85–105 W/m^3 (3.2–3.9 hp/10^3 ft^3)
Air input	4 m^3/m^3 · h (4 ft^3/ft^3 · h)
Energy requirement	9–15 kWh/m^3 of sludge throughput

*Adapted from ref. [50].

It should be noted that the specific power range of 85 to 105 W/m^3 (3.2–3.9 hp/10^3 ft^3) represents a high mixing intensity while the energy requirement of 9 to 15 kWh/m^3 of sludge throughput includes the energy required to operate the foam cutters [9,33,50].

3.2.4.5 ATAD: Temperature and pH considerations. For proper operation, an average temperature of at least 55°C (130°F) in the secondary reactor is desired. Although the temperature in the second reactor is often higher than that in the first, it should not be allowed to exceed 65°C (150°F) to prevent solubilizing of organic compounds. During batch feed, the first tank will experience a temperature decrease of between 5 and 10°C (9 and 18°F). To avoid microbial acclimation problems, the temperature should not be allowed to fall below 25°C (77°F). The design factors that have a significant impact on system temperature include the efficiency of the aeration systems, reactor tank insulation, foam management, and sludge prethickening.

Generally, digester pH does not have to be controlled in ATAD systems. The thermophilic operating temperatures effectively suppress nitrification, which is the principal cause for pH reduction in aerobic digester systems operated at lower temperatures. With a feed sludge pH of 6.5, the pH value in the first ATAD reactor tank is typically 7.2, and may be as high as 8.0 in the second tank.

3.2.4.6 ATAD foam control. The foam layer that forms in the ATAD reactor tank provides several important operational benefits, including (1) improved oxygen use, (2) enhanced biological activity, and (3) reduced heat loss [9,50]. However, despite these benefits, the foam layer also retards the amount of air entering the process.

The amount of foam should be managed within the ATAD reactor tank within 20 to 30 cm (8–12 in). Foam control consists of densifying the foam (i.e., breaking up the large foam bubbles into smaller bubbles) to form a compact layer floating above the liquid surface. A typical foam cutter is shown schematically in Fig. 3.63.

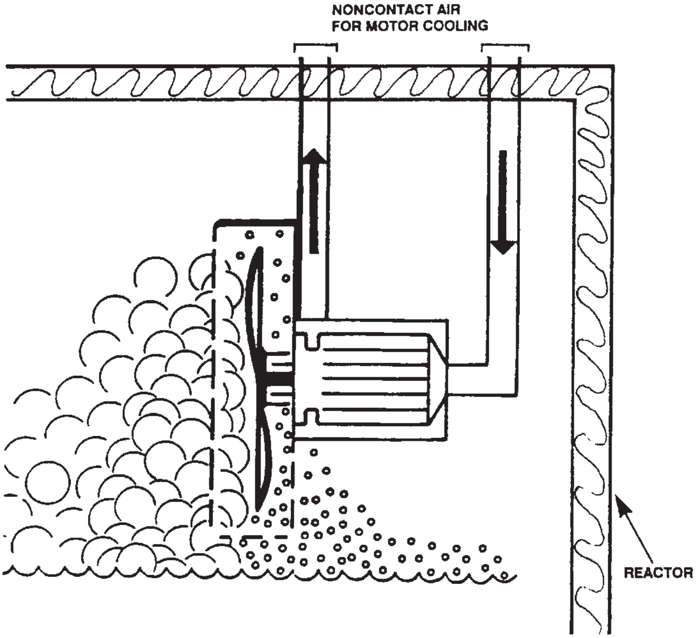


Figure 3.63 Schematic diagram of a foam cutter used in an ATAD systems [50].

The design basis for foam cutters is determined empirically and is a factor of the surface area of the reactor. All ATAD reactor tanks should be equipped with at least two foam cutters.

3.2.4.7 ATAD: Postthickening sludge management. In general, the gravity thickening performance of the effluent sludge is poor immediately after treatment by the ATAD process due to the thermal convection currents that occur in the thickening tank. If the sludge is allowed to cool in the postthickening/storage tank or additional heat exchangers exist that can cool the sludge, thickening usually is satisfactory. Within the sludge thickening tank, sludge has been found to thicken to a solids content in the range of 6 to 14 percent [9,33,50]. It should be noted that high thickening sludge solids concentrations can lead to problems with sludge pumping if not considered during design.

3.2.4.8 ATAD heat management. In principle, no heat exchangers are required to operate an ATAD process. However, heat-exchange equipment occasionally is used to achieve certain process objectives such as (1) cooling the stabilized effluent sludge coupled with the preheating of influent sludge and (2) cooling the reactor to prevent thermal “runaway” of reactor contents. The feasibility for energy recovery strongly depends on local conditions such as the (1) heating potential of the influent sludge, (2) heat demand of the process, (3) climatic conditions, and (4) potential uses of low-temperature heat.

Heat energy equivalent to 15 to 30 kWh/m^3 of sludge processed can be recovered from the ATAD system [50]. The feasibility for energy recovery strongly depends on local conditions, such as the heat potential of the sludge, the heat demand of the process itself, climatic conditions, and the potential proximate uses for low-temperature heat.

3.2.4.9 ATAD odor control. ATAD system odors have been characterized as humus-like with traces of ammonia. Ammonia is released by thermophilic anaerobic degradation of protenaceous material. The ammonia released is not nitrified due to the suppression of nitrification at thermophilic temperatures [9]. Depending on the pH of the reactor, ammonia can be stripped into the exhaust air. ATAD systems typically exhibit an elevated pH, particularly in the second-stage reactor tank, that enhances the stripping potential of ammonia.

Odor control can be accomplished by returning the exhaust air to the activated-sludge aeration tank or into a trickling filter [50]. For situations requiring long pipe runs or more precise control, odor-control devices are used. A commonly used odor-control device for air exhaust from ATAD systems is the biofilter (Fig. 3.64). Example 3.20 provides a standard step-wise approach for design of ATAD systems.

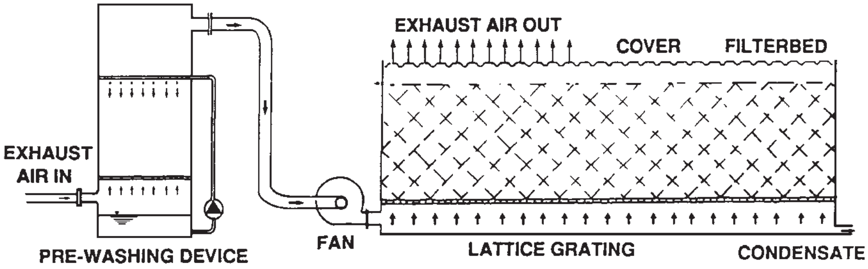


Figure 3.64 Schematic diagram of a biofilter system for ATAD gas treatment [50].

Example 3.20 The Garissa County Wastewater Treatment Plant has determined that the aerobic digester volume required for sludge stabilization is excessive (see Example 3.19) given the currently available space. The plant superintendent would like to evaluate the potential of stabilizing the sludge at higher temperatures by employing the autothermal aerobic digestion (ATAD) process. For a dual-stage ATAD system, estimate the volatile solids and fixed solids flow rate (lb/day) together with the oxygen requirements if the hydraulic retention time for the entire system is maintained at 6 days. Assume that the following conditions apply:

Performance expected:

- 45 percent reduction in volatile solids achieved by ATAD
- 65 percent of volatile solids removed in first stage
- Oxygen equivalent of volatile solids removed is 1.42 (i.e., 1.42 lb O_2 needed per lb VS removed)
- Oxygen represents 23.3 percent of air (weight basis)
- Density of air is 0.075 lb_m/ft^3

Feed conditions:

- Sludge flow rate (gal/day): 22,000
- Total solids content (percentage): 3.0 percent
- Volatile fraction of total solids (percentage): 80 percent

solution

Step 1. Estimate the total solids, volatile solids, and fixed solids in the ATAD feed to the first reactor.

Total solids loading (lb/day)

$$= \text{flow rate (MG/day)} \cdot \text{concentration (mg/liter)} \cdot 8.34\ \text{lb/MG} \cdot \text{(mg/liter)}$$

$$= \frac{22{,}000\ \text{gal/day}}{1 \cdot 10^6\ \text{gal/MG}} \cdot \frac{30{,}000\ \text{mg}}{\text{liter}} \cdot \frac{8.34\ \text{lb}}{\text{MG} \cdot \text{(mg/liter)}}$$

$$= 5504.4\ \text{lb/day}$$

$$\text{Volatile solids loading (lb/day)} = \text{total solids loading (lb/day)} \cdot 0.8$$

$$= 5504.4 \text{ lb/day} \cdot 0.8 = 4403.5 \text{ lb VS/day}$$

Fixed solids loading (lb/day)

$$= \text{total solids loading (lb/day)} - \text{volatile solids loading (lb/day)}$$

$$= 5504.4 \text{ lb TS/day} - 4403.5 \text{ lb VS/day}$$

$$= 1100.9 \text{ lb FS/day}$$

Step 2. Estimate the total solids, volatile solids, and fixed solids flow rates in the effluent from reactor I and reactor II for the dual-stage ATAD system.

Reactor I:

$$\text{Volatile solids flow rate (lb/day)} = 4403.5 \text{ lb VS/day } [1 - (0.45 \cdot 0.65)]$$

$$= 3115.5 \text{ lb VS/day}$$

$$\text{Fixed solids flow rate (lb/day)} = 1100.9 \text{ lb FS/day}$$

Total solids flow rate (lb/day)

$$= \text{volatile solids flow rate} + \text{fixed solids flow rate}$$

$$= 3115.5 \text{ lb VS/day} + 1100.9 \text{ lb FS/day}$$

$$= 4216.4 \text{ lb TS/day}$$

Reactor II:

$$\text{Volatile solids flow rate (lb/day)} = 4403.5 \text{ lb VS/day } (1 - 0.45)$$

$$= 2421.9 \text{ lb VS/day}$$

$$\text{Fixed solids flow rate (lb/day)} = 1100.9 \text{ lb FS/day}$$

Total solids flow rate (lb/day)

$$= \text{volatile solids flow rate} + \text{fixed solids flow rate}$$

$$= 2421.9 \text{ lb VS/day} + 1100.9 \text{ lb FS/day}$$

$$= 3522.8 \text{ lb TS/day}$$

Step 3. Estimate the size of each reactor. Since a system hydraulic retention time of 6 days is desired, each stage is assumed to contribute 50 percent, or 3 days, of retention time.

$$\text{Hydraulic retention time } \theta \text{ (days)} = \frac{\text{volume of reactor (gal)}}{\text{volumetric flow rate (gal/day)}}$$

or

$$\text{Volume of reactor (gal)} = \theta\ \text{(days)} \cdot \text{volumetric flow rate (gal/day)}$$

$$= 3\ \text{days} \cdot 22{,}000\ \text{gal/day} = 66{,}000\ \text{gal}$$

Step 4. Estimate the pounds of oxygen that must be supplied each day if 45 percent of the influent volatile solids are removed:

$$\text{Oxygen flow rate (lb } O_2\text{/day)}$$

$$= \text{rate of VS removed (lb VS/day)} \cdot (1.42\ \text{lb } O_2\text{/lb VS})$$

$$= (0.45)\,(4403.5\ \text{lb VS/day})\,(1.42\ \text{lb } O_2\text{/lb VS})$$

$$= 2813.8\ \text{lb } O_2\text{/day}$$

Step 5. Estimate the minimum airflow rate (ft^3/min) to meet the required oxygen demand:

$$\text{Airflow rate } (ft^3\text{/min})$$

$$= \frac{\text{oxygen flow rate (lb } O_2\text{/day)}}{0.232\ \text{lb } O_2\text{/lb air}} \cdot \frac{ft^3\ \text{air}}{0.075\ \text{lb air}} \cdot \frac{\text{day}}{1440\ \text{min}}$$

$$= \frac{2813.8\ \text{lb } O_2\text{/day}}{0.232\ \text{lb } O_2\text{/lb air}} \cdot \frac{ft^3\ \text{air}}{0.075\ \text{lb air}} \cdot \frac{\text{day}}{1440\ \text{min}}$$

$$= 112.3\ ft^3\ \text{air/min}$$

3.2.5 Lime stabilization

In addition to anaerobic and aerobic digestion, thickened sludge also may be stabilized by chemical treatment. The principal forms of chemical treatment are lime stabilization and chlorine oxidation. Although lime stabilization remains a popular method of sludge stabilization, because of growing concern regarding release of chlorinated organic compounds into the environment, chlorine oxidation of sludge is rarely used today.

Lime stabilization involves the addition of lime to thickened sludge to raise its pH to 12.0, where it should remain for at least 2 hours [38]. At this pH level, microbial activity ceases, resulting in a significant reduction in pathogen concentrations [9,74]. The principal advantages of lime stabilization over other stabilization processes are low cost and simplicity of operation.

It is important to recognize that no direct reduction of organic matter occurs during lime stabilization of thickened sludge. This fact results in two important limitations to the technology. First, lime addition does not render sludge chemically stable. If, for any reason, the sludge pH drops below 11.0, microbial activity may resume, resulting in the production of noxious odors [33]. Second, the quantity of sludge requiring disposal as a result of lime stabilization is not reduced as it is in biological stabilization processes. On the contrary, lime addition and the chemical precipitates that form during lime stabilization increase the quantity of sludge requiring disposal. Because of the increased quantities, the transport and disposal costs of lime-stabilized sludge are normally greater than those for sludge stabilized by other methods.

Lime stabilization also causes chemical changes in sludge, including reduced concentrations of soluble phosphate, ammonia, nitrogen, and total Kjeldahl nitrogen (TKN) compared with anaerobically digested sludge [74]. The reduction in nitrogen concentration in lime-stabilized sludge is caused by both dilution by chemical addition and volatilization loss of nitrogen (as ammonia). The lower nutrient concentrations in lime-stabilized sludge reduce its value for agricultural and land reclamation use. However, assuming that nitrogen limits the application rate, more lime-stabilized sludge can be applied per acre of land compared with sludge stabilized by other methods [38,74].

3.2.5.1 Lime characteristics. Lime is a caustic material that can cause severe injury, and therefore, special equipment must be used to handle it. The two forms of lime available commercially are quicklime (CaO) and hydrated lime [$Ca(OH)_2$]. Quicklime is rarely applied directly to thickened sludge. Normally, quicklime is first converted to hydrated lime (a process called *slaking*) according to the reaction described by Eq. (3.45):

$$\underset{\text{Quick lime}}{CaO} + H_2O \rightarrow \underset{\text{Hydrated lime}}{Ca\,(OH)_2} + \text{heat} \qquad (3.45)$$

If slaking is done by the lime manufacturer, hydrated lime is delivered to the wastewater treatment plant. At the treatment plant, the hydrated lime powder is slurried with more water prior to mixing it with sludge. Alternatively, slaking may be carried out at the wastewater treatment plant. In this case, the delivered product is quicklime that first must be slaked and then diluted prior to process application. The decision whether to purchase quicklime or hydrated lime is influenced by a number of factors, such as the size of the treatment facility, material cost, and storage requirements [38].

3.2.5.2 Process fundamentals for lime stabilization. Lime stabilization reduces the concentration of pathogens by creating an alkaline pH that is inhibitory to microbial activity. The three process parameters that must be considered in the design of a lime stabilization system include (1) pH, (2) contact time, and (3) lime dosage.

With regard to pH and contact time, the design objective is to maintain the pH above 12.0 for 2 hours to ensure pathogen destruction and to provide sufficient residual chemicals to ensure that the pH does not drop below 11.0 for several days. The recommended design criteria for accomplishing these objectives include

- Treat sludge in the liquid state.
- Bring sludge to a pH 12.5 by lime addition.
- Maintain pH above 12.5 for at least 30 minutes.

The required lime dosage for effective stabilization is determined by the following parameters: (1) sludge type (i.e., primary, secondary), (2) chemical composition, and (3) moisture content.

In general, primary sludge requires less lime than secondary sludge, except where either iron salts (e.g., ferric chloride) or alum has been used for chemical precipitation [38]. When iron salts or alum has been used in primary wastewater treatment, a significant portion of the added lime reacts to form iron and/or aluminum hydroxide species [33,74]. Under these circumstances, considerably more lime is required to achieve and maintain the required pH for adequate stabilization (Table 3.17).

If managed properly, significant pathogen concentration reductions can be achieved in thickened sludge that has been treated with lime. Table 3.18 provides field data about pathogen-reduction efficiency during full-scale lime stabilization operations.

3.2.5.3 Lime stabilization system description. The conceptual design for a lime stabilization process is presented in Fig. 3.65. Prior to lime addition, the thickened sludge typically is passed through an in-line grinder. This pretreatment step improves sludge mixing and flow characteristics and protects downstream processing equipment.

From the grinder (or comminutor), the thickened sludge is conveyed to one of two batch mixing tanks. Each tank has the capacity to treat the total sludge produced in a typical 8-hour shift at the wastewater treatment plant. While one tank is filling, sludge from the other tank is mixed continuously and dosed with lime until a pH of 12.5 is reached and maintained for at least 30 minutes. If the pH then remains at or above 12.0 for an additional 2 hours, the sludge is considered stable. The stabilized sludge may then be conveyed to the land treatment site or transferred to downstream processing operations.

Although the standard lime stabilization process treats liquid wastewater sludge, recent advances in the use of lime have permitted the stabilization of dewatered sludge as well. Figure 3.66 depicts a com-

TABLE 3.17 Lime Doses Required for Stabilization*

Sludge type	Solids concentration		Lime dosage, lb $Ca(OH)_2$/lb dry solids		pH average	
	Range	Average	Range	Average	Initial	Final
Primary sludge	3–6	4.3	0.06–0.17	0.12	6.7	12.7
Waste-activated sludge	1–1.5	1.3	0.21–0.43	0.30	7.1	12.6
Anaerobically digested sludge	6–7	5.5	0.14–0.25	0.19	7.2	12.4
Septage	1–4.5	2.7	0.09–0.51	0.20	7.3	12.7

*Adapted from ref. [38].

TABLE 3.18 Effect of Lime Dose on Pathogen Levels*

	Bacterial density, no/100 ml			
	Total coliforms	Fecal coliforms	Fecal streptococci	*Salmonella*†
Raw				
Primary	$2.9 \cdot 10^9$	$8.3 \cdot 10^8$	$3.9 \cdot 10^7$	62
Waste activated	$8.3 \cdot 10^8$	$2.7 \cdot 10^7$	$1.0 \cdot 10^7$	6
Septage	$2.9 \cdot 10^7$	$1.5 \cdot 10^7$	$6.7 \cdot 10^5$	6
Anaerobically digested	$2.8 \cdot 10^7$	$1.5 \cdot 10^6$	$2.7 \cdot 10^5$	6
Lime-stabilized				
Primary	$1.2 \cdot 10^5$	$5.9 \cdot 10^3$	$1.6 \cdot 10^4$	<3
Waste activated	$2.2 \cdot 10^5$	$1.6 \cdot 10^4$	$6.8 \cdot 10^3$	<3
Septage	$2.1 \cdot 10^3$	265	665	<3
Anaerobically digested	18	18	$8.3 \cdot 10^5$	<3

*Adapted from ref. [38].

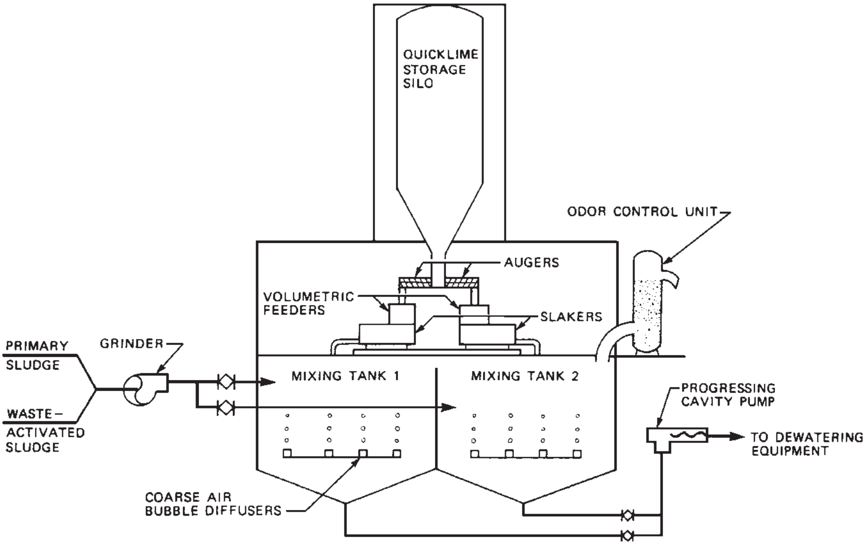


Figure 3.65 Schematic diagram of a lime stabilization system.

mercially available lime treatment process that stabilizes dewatered wastewater sludge while capturing and recycling the volatilized ammonia for use as a liquid fertilizer.

3.2.6 Chlorine oxidation

Stabilization of sludge by the addition of chlorine was developed as a proprietary process and is marketed under the registered trademark

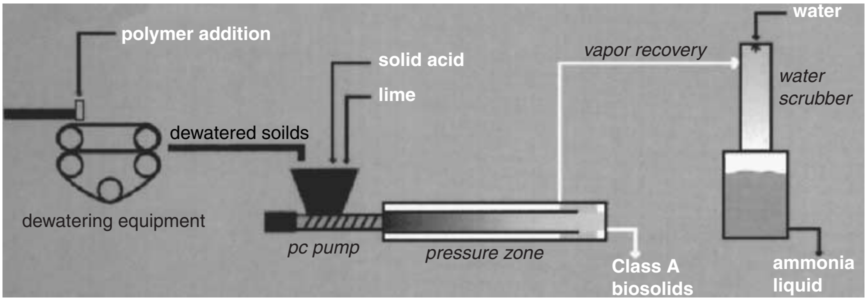


Figure 3.66 Schematic diagram of the BIOSET process. (*Courtesy of BIOSET, Inc.*)

Purifax. One advantage of this stabilization process is that because chlorine reacts rapidly with sludge, the required reactor volume (and hence capital costs) is relatively small. The process is effective in reducing pathogen concentrations but has only a marginal effect on reducing the volatile solids content [38]. Although effective in reducing pathogen concentrations, there is a concern that if chlorine-stabilized sludge is applied to land, it could cause adverse human health and environmental effects because of the presence of chlorinated organic compounds [38].

3.2.6.1 Process description. Chlorine oxidation stabilizes sludge both by reducing the number of microorganisms present (both pathogens and innocuous microorganisms) and by rending organic substrates less suitable for microbial metabolism and growth. The immediate reaction from the addition of gaseous chlorine to thickened sludge is described in Eqs. (3.46) and (3.47).

$$\mathrm{Cl_2 + H_2O \rightarrow HOCl + HCl} \tag{3.46}$$

$$\mathrm{HOCl \rightarrow H^+ + OCl^-} \tag{3.47}$$

The total amount of HOCl plus OCl^- that is formed is called the *free available chlorine*. The relative distribution of these two species is critical because pathogen reduction efficiency of HOCl is about 40 to 80 times that of OCl^-. Sufficient acid is generated during the chlorine stabilization process to reduce the pH of the sludge to between 2 and 3. Hypochlorous acid (HOCl) and residual Cl_2 react to oxidize ammonia to chloroamines that are effective in destroying pathogens. The formation of chloroamines depends on pH, temperature, contact time, and the ratio of chlorine to ammonia. The chlorine contained in the chloroamines is termed *combined available chlorine*.

Because of the concern that chlorine oxidation of sludge could result in increased levels of toxic chlorinated organic materials released into the environment, chlorine stabilization is not widely employed. Wastewater treatment facilities considering chlorine oxidation of sludge must work closely with local public health officials to ensure that both public health and the environment are not exposed to unnecessary risks [38].

3.2.7 Vermistabilization

Vermistabilization is the process of sludge stabilization and dewatering using earthworms [30]. The benefit of this technology is the potential to stabilize and dewater sludge in one step as opposed to thickening, digestion, conditioning, and dewatering in a conventional process (Fig. 3.67). It should be noted that this technology is only feasible for sludges that contain sufficient organic matter and nutrients to support the earthworm population.

In most locations, the facilities required for vermistabilization will be similar to an underdrained sand drying bed enclosed in a heated shelter. Research studies have indicated that the worm species *Eisenia foetida* has the best growth and reproduction responses when added to sludge [30,38]. The optimal temperature range for worm growth is 68 to 77°F (20–25°C).

In this process, worms are placed on the sludge bed in a single initial application of approximately 0.4 lb of worms (live weight) per

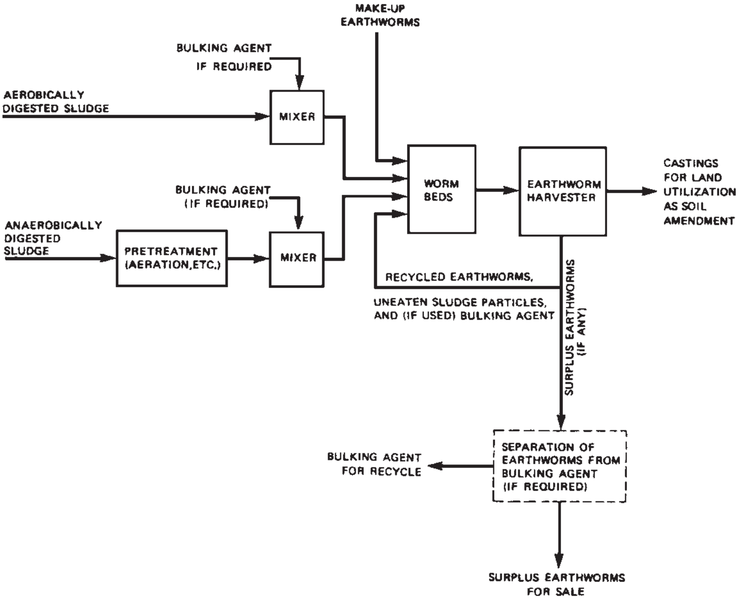


Figure 3.67 Process diagram for a vermistabilization system.

TABLE 3.19 Characteristics of Stabilized Sludge from Vermistabilization Operations*

Parameter	Range
Total solids (%)	14–24
Volatile solids (g/kg TS)	460–550
Chemical oxygen demand (g/kg TS)	606–730
Organic nitrogen (g/kg TS)	27–35
pH	6.6–7.1

*Adapted from ref. [38].

square foot of sludge bed (2 kg/m^2). A sludge loading rate of approximately 0.2 lb of volatile solids (VS) per square foot of sludge bed per week (1 kg VS/m^2 · week) is recommended for both liquid primary and liquid waste activated sludge [30]. Typical results from loading a sludge having a solids content of between 0.6 and 1.3 percent solids to a vermistabilization system are summarized in Table 3.19. The final stabilized sludge had a solids content that ranged from 14 to 24 percent, with the final physical and chemical characteristics found to be independent of the type of sludge applied initially (Table 3.19).

3.2.7.1 Vermistabilization process operation. During the vermistablization process, sludge is ingested and metabolized by the earthworms. The resulting solids pass through the guts of the earthworms and emerge as dry, virtually odorless castings (i.e., worm feces). The castings are suitable for use as a soil amendment or low-grade fertilizer if metals are within acceptable limits. Moreover, in several full-scale demonstration projects, no *Salmonella* species were detected in worm castings [30].

With regard to the excess earthworms produced within the system, the major markets include using the earthworms as bait for sport fishing as well as for processing into animal or fish food. However, numerous studies have shown that earthworms accumulate heavy metals from sludge [38]. Therefore, earthworms from a sludge stabilization operation should not be the major food source for animals or fish used in the commercial production of food for human consumption.

Effect stabilization processes result in the reduction of the concentration of pathogenic organisms. However, in some stabilization processes, e.g., lime stabilization, thermophilic aerobic digestion, thermophilic anaerobic digestion, chlorine oxidation, etc., the resulting sludge product is essentially free of all pathogens—a quality defined as *disinfection*. Although sludge disinfection is not currently required by U.S. regulations, sludge disinfection is becoming an important consideration, particularly at biosolids land-application sites to which there is significant public access. In addition to the stabilization processes that also can be employed to achieve sludge disinfection, several other technologies can be used to achieve sludge disinfection, namely, sludge pasteurization and irradiation.

3.2.8 Pasteurization

Applying heat to untreated or digested sludges can effectively eliminate pathogenic organisms. Heat may be used solely for pathogen reduction as in pasteurization or in conjunction with sludge conditioning. The critical technical requirement in sludge pasteurization is that all sludge be held above a predetermined temperature for a minimum time period. Although heat transfer can be accomplished by steam injection or with the use of heat exchangers, steam injection has been shown to be the most effective heating method for pathogen control [38,74].

Proper mixing is critical to the success of sludge pasteurization. Incomplete mixing will either increase the minimum required heating time, reduce process effectiveness, or both. However, overheating or an excessively long detention time is not desirable because trace metal mobilization may be increased, odor problems will be exacerbated, and unneeded energy will be expended. The flow scheme for a typical pasteurization system equipped with a one-stage heat recuperation unit is shown in Fig. 3.68. Principal system components include (1) a steam boiler, (2) a preheater, (3) a sludge heater, (4) a high-temperature

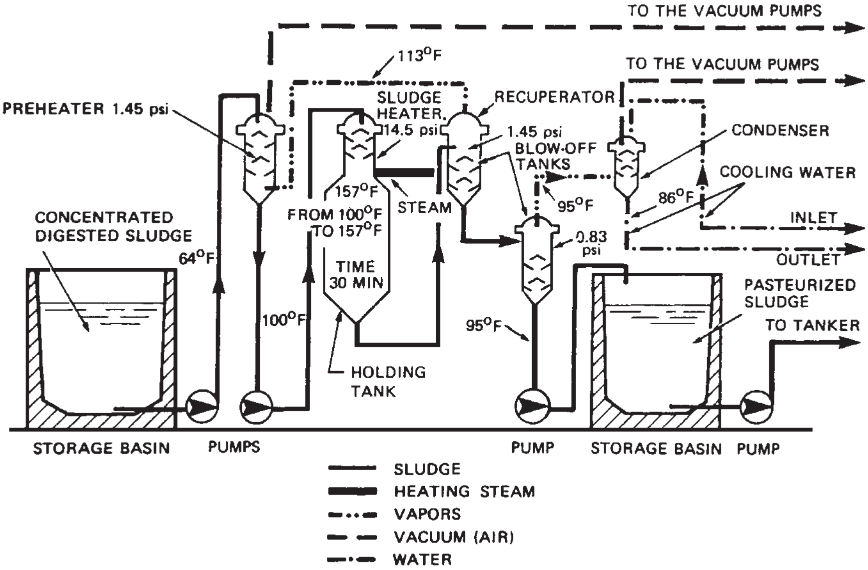


Figure 3.68 Sludge pasteurization system, single-stage heat recuperation.

sludge holding tank, (5) blow-off tanks, and (6) storage basins for the untreated and treated sludge.

In the typical batch processing operation, thickened sludge enters the preheater stage, where its temperature is raised from approximately 64°F (18°C) to 100°F (38°C) by hot vapors discharged from the blow-off tanks. After the preheating stage, the sludge is subjected to direct steam injection that raises its temperature to at least 157°F (70°C) in the pasteurization tank. For effective pathogen destruction, the sludge remains in the pasteurization tank for at least 30 minutes. Finally, the sludge is transferred to the blow-off tanks, where it is cooled through the use of fans or blowers to 113°F (45°C) at 1.45 psig (10 kN/m^2) pressure and then to 98°F (35°C) at 0.73 psig (5 kN/m^2) pressure [38].

For sludge flow rates of 50,000 to 70,000 gal/day, a single-stage heat recuperation system typically is used, whereas for flows of 70,000 to 130,000 gal/day, a two-stage system is employed. For larger sludge flows, a three-stage heat recuperation system would be used [38].

3.2.8.1 Sludge pasteurization system design. A pasteurization system should be designed to maintain the sludge at a uniform minimum temperature of 157°F (70°C) for at least 30 minutes. The use of in-line mixing of steam and sludge should be considered in order to increase the heat-transfer efficiency and to ensure uniform heating.

The sizing of the pasteurization system should be based on peak flow conditions, or sludge storage capacity should be integrated into the overall system design to reduce the required tank size. The capacity of sludge storage facilities and the pasteurization system will depend on the type of sludge treated, the average sludge flow, and the end use of the sludge. In general, storage capacity for pasteurized sludge should be adequate to hold at least a volume equivalent to 4 days' generation rate of processed sludge assuming average flow. If anaerobic digesters are not available for sludge storage, the storage facilities must be equipped for odor control or with aeration capacity to prevent septic conditions.

3.2.8.2 Pasteurization system operational considerations. To ensure proper conditions for pathogen destruction, temperature monitoring at several points in each pasteurization system is a minimum requirement. In addition to temperature monitoring, flowmetering devices, boiler controls, emergency pressure-relief valves, and level sensors in all process tanks should be employed. Figure 3.69 provides a schematic layout for the major process components.

One of the major considerations in the pasteurization process is the cost of heating. The required boiler capacity for steam production may be estimated using Eq. (3.48). Example 3.21 illustrates the basic approach for estimating the energy requirements for the steam boiler.

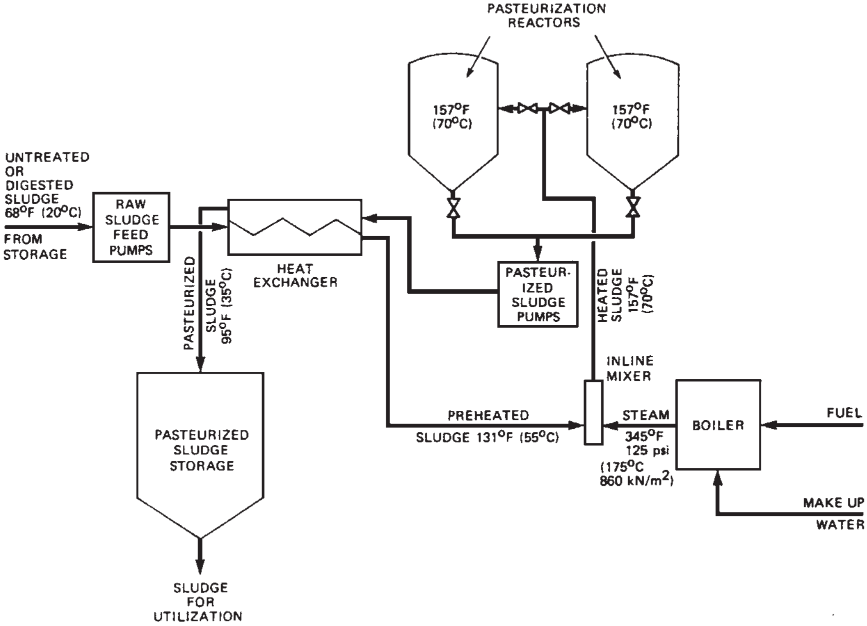


Figure 3.69 Process components in a sludge pasteurization system.

$$E = \frac{\Delta T h W}{e t} \tag{3.48}$$

where E = energy required to heat sludge, Btu/h
ΔT = temperature difference between sludge from the heat exchanger and the sludge in pasteurization tank
h = heat capacity of the sludge, Btu/lb · °F
W = wet weight of sludge, lb
t = time for heating, h
e = boiler energy conversion efficiency

Example 3.21 An engineering consultant hired by the Garissa County Wastewater Treatment Plant (see Example 3.20) has recommended that the facility employ pasteurization to reduce the pathogen content of its sludge. Assuming that the temperature of the preheated sludge exiting the heat exchanger is 48°C (118.4°F) and the final pasteurization temperature is 72°C (161.6°F), estimate the energy require by the boiler (Btu/h) to heat the sludge if the required heating time is 30 minutes and the boiler energy conversion efficiency is 80 percent. Other relevant information is as follows:

1. Sludge flow rate: 22,000 gal/day
2. Specific gravity of sludge: 1.03

3. Heat capacity of sludge: 1.1 Btu/lb · °F

solution

Step 1. Convert the volumetric flow rate of sludge to a given number of pounds per day (wet weight):

$$\text{Mass of wet sludge (lb/h)} = (22{,}000 \text{ gal/day})\,(\text{day}/24 \text{ h})\,(8.34 \text{ lb/gal}) \cdot 1.03$$

$$= 7874.4 \text{ lb/h}$$

Step 2. Using Eq. (3.48), estimate the minimum energy requirement:

$$E = \frac{\Delta T h W}{e t}$$

$$= \frac{(161.6°\text{F} - 118.4°\text{F})\,[1.1 \text{ Btu}/(\text{lb} \cdot °\text{F})]\,(7874.4 \text{ lb/h})}{(0.80)\,(0.5 \text{ h})}$$

$$= 935{,}472.8 \text{ Btu/h}$$

Although not used widely in the United States, sludge pasteurization is used widely in Europe and is required in both Switzerland and Germany prior to land application of biosolids [38,74]. In many wastewater treatment plants, biogas generated from anaerobic digestion is used as the fuel source for boiler operations. An example of a commercial system where this concept is employed is shown in Fig. 3.70. It should be noted that the pasteurization system in Fig. 3.70 employs three pasteurization reactors that permit the continuous processing and discharge of wastewater sludge.

Potential disadvantages of the pasteurization technology include odor problems and the need to provide storage facilities when land application of biosolids is not possible (i.e., winter months).

3.2.9 Sludge irradiation

High-energy radiation, particularly beta and gamma radiation, is an effective sludge disinfection process. Beta rays are high-energy electrons generated by an electron-accelerator device, whereas gamma rays are high-energy photons emitted from atomic nuclei of radioactive isotopes.

3.2.9.1 Disinfection with electron radiation. The pathogen-reducing power of beta rays (electron beams) depends on the number and energy of the electrons impacting the sludge. The energy of all radiation is measured in units called *rads*. One rad is equal to the absorption of $4.3 \cdot 10^{-6}$ Btu of energy per pound of material (i.e., 100 erg/g) [38]. Since beta radiation distributes energy throughout the volume of material regardless of the material penetrated, the disinfection effi-

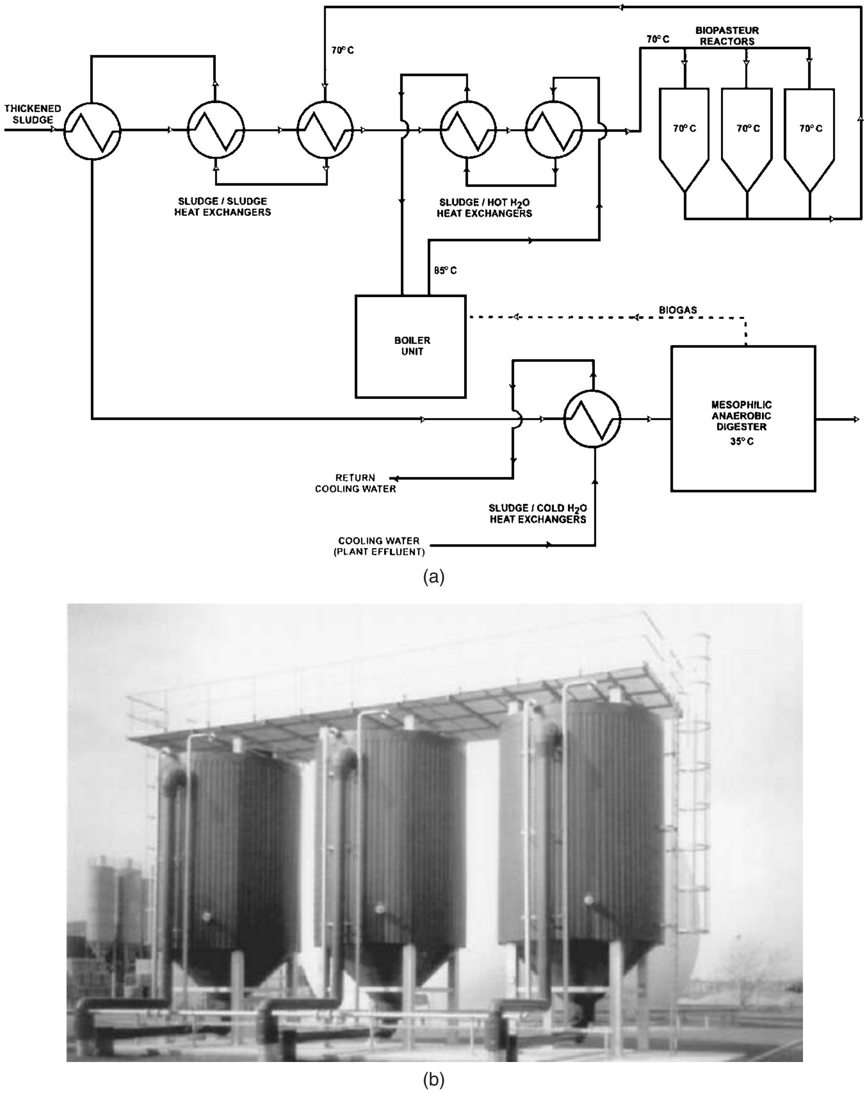


Figure 3.70 (*a*) Schematic diagram of the BioPasteur process. (*Courtesy of I. Kruger, Inc.*) (*b*) Pasteurization reactors. (*Courtesy of I. Kruger, Inc.*)

ciency only depends on the penetration depth of the radiation. The penetration depth of electrons is about 0.2 in (0.5 cm) in sludge when the electrons have been accelerated by a potential of 1 million V [38]. One approach to ensure effective beta-ray disinfection is to limit the thickness of the sludge layer being radiated to below this depth. The major system components of an electron-beam sludge radiation unit

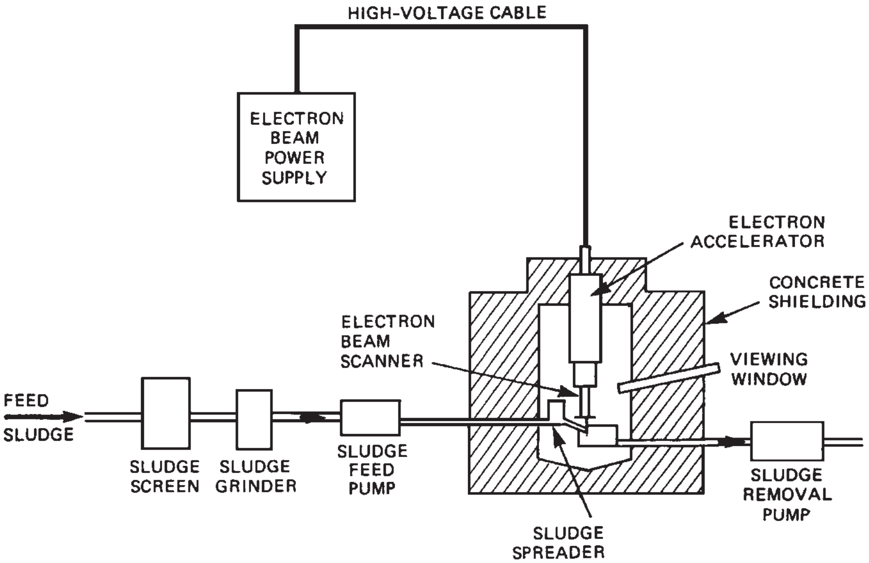


Figure 3.71 Major components of an electron-beam sludge disinfection system.

are depicted in Fig. 3.71. These components include (1) sludge screening, (2) sludge grinding, (3) sludge feed pumping, (4) a sludge spreader, (5) an electron-beam power supply, (6) an electron accelerator, (7) an electron-beam scanner, and (8) a sludge removal pump.

A concrete vault should be used to house the electron beam. Concrete is sufficient to provide radiation shielding for workers. The sludge pumps used in the electron-beam system should be progressive cavity or a similar type to ensure smooth sludge feed. Screening and grinding of sludge prior to radiation are mandatory to ensure that a uniform layer of sludge is passed under the electron beam. Design criteria for an electron-beam sludge facility should include a minimum electron radiation of 400,000 rads. This energy level effectively penetrates sludge depths of 0.2 in (0.5 cm), making the achievement of a uniform sludge layer less important than with lower-energy electrons. A schematic design of a typical sludge feed system is shown in Fig. 3.72.

As illustrated in Fig. 3.72, the beta rays (i.e., electrons) are accelerated and discharged in a continuous beam that is scanned back and forth over the sludge at 400 times per second. Adjusting the height of the underflow weir can be used to vary the radiation dosage.

3.2.9.2 Disinfection with gamma radiation. Gamma radiation produces similar pathogen destruction effectiveness as the electron-beam system. However, gamma rays differ from electrons in two major aspects. First, gamma radiation can penetrate to much greater depths than

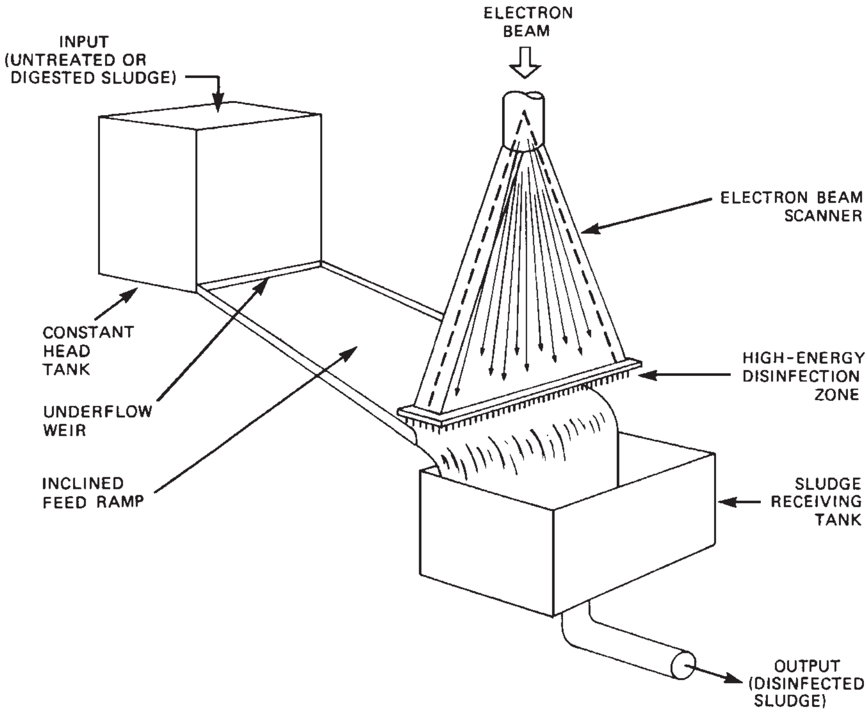


Figure 3.72 Sludge feed system for a beta-ray sludge disinfection system.

electrons. For example, a 25-in (0.64-m) layer of water is required to reduce the energy of gamma rays generated from a cobalt-60 source to 10 percent of its initial intensity [38]. Second, gamma rays are generated as the result of radioactive isotope decay, and therefore, their emission is continuous and uncontrollable. Moreover, the energy level (or levels) of the typical gamma ray from a given radioactive isotope is also relatively constant. Once an isotope is chosen for use as a radiation source, the applied energy to the sludge can only be varied with exposure time.

The two radioisotopes cesium-160 (^{160}Cs) and cobalt-60 (^{60}Co) are commonly used in sludge radiation systems. ^{137}Cs has a half-life of 30 years and emits a 0.66-MeV gamma ray, whereas ^{60}Co has a half-life of 5 years and emits two gamma rays with an average energy of 1.2 MeV.

3.2.9.3 Gamma-ray disinfection: Process description. Two general types of gamma-ray systems have been proposed for sludge disinfection. The first is a batch-type system for liquid sludge where the sludge is circulated in a closed vessel surrounding the gamma-ray source. Dosage of radiation is regulated by detention time and source strength.

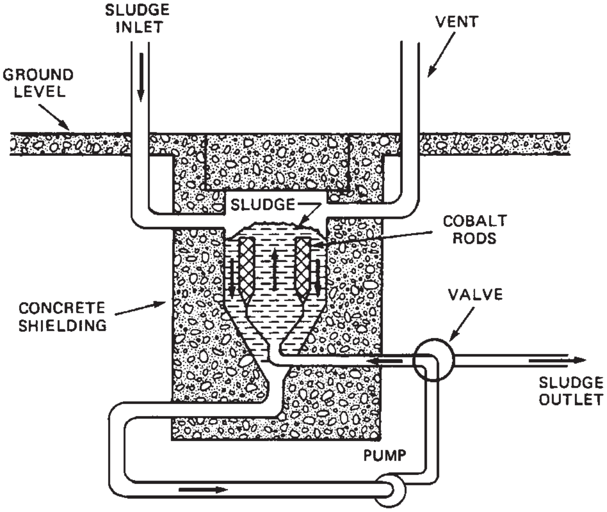


Figure 3.73 Gamma-ray disinfection system designed to treat liquid sludge.

A schematic diagram of such a liquid-sludge disinfection treatment system is provided in Fig. 3.73.

The second proposed system is for dewatered or composted sludge. A specially designed hopper conveyor is used to carry the material for irradiation to the gamma-ray source. Conveyor speed is used to control the radiation dosage to the sludge. A typical system for treatment of dewatered sludge is shown in Fig. 3.74.

When a dry-system gamma-radiation source is not in use, it should be shielded in a steel-lined concrete vault. The vault should be designed to be flooded with water during loading and unloading of the radiation

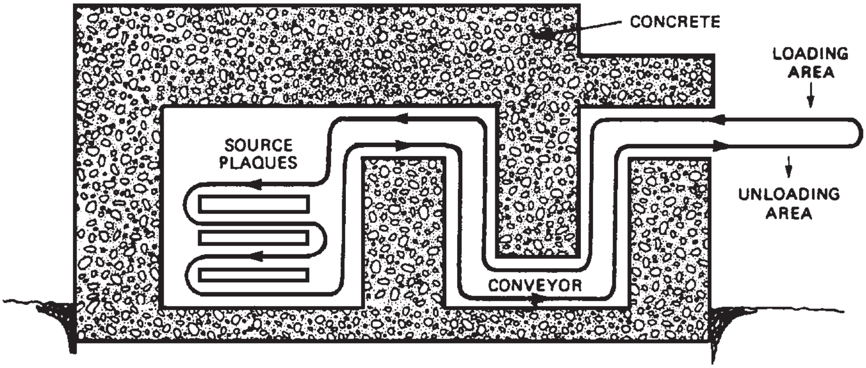


Figure 3.74 Gamma-ray disinfection system designed to treat dewatered sludge.

source to protect workers. Cooling air is circulated around the source during both system operation and downtimes. This air must be filtered to prevent a radioactive air release.

The design criteria for gamma-radiation facilities depend on the type of wastewater sludge being treated. Although a dose of 400 krad is sufficient to ensure complete virus removal, the dose level should be varied in relation to other treatment the sludge receives. For example, a composted and bagged biosolids product with an 80 percent solids content would require a lower radiation dose than a mixture of raw primary and waste-activated sludge because the dried product already has a reduced pathogen level.

3.2.10 Composting

Stabilization using anaerobic/aerobic digestion or chemical treatment involves the treatment of thickened sludge. In some cases, it may be cost-effective to postpone stabilization until after the sludge has been dewatered. An economical stabilization process used for dewatered sludge is composting.

Composting is the aerobic thermophilic stabilization of dewatered sludge [9,11]. The composting process is considered complete when the concentrations of pathogenic organisms in the final material have been reduced to levels specified in 40 CFR Part 503 and the material can be stored without giving rise to nuisance odors. If temperature and treatment time satisfy the new regulatory criteria, the final product should meet the Class A pathogen and vector attraction reduction requirements [11,38]. Several types of composting designs are available for dewatered sludge stabilization, including (1) windrow, (2) aerated static pile, and (3) mechanical in-vessel systems. Although each composting design is unique, some of the processing steps common to all compost operations include

- Bulking agents are added to dewatered sludge for porosity and moisture control prior to initiation of composting treatment.
- Temperatures in the range of 130 to 150°F (55–65°C) are maintained for at least 48 hours to ensure destruction of pathogenic microorganisms.
- Compost is stored for extended periods of time (typically 4 to 6 weeks) after the primary composting reactions are complete to further stabilize the sludge at lower temperatures (a process called *curing*).

Successful compost stabilization of dewatered sludge depends on maintaining a suitable environment for microbial activity. Operational parameters important for process control include (1) moisture content, (2) oxygen concentration, (3) carbon-nitrogen ratio, (4) temperature, and

(5) pH. The importance of each of these parameters to the proper management of the composting process is described in the following sections.

3.2.10.1 Compost moisture content. The decomposition of organic matter during the composting process depends on maintenance of a suitable moisture content. The optimal moisture content for biodegradation of organic matter is approximately 50 to 70 percent (mass basis). Since dewatered sludge is often too wet to meet optimal composting conditions, dry bulking material (e.g., wood chips) is added to absorb moisture and to increase sludge porosity. The amount of bulking agent required is a function of the initial sludge moisture content. Sludges with a 15 to 25 percent solids content may require a 2:1 to 3:1 ratio of wood chips to sludge (volume basis) to attain the desired moisture content. Figure 3.75 illustrates the effect of wood chip addition on compost

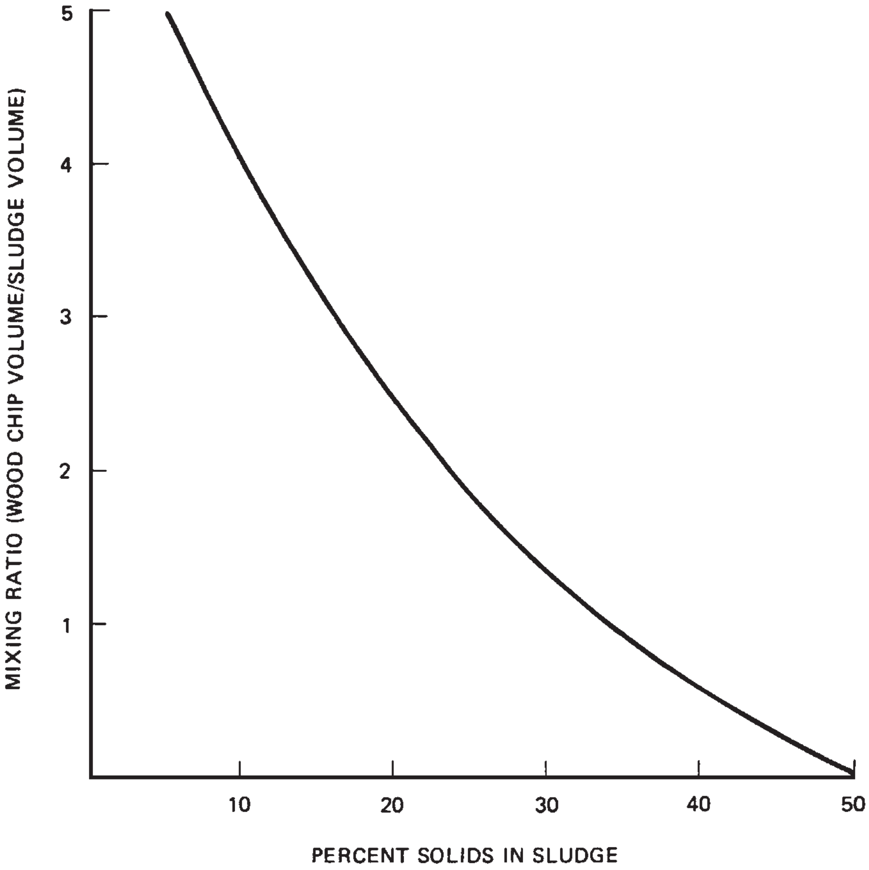


Figure 3.75 Effect of wood chips to sludge ratio on a composting operation.

Figure 3.76 A technician evaluating various compost mixtures using a laboratory scale compost system. (*Courtesy of Cache Environmental Laboratory, P.C.*)

operation. It is important to note that it is the ratio of wood chips to sludge that is the critical process design parameter and not the mass of wood chips alone.

In many cases, laboratory- and/or pilot-scale studies are employed to determine the optimal composting mixture. Figure 3.76 depicts a laboratory-scale compost system used to determine the optimal ratio of sludge, wood chips, and nutrient amendments.

Mixing of the dewatered sludge and bulking agent may be achieved using various means, including manual mixing, front-end loader, or specially designed mixing vessels. Figure 3.77 depicts a converted asphalt furnace that is presently being employed as a rotational mixing tank to prepare sludge for composting.

In addition to reducing the initial moisture content to the range necessary to initiate the composting process, a temperature range of an active compost system of 130 to 150°F (55–65°C) results in a significant volume of water being removed through evaporation [12]. To avoid desiccation of the compost, moisture is added as necessary to

Figure 3.77 Rotational mixer used to prepare sludge–bulking agent mixtures.

maintain proper compost operational conditions. Depending on the mode of compost operation, moisture can be added daily, weekly, or semimonthly [12,38].

3.2.10.2 Compost temperature. For efficient destruction of human pathogens, maintaining compost temperatures in the range of 130 to 150°F (55–65°C) for several days is necessary. The rise in compost temperature from ambient conditions is due to the enhanced microbial activity and the limited heat-transfer capacity of the compost material. Under proper conditions, active compost piles will achieve this temperature range within 12 to 24 hours and, under normal operating conditions, they can sustain these elevated temperatures for periods exceeding 1 to 2 weeks. The periodic mixing of compost will ensure that all the sludge will be exposed to this range of temperature. At completion of the composting process, the temperature will decrease, indicating a reduction in microbial activity.

The temperature in the compost pile can vary significantly depending on where measurements are taken. Several approaches available for establishing sludge compost temperature are outlined in the following references [11,12,38]. It should be noted that moisture content, aeration rates, size and shape of the compost pile, atmospheric conditions, and nutrient availability all affect the temperature distribution [11,12].

3.2.10.3 Compost pH. The optimal pH for compost operation is between 6 and 8, although the pH normally varies throughout the compost material [11]. The concern over compost pH stems from its effect on both microbial activity and nutrient availability. For example, a high initial compost pH resulting from the use of conditioning chemicals (e.g., lime) will solubilize nitrogen in the compost and contribute to the loss of nitrogen by ammonia volatilization [38].

3.2.10.4 Compost nutrient availability. Both carbon and nitrogen are required for effective compost operation. A biodegradable carbon-to-nitrogen (C/N) weight ratio of 25 to 35 has been found to provide optimal conditions for compost operations [38]. Lower C/N ratios increase the loss of nitrogen by leaching (e.g., nitrate mobilization) and ammonia volatilization, whereas higher levels necessitate progressively longer composting times as nitrogen becomes the microbial growth–limiting nutrient. Sludge normally has C/N ratios in the range of 10 to 20. To offset an imbalance in the C/N ratio, compost amendments usually are necessary. Typical compost amendments include material with high C/N ratios such as (1) sawdust, (2) rice hulls, (3) peanut shells, and (4) old compost [12]. Example 3.22 illustrates the approach for estimating the amount of nutrient amendments necessary for proper sludge composting.

Example 3.22 Poole County Wastewater Treatment Plant desires to employ windrow composting to stabilize its dewatered sludge. As the design engineer, you have proposed mixing sawdust (C/N ratio of 200) available from a local lumber yard with the dewatered sludge (C/N ratio of 12) so that the final compost mixture will have a C/N ratio of 25. Estimate the pounds of sawdust required per day as compost amendment if the plant generates 1500 lb of dewatered sludge (dry-weight basis). Assume that the following data apply:

- Moisture content of sludge: 70 percent
- Moisture content of sawdust: 40 percent
- Nitrogen content of sludge: 5 percent (dry-weight basis)
- Nitrogen content of sawdust: 0.4 percent (dry-weight basis)

solution

Step 1. Determine the pounds of nitrogen and carbon in 1 lb of sludge and 1 lb of sawdust.

One pound of sludge:

$$\text{Mass of water} = 1\text{ lb}\,(0.70) = 0.7\text{ lb}$$

$$\text{Dry weight} = 1.0 - 0.70 = 0.3\text{ lb}$$

$$\text{lb nitrogen} = (0.3\text{ lb})\,(0.05) = 0.015\text{ lb N}$$

$$\text{lb carbon} = (0.015\text{ lb N})\,(12\text{ lb C/lb N}) = 0.18\text{ lb C}$$

One pound of sawdust:

$$\text{Mass of water} = 1\text{ lb}\,(0.40) = 0.4\text{ lb}$$

$$\text{Dry weight} = 1.0 - 0.40 = 0.6\text{ lb}$$

$$\text{lb nitrogen} = (0.6 \text{ lb})\,(0.004) = 0.0024 \text{ lb N}$$

$$\text{lb carbon} = (0.0024 \text{ lb N }(200 \text{ lb C/lb N}) = 0.48 \text{ lb C}$$

Step 2. Estimate the mass of sludge X that must be added to 1 lb of sawdust to achieve a C/N ratio of 25:

$$\frac{\text{Carbon}}{\text{Nitrogen}} = \frac{25}{1} = \frac{\text{lb carbon/lb sawdust} + X\,(\text{lb carbon/lb biosolids})}{\text{lb nitrogen/lb sawdust} + X\,(\text{lb nitrogen/lb biosolids})}$$

$$= \frac{0.48 + X0.18}{0.0024 + X0.015}$$

or

$$X = \frac{2.15 \text{ lb sludge}}{1.0 \text{ lb sawdust}}$$

Step 3. Estimate the daily sawdust addition rate (lb/day):

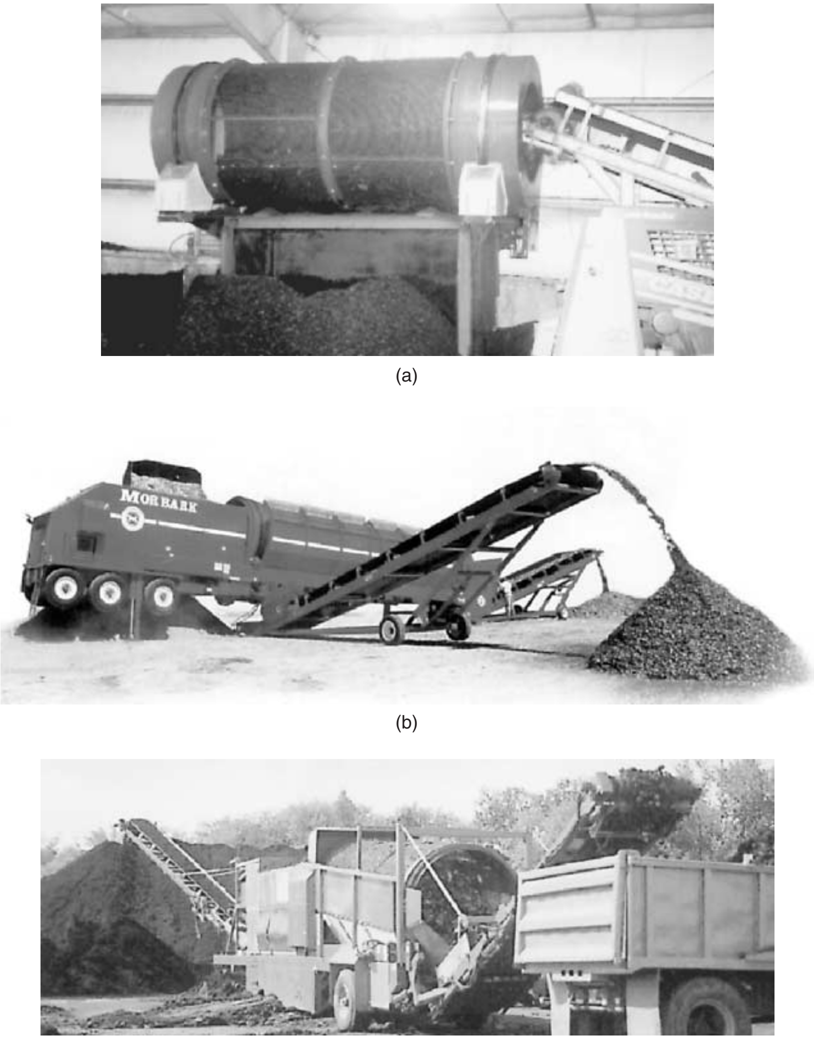
$$\frac{\text{lb sawdust}}{\text{day}} = \frac{1500 \text{ lb sludge}}{\text{day}} \cdot \frac{1.0 \text{ lb sawdust}}{2.15 \text{ lb sludge}}$$

$$= 696 \text{ lb sawdust/day (dry weight)}$$

3.2.10.5 Compost oxygen concentration. Optimal oxygen concentrations in a composting system range from 5 to 15 percent (volume basis). Increasing the oxygen content beyond 15 percent by adding air normally will result in a temperature decrease because of greater convective heat losses [12]. Although oxygen concentrations as low as 0.5 percent have been observed in some systems, a 5 percent oxygen concentration in the air phase is required to prevent the formation of anaerobic pockets within the compost pile [12,38].

Since dewatered sludge has a relatively low porosity, the addition of bulking agents ensures adequate oxygen transfer during composting. Typical bulking agents include wood chips and chipped automobile tires [38]. To reduce operational costs, bulking agents normally are recycled by screening the finished compost using a trommel screen or equivalent device (Fig. 3.78).

3.2.10.6 Composting operation designs. In general, three types of composting operation designs presently are employed in sludge stabilization. They include (1) the windrow process, (2) the aerated static pile, and (3) the mechanical in-vessel systems. Each design varies in operational complexity and therefore has unique advantages and



(a)

(b)

(c)

Figure 3.78 (*a*) A small trommel screen used to recycle wood chips. (*b*) A large trommel screen used for bulking-agent recycling. (*Courtesy of Morbark Inc.*) (*c*) A large trommel screen used for bulking-agent recycling. (*Courtesy of McCloskey Bros. Mfg.*)

disadvantages. The final decision about which design to employ depends on available land area, purchase price, operational costs, and maintenance costs [11].

3.2.10.7 Windrow process. In the windrow composting process, the compost mixture (i.e., dewatered sludge, bulking agent, and compost amendments) is stacked in long parallel rows of triangular or trape-

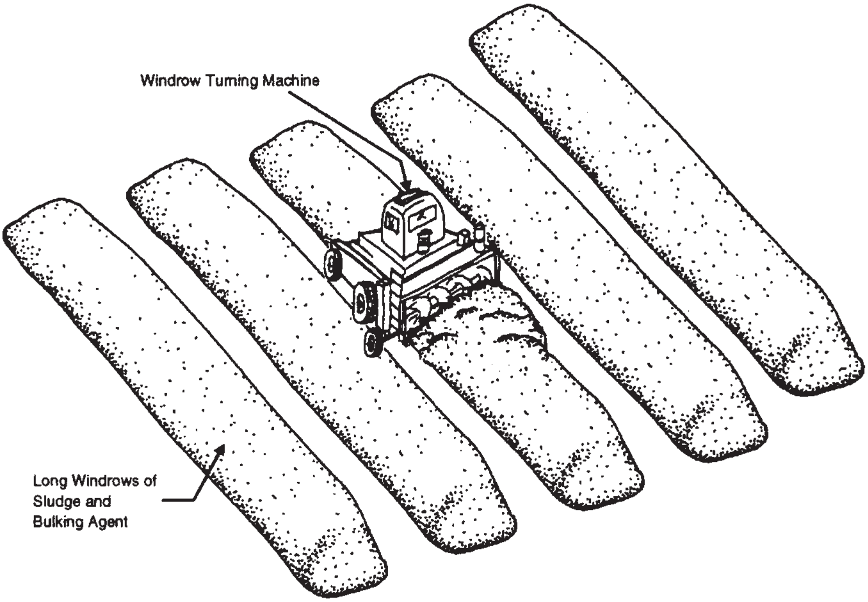


Figure 3.79 Diagram of a windrow composting system.

zoidal piles called *windrows* (Fig. 3.79). The choice of configuration depends largely on the characteristics of the equipment used for mixing and turning the piles. The width of a typical windrow is 15 ft (4.6 m), and the height normally ranges from 3 to 7 ft (0.91–0.21 m). It is important to note that the specific volume (volume/surface area) of the pile will influence the moisture evaporation rate and therefore the operational costs. In general, the larger the windrow, the slower the overall moisture evaporation rate.

The windrow process normally is conducted in uncovered areas. This process relies on natural ventilation or the chimney effect (i.e., convection of warmer air leaving the top of the windrow with cooler air entering the windrow from lower sections), with periodic mechanical mixing of the piles to maintain aerobic conditions. In areas of significant rainfall, it may be desirable to provide a roofed structure. Although the temperature of active compost windrows normally will be in the range of 130 to 150°F (55–65°C), during wet periods or winter conditions, maximum temperatures may only reach as high as 130°F (55°C). In these circumstances, the treatment process may be modified so that each windrow undergoes a longer period of treatment (e.g., 8 to 10 weeks). In all cases, leachate from composting systems must be either treated on site or returned to the wastewater treatment plant (see Chap. 5).

Under normal operations, the windrow is turned once or twice a week. After turning the pile, moisture is added to moisten the compost material. The turning and moisture addition rapidly reduce the compost temperature. However, for an active compost windrow, the temperature normally recovers to the 130 to 150°F (55–65°C) range within 4 to 8 hours.

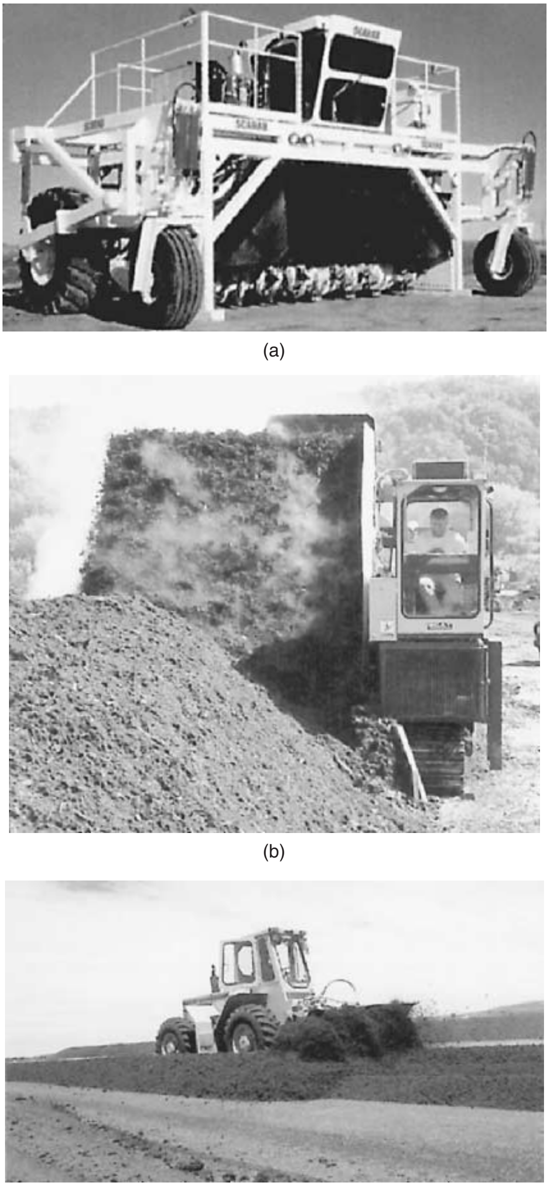
Depending on the size of the compost pile, 4 to 8 weeks is adequate for the first stage or high-rate phase of the compost process to be completed. Turning of the windrows may be accomplished by use of a front-end loader. Alternatively, specially designed windrow turners are available commercially (Fig. 3.80).

A satisfactory degree of stabilization is indicated by a gradual decline in the overall compost temperature to approximately 110°F (43°C). To provide an additional margin of safety for pathogen control, sludge that has completed the first stage is then transferred to a curing area, where the compost material is allowed to undergo a reduced rate of stabilization treatment for an additional 2 to 4 weeks. After curing, the compost is evaluated for pathogen levels, and if the pathogen levels are acceptable, the material is processed to remove the bulking agent.

Because of their simplicity, windrow composting systems normally are the least expensive operational design. However, use of windrow composting requires that suitable space be available to maintain several active windrows and a sizable curing area. If land area is limited, the aerated static pile process can achieve satisfactory sludge stabilization using a fraction of the space of that required for windrow composting.

3.2.10.8 Aerated static pile process. Aerated static pile (sometimes referred to as *forced aeration windrows*) is a more complex composting approach to stabilizing sludge than windrow composting. However, this method is recommended if space is limited and/or the desired stabilization processing time is less than 8 weeks. In addition to a shorter processing time, an aerated static pile system provides more flexible operation and control over oxygen and temperature conditions than the windrow system. Figure 3.81 is a schematic diagram of an aerated static pile system.

In the aerated static pile composting process, piles or windrows of compost material (i.e., dewatered sludge, bulking agents, and nutrient amendments) are placed on loops of perforated plastic pipe (4–6 in in diameter) on the composting pad. To facilitate aeration, a 6- to 8-in layer of bulking agent typically is placed over the pipes to maintain an even distribution of influent air during compost operation. Once the pile is formed, it is covered with at least 12 in of cured and screened compost or 18 in of unscreened compost. This outer layer is critical for


(a)

(b)

(c)

Figure 3.80 Mechanical equipment used to turn windrows: (*a*) *Courtesy of SCARAB Manufacturing and Leasing, Inc.*; (*b*) *Courtesy of SCAT Engineering, Inc.*; (*c*) *Courtesy of BROWN BEAR Corporation.*

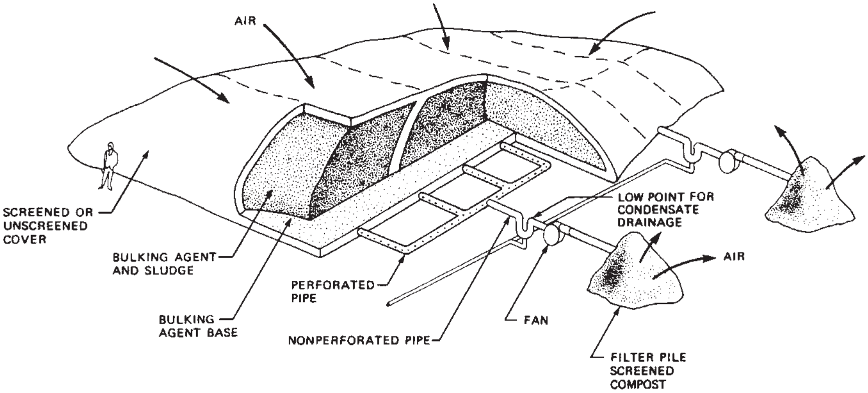


Figure 3.81 Schematic diagram of an aerated static pile system.

successful operation because it provides thermal insulation and minimizes escape of odors during sludge composting.

Air can be supplied to the aerated static pile through a suction system or a positive-pressure system. The suction system draws air into and through the pile. The air then travels through a central pipe and is vented through a pile of finished compost that acts as an odor filter (Fig. 3.82). The ability to contain odors is an important advantage of suction aeration. However, the suction of air through the odor filter significantly increases the pressure losses, requiring a larger fan or blower. A typical aeration rate for the suction-type aeration pile is typically 14 m^3/h (8 ft^3/min) per ton of sludge dry solids. Finally, in the suction mode of operation, condensate from water vapor drawn from the compost pile must be removed before the air reaches the blower.

In the positive-pressure system, a blower is used to push air into the actively composting pile. The air travels through the pile and is vented over the entire pile surface. Since the air is vented over the entire surface, odor control is difficult. However, because of the absence of an odor filter, there is less pressure loss, and consequently, a larger amount of air can be moved through the pile for the same blower power as compared with the suction system. For positive-pressure aeration systems, a typical aeration rate of 500 ft^3/h per ton dry sludge (15.6 $m^3/\text{ton} \cdot \text{h}$) normally maintains aerobic conditions within the pile [38]. In addition, positive-pressure systems can be more effective at cooling a compost pile and, therefore, are the preferred design when warm temperatures are a concern [38]. However, positive-pressure aeration systems, if not controlled carefully, can result in desiccation of the lower part of the pile and reduction in pathogen removal efficiency.

To ensure that the compost process proceeds at a high rate, the temperature, moisture, and oxygen levels must be monitored closely. Since

Figure 3.82 Odor control piles for aerated static pile compost system.

the aeration rate will affect the temperature, moisture, and oxygen levels, blower operation is the critical operational variable in aerated static piles. Blowers can be operated continuously or intermittently. Continuous operation of the blower permits a lower airflow rate to achieve satisfactory results because oxygen addition and heat removal occur constantly. However, continuous blower operation leads to less uniform pile temperatures. Intermittent blower operation is preferable but requires a programmable timer or temperature feedback system. Aeration cycles of 20 to 30 minutes with the fan operating at one-tenth to one-half the cycle have proven satisfactory [11]. While the fan is not operating, the natural convective movement of warm air (i.e., chimney effect) typical of windrows takes place. In the absence of forced aeration, the chimney effect causes warming of the outer edges, which further reduces pathogens. The direction of airflow can be reversed during the latter stages of active composting to elevate the pile temperature above the required 55°C (130°F). With a temperature feedback system, temperature sensors (e.g., thermocouples or thermistors) switch the blower on or off when the temperature exceeds or falls below a predetermined temperature setpoint [38].

The minimum processing time and temperature requirements of either a windrow or static pile composting system depend on the desired level of pathogen reduction. If "significant" pathogen reduction is acceptable (i.e., Class B biosolids are to be generated), then the minimum temperature/time requirement for effective treatment is 5 days at 105°F (40°C) with 4 hours at 130°F (55°C) or higher for either system. If "further" pathogen reduction is desired (i.e., Class A biosolids are to be generated), the minimum time/temperature requirement for effective treatment is 3 days at 130°F (55°C) for the aerated static pile system or 15 days at 130°F (55°C) with five turnings for the windrow process. In addition to meeting the minimum temperature/time requirements, the total active composting time must be at least 21 days, with an additional 21 days of curing.

The pad area for either windrow or aerated static pile composting should be paved. Although concrete has been the most successful paving material, asphalt may be suitable. It should be noted that asphalt may soften at higher composting temperatures and may be susceptible to composting reactions. The minimum pad area for a composting operation may be estimated using Eq. (3.49):

$$\text{Area} = \frac{1.1S\,(R + 1)}{H} \tag{3.49}$$

where A = pad area for active compost piles, ft^2 (m^2)
S = total volume of sludge produced in 4 weeks, ft^3 (m^3)
R = ratio of volume of bulking agent (and amendments) to sludge volume
H = height of pile, ft (m)

For an aerated static pile system employing an odor-control filter, the pile should add an additional 10 percent to the area calculated using Eq. (3.49). It should be noted that Eq. (3.49) assumes a 21-day composting period but provides an additional 7 days of capacity to allow for low temperature, excess precipitation, and malfunctions. Moreover, in many locations, the finished product from aerated static pile compost operations may still be moist, so additional drying may be necessary. The processing area for this drying and screening to separate the bulking agent typically is equal to the size of the active composting area.

An area capable of accommodating 30 days of compost production is recommended as the minimum for curing operations. Additional storage area may be necessary depending on the end use of the compost. Winter storage may be required if the finished compost is used only during the growing season.

Finally, access roads, turnaround space, and a wash rack for vehicles are all required. If runoff from the site and leachate from the aeration system cannot be returned to the sewage treatment plant, then a runoff collection pond must be included. Most composting operations also have a buffer zone around the site for odor control and visual aesthetics. The size of the buffer zone will depend on the local conditions and regulatory requirements. Example 3.23 illustrates the standard approach for estimating the minimum required composting area.

Example 3.23 The Maralal City Solid Waste Recovery Facility is planning to expand its composting operations to receive 70 tons of wet wastewater treatment sludge per day. Preliminary tests have shown that the volumetric ratio of bulking agent (e.g., woodchips) to sludge necessary for adequate composting using a 6-ft aerated static pile system is 2:1. If the wastewater sludge has a solids content of 20 percent and a bulk density of 1600 lb/yd^3, estimate the minimum size for the entire composting operation. Assume that the composting curing area is equivalent to the size of the composting area and that the area needed for access roads, turnarounds, runoff collection, etc. is approximately 40 percent of the total area.

solution

Step 1. Estimate the volume of sludge produced over a 4-week period (1 month):

Monthly production rate of sludge (ft^3/month)

$$= 70 \text{ tons/day} \cdot 2000 \text{ lb/ton} \cdot \text{yd}^3/1600 \text{ lb} \cdot 27 \text{ ft}^3/\text{yd}^3 \cdot 7 \text{ days/week} \cdot 4 \text{ weeks/month}$$

$$= 66{,}150 \text{ ft}^3/\text{month}$$

Step 2. Using Eq. (3.49), estimate the minimum area necessary for composting:

$$\text{Area (ft}^2) = \frac{1.1\, S\, (R + 1)}{H}$$

$$= \frac{1.1 \cdot 66{,}150 \text{ ft}^3 \cdot (2 + 1)}{6 \text{ ft}}$$

$$= 36{,}382.5 \text{ ft}^2 \text{ (0.84 acre)}$$

Step 3. Estimate the total area necessary for composting operations accounting for the curing area and ancillary operations:

$$\text{Composting area plus curing area} = 2 \cdot 0.84 = 1.68 \text{ acres}$$

$$\text{Total area for composing operations} = 1.68 \text{ acres} + 0.4\,(1.68 \text{ acres}) = 2.35 \text{ acres}$$

For situations where available land is extremely limited and there is a need to stabilize sludge at a high rate, mechanical in-vessel composing systems offer a technically feasible alternative.

3.2.10.9 Mechanical in-vessel composting systems. Mechanical in-vessel composting is a highly automated method of stabilizing sludge within an enclosed container or reactor (Fig. 3.83). Mechanical in-vessel

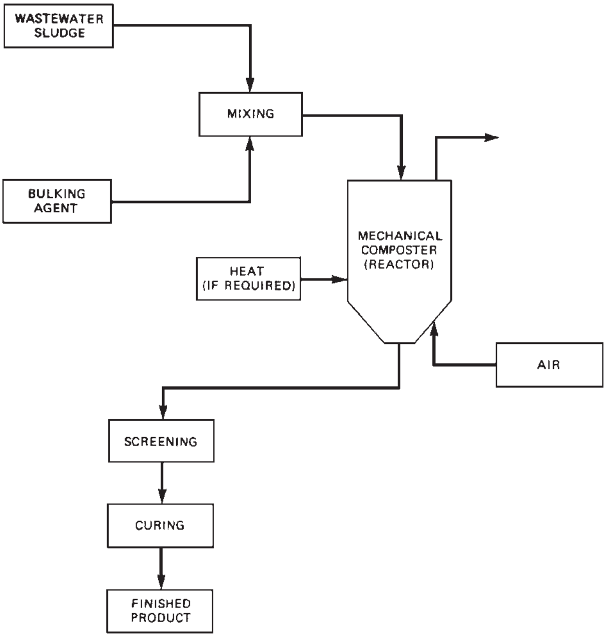


Figure 3.83 Schematic diagram of a mechanical composting system.

composting systems are designed to minimize odors and processing time by controlling environmental conditions such as airflow, temperature, and oxygen concentrations [49]. A mechanical in-vessel sludge composting system is warranted if any of the following conditions exists: (1) the complete stabilization process must occur rapidly (less than 4 weeks), (2) odor and/or leachate control is a priority, (3) space is limited, and (4) sufficient resources are available.

Mechanical in-vessel composting systems normally fall into one of two categories: (1) rotating-drum or (2) tank systems [38,49]. Rotating-drum systems rely on a tumbling action to continuously mix the compost mixture (sludge, bulking agent, and nutrient amendments). The drums typically are long cylinders approximately 9 ft in diameter that are rotated slowly (usually at less than 10 rev/min) [61]. Figure 3.84 illustrates a typical rotating-drum composting system. Oxygen is introduced into the rotating-drum system through nozzles connected to a blower or fan. The tumbling of the compost material allows oxygen to be maintained at high and relatively uniform levels throughout the drum.

Mechanical in-vessel tank systems are available in either horizontal or vertical varieties. Rectangular agitated-bed compost systems are one type of horizontal system (Fig. 3.85). These tanks are long containers in

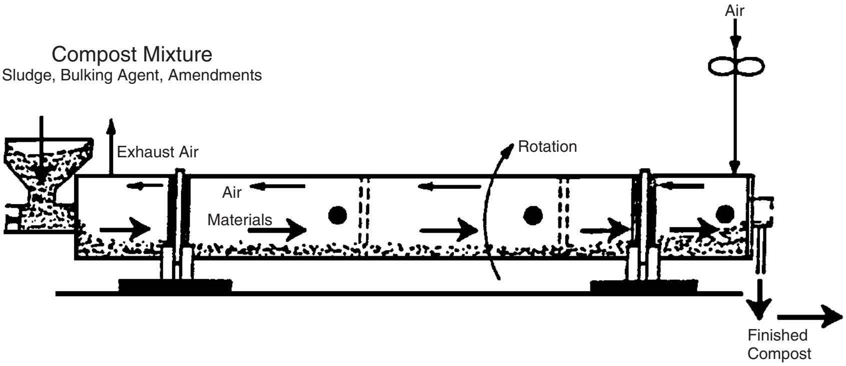


Figure 3.84 Rotating-drum compost system.

which aeration is accomplished through the use of an external blower that forces air through the perforated bottom of the tank. Mixing of the compost material is achieved by mechanically passing a moving belt, paddle wheel, or flail-covered drum through the composting material. This action agitates the compost material, breaks up clumps of particles, and maintains compost porosity [61]. Suggested composting periods for continuously agitated bed systems range from 2 to 4 weeks.

Vertical tank in-vessel composting systems consist of one or more vertical tanks in which air is introduced through blowers. Vertical tank systems can use conveyors, rotating screws, air feeds, etc. to mechanically mix the compost material and to supply oxygen and moisture. Figure 3.86 provides schematic diagrams of several types of vertical tank systems.

Under most conditions, the brief detention times that equipment manufacturers specify for mechanical in-vessel composting (i.e., 10–15 days) does not allow adequate stabilization of sludge. Normally, a 4- to 6-week maturation period (i.e., curing) is necessary to reduce the remaining volatile matter. Therefore, the amount of time and total land area required for mechanical composting often approach those necessary for aerated static pile systems.

3.2.10.10 Compost curing. Regardless of the type of compost system employed for sludge stabilization, once the compost has undergone the high-rate decomposition process, it must be cured. During the curing stage, compost is further stabilized as any remaining pathogens are destroyed (Fig. 3.87). Moreover, additional organic matter is metabolized, reducing the volatile solids content [38,45].

To begin the curing process, finished compost material is collected and formed into piles or windrows and left unattended until the curing

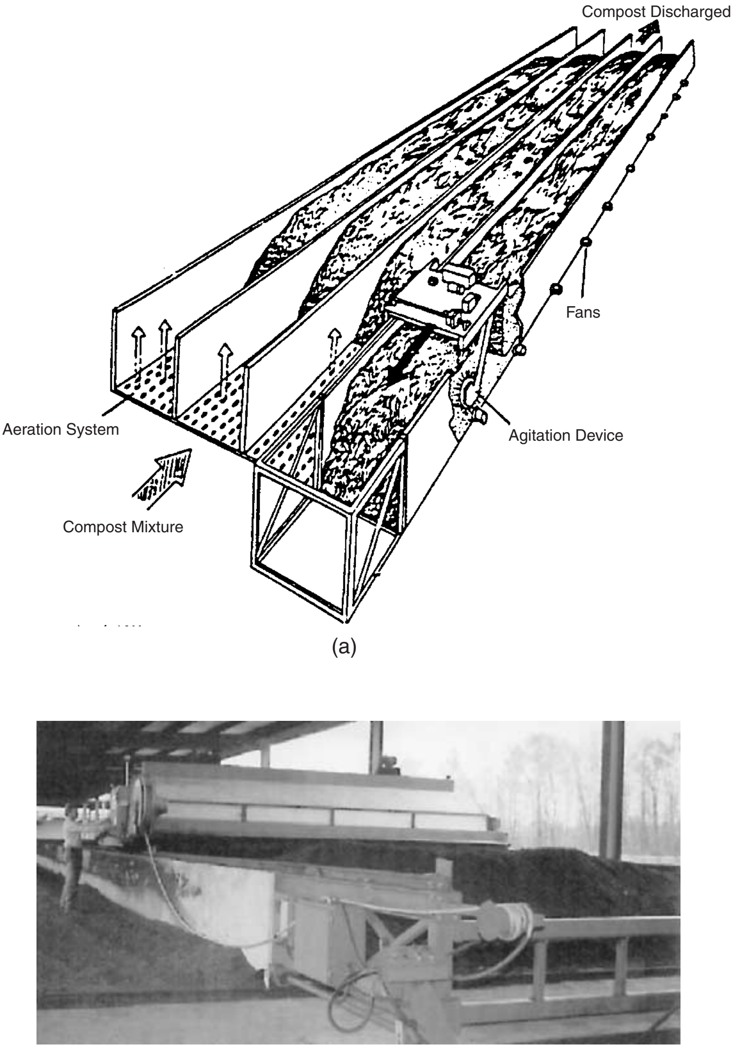
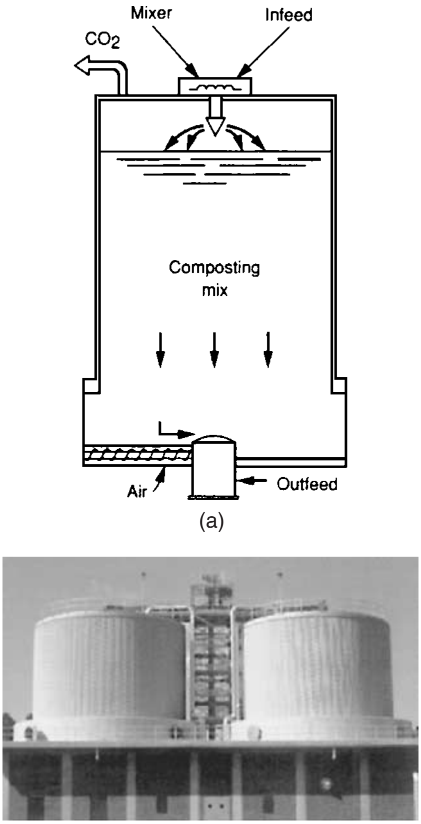


Figure 3.85 (*a*) Schematic diagram of a rectangular agitated-bed compost system. (*b*) Photograph of a rectangular agitated-bed compost system. (*Courtesy of US Filter/Davis Process.*)

period has passed (not less than 4 weeks). Since the curing piles undergo slow decomposition, curing piles should be made small enough (*ca.* 4 to 6 ft in height) to permit adequate diffusion of atmospheric oxygen and to prevent the onset of anaerobic conditions.

Another important characteristic of cured compost is its carbon-to-nitrogen (C/N) ratio. Properly cured compost will have a C/N ratio that



(a)

(b)

Figure 3.86 (*a*) Schematic diagram of a vertical tank mechanical compost system. (*b*) Photograph of a dual-silo mechanical biosolids composting system. (*Courtesy of US Filter/Davis Process.*)

Figure 3.87 Photograph of curing piles.

ranges from 20:1 to approximately 30:1. If the ratio is too high (i.e., 40:1 or greater), land application of the compost may stimulate immobilization of soil nitrogen, causing a reduction in crop yield. On the other hand, if the C/N ratio is too low (i.e., 15:1 or lower), ammonia or other nitrogenous compounds will be released in sufficient amounts to "burn" crop roots and inhibit plant growth [38].

3.2.10.11 Estimation of composting mixing ratios. The basic criteria for successful composting are that the compost mixture be porous and structurally stable. Moreover, the compost mixture must contain sufficient degradable organic material that the compost reactions are self-sustaining (i.e., require no supplemental energy input). In general, the initial compost mixture has been found to be capable of sustaining itself under the following conditions:

$$W = \frac{\text{mass of water in initial compost mixture (lb, kg)}}{\begin{array}{c}\text{mass of biodegradable organic material}\\ \text{in compost mixture (lb, kg)}\end{array}} \leq 10 \qquad (3.50)$$

If the ratio W is less than 10, the initial compost mixture has sufficient energy available for temperature elevation and water evaporation [11]. Having a value for the W ratio of less than 10 does not necessarily guarantee a successful compost process since climatic conditions affect the thermodynamic energy requirements. In hot, arid climates, an appropriate value for W may be higher because evaporation of water from the compost is increased. In cold climates, more biological energy is required to heat the compost and, therefore, an appropriate value for W may have to be as low as 7 [12]. Compost mixtures can be adjusted by adding amendments that contain high concentrations of degradable organic material (e.g., wood chips). These materials are usually dry and can reduce the ratio W by both increasing the organic matter content and decreasing the moisture fraction of the mixture.

Figure 3.88 illustrates a generalized mass-balance approach for modeling a basic composting operation. From Fig. 3.88 a set of mass-balance equations can be develop for designing compost mixtures. Equation (3.51) is used to compute the quantity of recycled materials X_R, whereas Eq. (3.52) is used to estimate the ratio W.

$$X_R = \frac{X_C\,(S_M - S_C) + X_A\,(S_M - S_A) + X_B\,(S_M - S_B)}{S_R - S_M} \qquad (3.51)$$

$$W = \frac{X_C\,(1 - S_C) + X_A\,(1 - S_A) + X_B\,(1 - S_B) + X_R\,(1 - S_R)}{X_C S_C V_C K_C + X_A S_A V_A K_A + X_B S_B V_B K_B + X_R S_R V_R K_R} \qquad (3.52)$$

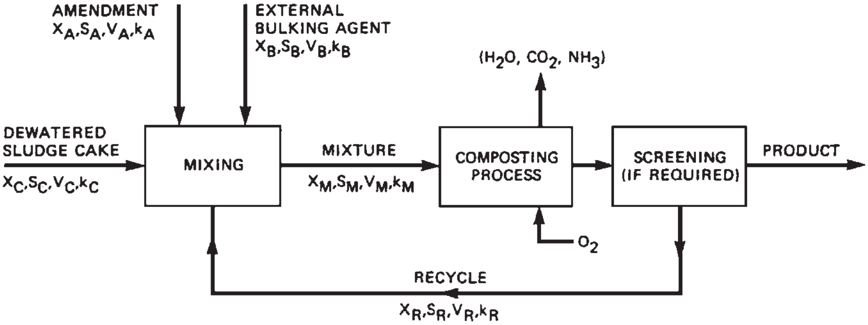


Figure 3.88 Mass-balance model for compost mixture design. Process variables and range of average values (in parentheses): X_C = total wet weight of sludge cake produced per day; X_A = total wet weight of amendment per day; X_R = total wet weight of recycle per day; X_B = total wet weight of external bulking agent per day; X_M = total wet weight of mixture per day; S_C = fractional solids content of sludge cake (0.20 to 0.55); S_A = fractional solids content of amendment (0.50 to 0.95); S_R = fractional solids content of recycle (0.60 to 0.75); S_B = fractional solids content of external bulking agent (0.50 to 0.85); S_M = fractional solids content of mixture (0.40 to 0.50); V_C = volatile solids content of sludge cake, fraction of dry solids (0.40 to 0.60, digested; 0.60 to 0.80, raw); V_A = volatile solids content of amendment, fraction of dry solids (0.80 to 0.95); V_R = volatile solids content of recycle, fraction of dry solids (0.00 to 0.90); V_B = volatile solids content of external bulking agent, fraction of dry solids (0.55 to 0.90); V_M = volatile solids content of mixture, fraction of dry solids (0.40 to 0.80); k_C = fraction of sludge cake volatile solids degradable under composting conditions (0.33 to 0.56); k_A = fraction of amendment volatile solids degradable under composting conditions (0.40 to 0.60); k_R = fraction of recycle volatile solids degradable under composting conditions (0.00 to 0.20); k_B = fraction of external bulking agent volatile solids degradable under composting conditions (0.00 to 0.40); k_M = fraction of mixture volatile solids degradable under composting conditions (0.20 to 0.60).

All terms are defined in Fig. 3.88.

When initially designing the compost mixture, Eq. (3.51) should be solved, assuming that no amendment or bulking agent is added and that the mixture solids content S_M is 50 percent (0.5). Once X_R is calculated, estimate the ratio *W*. If *W* is less than 10, no additives for moisture control are required (compost amendment additions may be necessary to maintain a proper carbon-nitrogen ratio). If the ratio *W* is greater than 10, bulking agents and/or amendments should be added in sufficient quantities until the *W* requirement is satisfied.

3.2.10.12 Compost facility siting. Proper siting and design are prerequisites to establishing a safe and effective sludge composting operation. Throughout the siting and design process, local and state requirements should be reviewed. Many states have established specific environmental and safety criteria that composting facilities must address during siting and design such as water quality, leachate control, air quality, run-on/ponding, vectors, fire potential, noise, etc. [38]. Many of the regulatory criteria address technical concerns, including those related to protecting human health and the environment, and can have an impact on facility location, land use, size, and other considerations. In general, detailed engineering plans must be submitted and approved by the regulatory authority in order to construct a sludge compost facility.

3.3 Conditioning

In many cases, stabilized sludge may be applied directly to land. If it is desired to land apply stabilized sludge in liquid form, sludge from anaerobic digestion, aerobic digestion, lime stabilization, chlorine oxidation, or pasteurization may be transported to the land-application site via a pipeline or tank truck. If the sludge is to be applied to land in solid form or if the land-application site is located a considerable distance from the wastewater treatment plant and it is desired to reduce transportation costs, it may be necessary to dewater the sludge. The product of the dewatering process is a sludge that has the mechanical properties of a solid. To facilitate sludge dewatering, the sludge must be conditioned.

Conditioning is the treatment of sludge to facilitate water removal. Conditioning prior to dewatering involves the chemical and/or physical treatment of sludge to enhance water removal and to improve solids capture. Most sludge conditioning systems employ inorganic chemicals, organic polymers, or heat. It should be noted that since conditioning always has an effect on the efficiency of the dewatering process that follows, any evaluation of the conditioning process must take into account the capital, operating, and maintenance costs for the entire conditioning/dewatering system. These costs must include the impact of sidestreams on other plant processes, the plant effluent, and the resulting air quality [38,80].

A primary objective of chemical conditioning is to increase the sludge particle size by combining smaller particles into large aggregates. Chemical conditioning is a two-step process consisting of coagulation and flocculation. *Coagulation* involves destabilization of the sludge particle by decreasing the magnitude of the repulsive electrostatic forces between particles, whereas *flocculation* is the agglomeration of colloidal suspended matter by gentle mixing.

If the flocculated sludge is subjected to physical stress, floc shearing can occur. Therefore, mixing should provide just enough energy to disperse the conditioning chemical throughout the sludge and bring the particles and colloidal suspension together. Consideration should be given to providing individual conditioning systems for each dewatering unit, since it is neither economical nor good practice to provide one common conditioning unit for several dewatering units.

Many factors influence the dewatering and conditioning requirements of sludge. Raw municipal wastewater contains a significant quantity of colloids and fines, most of which pass through the primary treatment system but are effectively captured in the biological or secondary treatment unit operations. As a result, biological sludges, especially waste-activated sludges, are difficult to dewater and have a high demand for conditioning chemicals. Some of the important factors affecting conditioning operations include

- Particle surface charge and hydration
- Particle size
- Compressibility
- Sludge temperature
- Ratio of volatile solids to fixed solids
- Sludge pH

Each of these factors is reviewed briefly in the following sections.

3.3.1 Particle surface charge and hydration

Normally, sludge solids have a negative surface charge and therefore repel one another. This repulsive force increases exponentially as the particles are forced closer together [81]. In addition to electrostatic repulsion, sludge particles retain water molecules on their surface either by adsorption or by capillary action. Although loosely held at the particle surface, the attraction of water to the particle surface interferes with the dewatering process.

Conditioning chemicals (e.g., lime, ferric chloride, polymers, etc.) are used to overcome the effects of both sludge surface charge and hydration. Generally, these chemicals act by reducing or eliminating the repulsive force, thus permitting the particles to come together (i.e., flocculate), which facilitates water removal in subsequent dewatering steps.

3.3.2 Particle size

Particle size generally is recognized as the most important factor influencing sludge dewaterability and therefore conditioning requirements. As the average particle size decreases, the surface area for a given sludge mass increases. The effects of increasing the specific surface area of sludge solids include

- Greater electrical repulsion between particles due to larger negatively charged area
- Greater frictional resistance to water movement
- Greater attraction to water due to more surface adsorption sites

Sludge particle size is influenced by sludge source and its prior treatment. For example, since fine and colloidal particles tend to pass through the primary clarifier, primary sludge generally has a larger particle size than secondary sludge. The fine particles generally are removed in the secondary clarifier together with the less-dense flocculated secondary sludge. As a result, primary sludge typically can be dewatered more effectively than can secondary sludge [48].

Anaerobic and aerobic stabilization of primary and secondary sludge also decreases the average particle size. This is the principal reason that digested sludge is more difficult to dewater than undigested sludge. Other operations that can result in decreased sludge particle size include (1) sludge mixing, (2) sludge storage, and (3) sludge pipe transport [38,48].

3.3.3 Compressibility

The compressibility of sludge results in particle deformation and a reduction in void volume between particles. The reduction in void volume inhibits the movement of water through the compressed portion of sludge (i.e., lower dewatering efficiency). Proper conditioning produces a flocculent sludge matrix that maintains a substantial porosity during the dewatering process. Therefore, the net result of conditioning is a more rapid removal of water [38,48].

To improve the filtration process, filter aids sometimes are employed in mechanical dewatering systems. Filter aids are materials used to increase the removal rate of water (i.e., filtration rate) by lowering the compressibility of the sludge by physical means. Filter aids include such materials as (1) diatomaceous earth, (2) perlite, or (3) carbon. Filter aids typically are added in fixed amounts to the thickened sludge prior to pumping the material to the dewatering system [38,48].

3.3.4 Sludge temperature

As sludge temperature increases, the overall viscosity of the sludge decreases due to the effect of temperature on water viscosity (Table 3.20). The viscosity of sludge is particularly important in centrifugation because the rate of solids separation increases as the viscosity decreases.

It should be noted that dewatering processes that use filtration are unaffected by sludge temperature.

3.3.5 Ratio of volatile solids to fixed solids (VS/FS)

Sludge tends to dewater more effectively as the percentage of fixed solids increases relative to volatile solids, assuming that all other factors are equal [48,73]. The increase in volatile solids results in larger adsorption of water, which reduces the pore space and, consequently, the ability of the sludge to transmit water.

TABLE 3.20 Viscosity of Water as a Function of Temperature*

°C	°F	Viscosity (cP)†
10	50	1.308
15	59	1.140
20	68	1.005
25	77	0.894
30	86	0.800
35	95	0.723

*Adapted from ref. [38].
†cP = centipoise (0.001 Pa · s).

3.3.6 Sludge pH

Sludge pH affects the surface charge on the sludge particles as well as the type of polymer that may be used in conditioning [38]. Generally, anionic (i.e., negatively charged) polymers are most useful when the sludge is first lime conditioned and has a high initial pH (i.e., above 8.0). Cationic (i.e., positively charged) polymers are most suitable at sludge pH values slightly above or below neutrality (i.e., a pH of 7.0).

3.3.7 Chemicals used in sludge conditioning

The predominant chemicals used in sludge conditioning processes include: lime, ferric chloride, and organic polymers. It should be noted that ferrous sulfate and aluminum sulfate also have been used, but sparingly. These chemicals destabilize the surface charge on the sludge particles, resulting in a flocculent matrix that is more easily dewatered than the discrete particles [38,73].

When comparing the various conditioning chemicals, a number of factors must be evaluated in addition to performance and costs. These include (1) the volume and weight changes that result from the use of conditioning chemicals, (2) storage and handling of conditioning chemicals, (3) conditioning chemical availability, and (4) maintenance of the subsequent dewatering equipment as a result of the conditioning chemical(s) used. Table 3.21 identifies the most common conditioning chemicals used as a function of the type of dewatering equipment employed, whereas Table 3.22 presents general dosages of conditioning chemicals used in each dewatering process.

3.3.7.1 Ferric chloride. Ferric chloride addition to thickened sludge results in the formation of positively charged iron complexes that neutralize the negatively charged sludge particles. Ferric chloride also reacts with existing alkalinity, resulting in the production of ferric hydroxide [$Fe(OH)_3$], which acts to flocculate the destabilized sludge

TABLE 3.21 Chemicals Typically Used for Sludge Conditioning*

Dewatering process	Lime†	Ferric chloride	Polymer
Centrifugation			C
Belt filter press			C
Vacuum filter	C	C	C
Recessed filter press	C	C	P
Drying bed		P	

NOTE: C, commonly used; P, possibly used but not common.
*Adapted from ref. [38].
†Lime and ferric chloride typically are used together.

TABLE 3.22 Typical Chemical Dosages Used for Sludge Conditioning*

Dewatering process	Primary sludge (lb/ton)†	Primary plus secondary sludge (lb/ton)	Anaerobically digested sludge (lb/ton)
Centrifugation (polymer)	0–5	1–10	2–10
Belt filter press (polymer)	4–8	4–10	8–15
Vacuum filter			
Polymer	4–10	6–12	—
Lime‡	160–200	180–320	300–420
Ferric chloride	4–80	50–120	60–120
Recessed plate filter press			
Lime‡	220–280	220–320	220–600
Ferric chloride	80–120	80–140	80–200

*Adapted from ref. [38].
†lb/ton = lb of conditioning chemical per ton sludge (dry-mass basis).
‡Lime and ferric chloride typically are used together.

particles [Eq. (3.53)]. Ferric chloride is used most commonly in conjunction with lime for conditioning sludge prior to dewatering using either vacuum filtration or a recessed-plate filter press [38,49].

$$2FeCl_3 + 3Ca\,(HCO_3)_2 \rightarrow 2Fe\,(OH)_3 + 3CaCl_2 + 6CO_2 \quad (3.53)$$

Ferric chloride may be purchased as either a liquid or a solid. The liquid form is generally 20 to 45 percent ferric chloride and contains about 12 to 17 percent iron by weight [33,36]. Ferric chloride solutions generally are fed at the concentrations received from the supplier because dilution can lead to hydrolysis reactions and precipitation of ferric hydroxide.

An advantage of using ferric chloride for sludge conditioning is that it can be stored for long periods of time without deterioration. Normally, the chemical is stored in above-ground tanks constructed of resistant plastic or steel [38]. However, at lower temperatures, ferric chloride has the potential to crystallize. The crystallization tempera-

TABLE 3.23 Crystallization Temperatures for Ferric Chloride Solutions*

Solution strength (% $FeCl_3$)	Crystallization temperature (°F)
20	−5
25	−25
30	−50
35	−40
40	−10
45	+30

*Adapted from ref. [38].

ture varies with the concentration of ferric chloride in solution, as indicated in Table 3.23.

It is critical to note that the lowest freezing temperature occurs in the concentration range of 30 to 35 percent $FeCl_3$. Higher crystallization temperatures occur at both more dilute and more concentrated solutions. Because of its crystallization potential, ferric chloride storage tanks should be located indoors. If tanks must be placed outdoors, the tanks should be equipped with heaters and tank insulation.

An important consideration in the use of ferric chloride is its corrosive nature. Special materials must be used in handling ferric chloride, such as (1) epoxy, (2) rubber, (3) PVC, (4) vinyl, (5) synthetic resins, etc. Contact with skin and eyes must be avoided. Rubber gloves, goggles, or a face shield and a rubber apron should be used when handling this chemical.

3.3.7.2 Lime. Hydrated lime is usually used in conjunction with ferric iron salts. Iron salts (ferric chloride or ferrous sulfate) are usually added at a dose rate of approximately 40 to 125 lb/ton (20–62 kg/mt) of dry solids in the sludge feed regardless of whether lime is used. Lime dosage usually varies from 150 to 550 lb/ton (75–277 kg/mt) of dry solids dewatered. Although lime has some slight dehydration effects on colloids, it is chosen for sludge conditioning principally because it provides pH control, odor reduction, and disinfection. Another important consideration in the use of inorganic chemicals in conditioning is that the final sludge mass to be managed increases considerably with the use of these materials. The design engineer should expect 1 lb of additional sludge solids for every pound of lime and ferric chloride added in conditioning.

When lime is added to sludge, the calcium hydroxide reacts with the bicarbonate alkalinity to form calcium carbonate, which is insoluble [Eq. (3.54)]. The formation of calcium carbonate enhances the dewaterability of the sludge by increasing the porosity and reducing sludge compressibility.

$$\mathrm{Ca\,(OH)_2 + Ca\,(HCO_3)_2 \rightarrow 2CaCO_3 + 2H_2O} \qquad (3.54)$$

It should be noted that the high-pH environment resulting from adding lime enhances the volatilization rate of ammonia. Measures should be taken at the wastewater treatment plant to minimize ammonia release (e.g., elutriation) if lime conditioning is employed.

Two types of lime are used in sludge conditioning, quick lime (CaO) and hydrated or slaked lime [$Ca(OH)_2$]. Quicklime is less expensive to purchase than slaked lime and generally is used in larger facilities (i.e., those treating more than 5 MG/day of wastewater). However, quicklime use requires slaking (i.e., addition of water) prior to it being added to sludge. Although hydrated lime is more costly, it is commonly used in smaller facilities due to its convenience. Either form of lime may be purchased in bulk form or in bags. If purchased in bags, a waterproof building should be used for storage, with a maximum storage capacity restricted to approximately 60 days. If bags of quicklime are allowed to become wet, slaking will start within the bag, and the resulting heating and swelling may cause the bags to burst. If lime is stored in bulk, storage hoppers should be both watertight and airtight. Lime is not corrosive to steel or concrete, and either can be used as storage bin materials.

In addition to ferric salts and lime, other inorganic materials have been employed successfully to condition sludge. These materials include (1) pulverized coal, (2) cement kiln dust, and (3) fly ash. Although these materials are all effective in conditioning, in several cases materials handling problems have been reported with their use [39,71].

3.3.7.3 Polymers. Although initially used exclusively for conditioning primary sludges, improvements in polymer effectiveness have led to their use with all types of sludge. Polymers have been found to be popular in sludge conditioning operations because of their ease in handling and storage and the fact that they add little additional solids mass [48,71]. Polymers (or polyelectrolytes) are long-chain, water-soluble organic chemicals that are categorized as anionic, cationic, or nonionic depending on whether they are negatively, positively, or not charged [81]. The various polymers available commercially differ significantly in chemical composition, function, and cost. Selection of the correct polymer requires that the design engineer work closely with polymer suppliers, equipment suppliers, and plant operating personnel. Since new types and grades of polymers are being introduced continually, the evaluation of polymers must be an ongoing process. Evaluation of polymers should be made on site with the actual sludge to be conditioned.

Cationic (or positively charged) polyacrylamide polymers are the most widely used type of polymer for sludge conditioning. In addition to cationic polymers, anionic and nonionic polymers also may be used in sludge conditioning. However, these types of polymers generally are employed in conjunction with inorganic conditioning agents (e.g., lime). In this role, the polymer is responsible for interparticle bridging of sludge particles that have been destabilized by the inorganic conditioning chemical [81].

The characteristics of sludge to be processed and the type of dewatering device will determine which of the cationic polymers will work best and is most cost-effective. For example, an increasing degree of positive charge on the polymer is necessary when the sludge particles become finer (e.g., waste-activated sludge compared with primary sludge).

3.3.7.4 General design of a polymer feed system. A typical polymer feed system should include a day tank, polymer feed pumps, a dilution water system, and alternative feed points to the dewatering units. It should also include an in-line static mixer. The day tank should be sized to hold a 1-day supply of diluted polymer or less, be made of fiberglass (for corrosion resistance), and be equipped with a slow-speed mixer and a sight glass or level gauge.

The feed pumps should be either progressive cavity or diaphragm-type to minimize particle shearing. By using variable-speed pumps, the operating personnel can adjust the polymer dosage to compensate for changes in sludge characteristics. It should be noted that overconditioning of sludge is just as undesirable as underconditioning and can produce a sludge that is difficult to dewater.

The location of polymer feed points can significantly affect the performance of the polymer and therefore the dewatering operation. For centrifuges, the polymer feed point usually is located inside the dewatering unit. For the belt filter press, at least two or three optional locations should be specified, one adjacent to the dewatering unit, one about 1.0 to 1.5 m (3–5 ft) upstream and tied into the sludge feed line, and one about 6 to 9 m (20–30 ft) upstream. Connecting the feed points into the upstream sludge piping allows the polymer more time to mix thoroughly with the sludge and to form a better floc.

Polymers may be purchased in dry form, as emulsions, or as liquids. Moreover, dry polymers are available in powdered, granular, bead, or flake form. Dry polymers have high activities (i.e., amount of polymeric chemical contained in the product) and have a typical shelf life of 1 year or more. The active solids concentration of dry polymers is usually as high as 90 to 95 percent. Representative characteristics of dry cationic polymers are given in Table 3.24.

Dissolving dry polymer prior to mixing with sludge requires skill and care. A typical dry polymer makeup system is shown in Fig. 3.89.

The polymer preparation system should include an eductor or other polymer wetting device that allows for the proper rewetting of the

TABLE 3.24 Cationic Polymers Typically Used in Conditioning*

Relative cationic density†	Molecular weight‡	Approximate dosage (lb/dry ton sludge)
Low	Very high	0.5–10
Medium	High	2.0–10
High	Medium high	2.0–10

*Adapted from ref. [48].

†Low, <10 mol%; medium, 10–25 mol%; high, >25 mol%.

‡Very high, $4 \cdot 10^6$ to $8 \cdot 10^6$ g/mol; high, $1 \cdot 10^6$ to $4 \cdot 10^6$ g/mol; medium high, $5 \cdot 10^5$ to $1 \cdot 10^6$ g/mol.

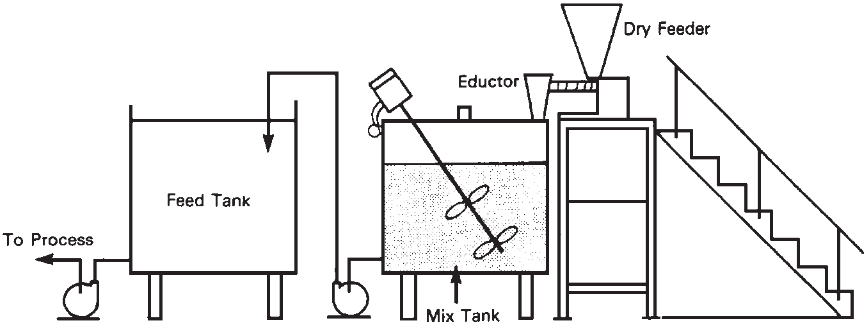


Figure 3.89 Dry polymer preparation system.

polymer particles before they enter the mixing tank. After the prewetted polymer enters the tank, it should be mixed slowly until it is completely dissolved (typically 60 minutes). Undissolved polymer can cause problems, including clogging of pumps and fouling of the filter belts or cloths. In addition to facilitating polymer dissolution, mixing also allows time for the polymer to age. During the aging process, the polymer molecule uncoils and takes on a form that enables it to cause flocculation of the sludge.

Liquid polymers can be supplied as either concentrated aqueous solutions or polymeric emulsions. Like dry polymers, liquid polymers are available in various activities (percentage of active solids). The concentration of the polymeric material that the manufacture can dissolve in water is usually controlled by the viscosity of the final solution.

Aqueous solutions of polymer are purchased in either 55-gal (208-liter) drums, in liquid bins of 275 gal (1040 liters), or in bulk quantities of about 5000 to 6000 gal (19,000–23,000 liters). In colder climates, storage areas for drums, liquid bins, and bulk liquids should be located indoors and heated. If the bulk storage tank must be located outdoors, the tank should be heated so that the solution's viscosity will not limit

TABLE 3.25 Cationic Liquid Polymers Typically Used in Conditioning*

Polymer type	Relative cationic density†	Molecular weight‡	Percent solids
Solution type			
Mannich product§	High	High to very high	4–8
Tertiary polyamine	High	Low	20–50
Quaternary polyamine	High	Very low to medium	20–50
Quaternary polyDADM	High	Low to medium	20–40
Emulsion type			
Polyacrylamide copolymer	Low Medium High	High to very high	25–60

*Adapted from ref. [48].

†Tertiary amine charge is affected by solution pH. Amines lose cationic charge in alkaline environments.

‡Very high, $4 \cdot 10^6$ to $8 \cdot 10^6$; high, $1 \cdot 10^6$ to $4 \cdot 10^6$; medium, $2 \cdot 10^5$ to $1 \cdot 10^6$; low, $1 \cdot 10^5$ to $2 \cdot 10^5$; very low, $<1 \cdot 10^5$.

§Product of condensation-type polymerization.

pumping. Mixing is normally not required. Characteristics of representative of liquid polymers are provided in Table 3.25.

To safely convey the polymer from drums into the storage tank, the operator should use either a polymer transfer pump or a drum lifting device to empty the drum. Polymer transfer pumps are also required to transfer the material from the storage tank to the mix tank. The pumps should be of the progressive cavity type so that the polymer is not subjected to high shear forces.

An illustration of a liquid polymer system is provided in Fig. 3.90. The system includes a mixing tank and a day storage tank for the diluted polymer. Typically, the operator will prepare a 0.1 percent solution of polymer. The concentration polymer and water should be mixed for about 30 minutes to ensure a homogeneous solution. Once the polymer has been diluted, it is usually stable for about 24 hours. Therefore, only enough polymer should be prepared for use in that time period.

To calculate the final polymer concentration for an aqueous solution of polymer, Eq. (3.55) may be used:

$$C_n = \frac{W_p}{V_w + V_p} \tag{3.55}$$

where C_n = polymer concentration, g/liter
W_p = weight of polymer added to mixing tank, g (lb)
V_w = volume of water added, liters (gal)
V_p = volume of polymer added, liters (gal)

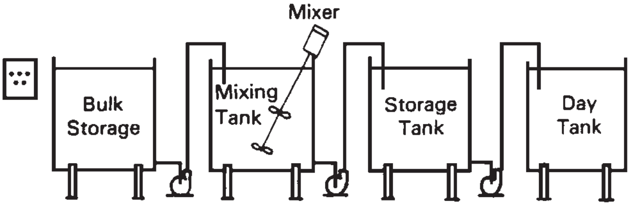


Figure 3.90 Schematic diagram of a liquid polymer feed system.

To estimate the polymer used per dry ton of solids dewatered, Eq. (3.56) should be used:

$$P_t = \frac{P_U}{W_s} \tag{3.56}$$

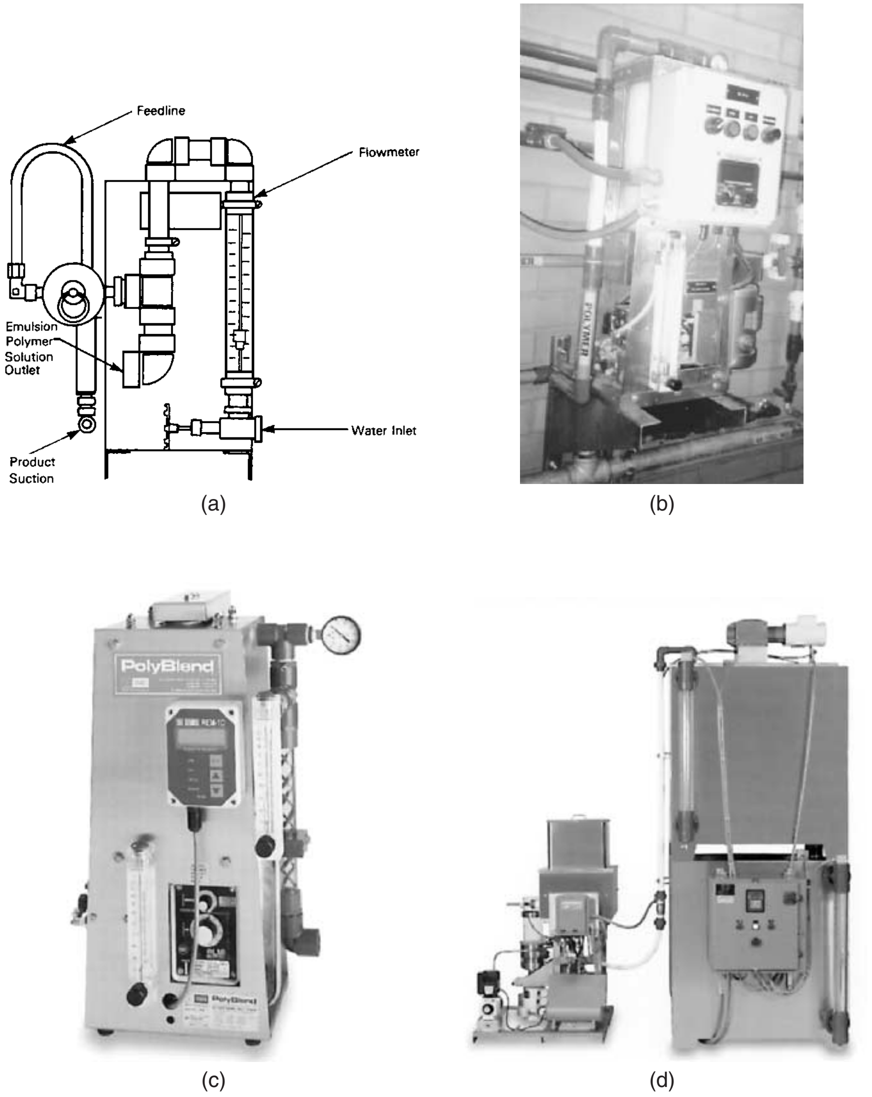
where P_t = pounds (kg) of polymer used per ton (mt) of dewatered sludge
P_U = weight of polymer used per day, lb/day (kg/day)
W_S = dry weight of sludge dewatered per day, ton/day (mt/day)

In addition to aqueous solutions, polymers may be purchased as emulsions. Emulsions are dispersions of polymer particles contained in a hydrocarbon oil. Surfactants are used to prevent separation of the polymer-oil phase from the water phase. Polymeric activities as high as 25 to 50 percent are common with emulsions [48].

Polymeric emulsions are available in 55-gal (208-liter) drums, in 275-gal (1040-liter) liquid bins, or in bulk quantities. Storage requirements are the same as for aqueous polymeric solutions, except that no water must come into contact with the emulsion until it is ready for mixing. Premature exposure to water will cause the polymer to coagulate. A schematic diagram of emulsion polymer preparation system is depicted in Fig. 3.91.

Compact and portable polymer feed automation equipment is available for in-line use that requires no batch mixing or aging tanks for emulsion-type polymer solutions (Fig. 3.91). However, effective application of polymer emulsion solutions requires that the emulsion be broken. Usually, dispersion using high-pressure water is employed to break the emulsion. The polymer should then be aged for at least 30 minutes, after which it is ready for use.

3.3.7.5 Polymer use in belt filter press operation. Compared with other mechanical dewatering processes, belt filter presses have the greatest need to optimize the polymer dosage based on incoming sludge characteristics. Underconditioning of influent sludge results in inadequate drainage of the free water in the gravity dewatering zone. Inadequate drainage can cause sludge overflow in the gravity zone or extrusion of



(a) (b) (c) (d)

Figure 3.91 (*a*) Schematic diagram of an emulsion polymer mixing system. (*b*) Photograph of an automatic polymer mixing and discharge system. (*c*) Photograph of a liquid polymer mixing system. (*Courtesy of US Filter/Stranco.*) (*d*) Photograph of a dry polymer mixing system. (*Courtesy of US Filter/Stranco.*)

the sludge in the pressure zone. Underconditioned biological sludges also can result in clogging of the filter belt.

Overconditioning of influent sludge can cause the sludge to become extremely sticky, resulting in belt clogging together with problems with cake release. Moreover, overflocculated sludge may drain so rapidly that the solids are not distributed evenly across the belt.

TABLE 3.26 Typical Dosages of Dry Polymers for a Belt Filter Press*

Sludge type	Pounds of dry polymer per ton of dry solids	Typical value
Raw		
Primary	2–9	5
Primary plus trickling filter	3–15	10
Primary plus WAS	2–20	7
Waste-activated sludge (WAS)	2–20	10
Anaerobically digested		
Primary plus WAS	4–15	10
Aerobically digested		
Primary	2–10	3
Primary plus WAS	3–15	6

*Adapted from ref. [38].

Uneven distribution of sludge can cause tracking problems with the belt and result in a poor-quality sludge cake. Table 3.26 lists typical levels of dry polymer addition to influent sludge for proper dewatering on a belt filter press.

3.3.7.6 Polymer use in solid-bowl centrifuge operation. Solid-bowl centrifuges usually require addition of polymer to obtain satisfactory performance. Table 3.27 lists levels of dry polymer dosages required for this type of dewatering operation.

3.3.7.7 Polymer use in vacuum filter operation. Many of the vacuum filters in the United States have now converted from inorganic conditioning chemicals to organic polymers. With polymers, there are many advantages, such as lower costs and fewer materials handling problems. Moreover, the mass of solids to be disposed will not increase, as

TABLE 3.27 Typical Dosages of Dry Polymers for a Solid-Bowl Centrifuge*

Sludge type	Pounds of dry polymer per ton of dry solids	Typical value
Raw		
Primary	2–7	4
Primary plus WAS	4–15	8
Anaerobically digested		
Primary	6–10	6
Primary plus WAS	7–15	8
Thermally conditioned		
Primary plus WAS	3–5	3
Primary plus trickling filter	7–15	8

*Adapted from refs. [38,49].

TABLE 3.28 Typical Dosages of Dry Polymers for Vacuum Filtration*

Sludge type	Pounds of dry polymer per ton of dry solids	Typical value
Raw		
Primary	0.5–1	1
Primary plus trickling filter	2.5–5	4
Primary plus WAS	4–10	6
Waste-activated sludge (WAS)	8–15	12
Anaerobically digested		
Primary	1.5–4	1.5
Primary plus WAS	5–12	7

*Adapted from refs. [38,48].

occurs with inorganic conditioning chemicals, and the volatile content of the cake will be much higher. Table 3.28 lists dry polymer dosages required for this type of dewatering operation.

3.3.7.8 Polymer use in drying bed operation. Sludge dewatered through the use of drying beds can be conditioned with polymers. The addition of approximately 0.5 to 2.0 lb of cationic polymer per ton of dry solids (0.25–1.0 kg/mt) has been found to significantly increase dewatering rates [48].

3.3.7.9 Chemical conditioning installation design. The first step in the design of a new chemical conditioning system is to estimate the amount and type of sludge that will be produced at the wastewater treatment plant. If no plant data are available, the following average data should be used: (1) 2980 gal of primary sludge having a solids content of 4 percent is produced per million gallons of wastewater treated (3 liters/m^3 of wastewater treated) and (2) 28,025 gal of secondary sludge having a solids content of 1 percent is produced per million gallons of wastewater treated (18 liters of secondary sludge produced per cubic meter of wastewater treated) [38]. Once the sludge flow rates have been estimated, the necessary conditioning equipment can be designed, as illustrated in Example 3.24.

Example 3.24 The Lodwar County Water Reclamation Plant is currently treating 2 million gal/day of municipal wastewater using both primary and secondary treatment. An engineering consultant has indicated that the facility can significantly reduce the volume of its sludge by employing polymer conditioning. If the sludge generated from primary and secondary treatment is approximately 1 and 4 percent total solids, respectively, estimate the pounds of conditioning polymer used and the volume of the polymer day tank. Assume that the combined primary and secondary sludge is thickened to 5 percent total solids prior to polymer conditioning and that polymer is added at a rate of 20 lb per dry ton of solids. Assume that polymer solution in the day tank is maintained at 0.5 percent polymer concentration (weight basis) and that sludge will only be processed 5 days per week.

solution

Step 1. Estimate the daily volume of primary and secondary sludge generated using typical estimates:

$$\text{Gallons of primary sludge} = 2\ \text{MG/day} \cdot \frac{2980\ \text{gal}}{\text{MG/day}}$$

$$= 5960\ \text{gal (at 4 percent TS)}$$

$$\text{Gallons of secondary sludge} = 2\ \text{MG/day} \cdot \frac{28{,}025\ \text{gal}}{\text{MG/day}}$$

$$= 56{,}050\ \text{gal (at 1 percent TS)}$$

Step 2. Estimate the daily volume of the combined thickened sludge:

$$\text{Volume of thickened sludge (gal)} \cdot (50\ \text{g/liter})$$

$$= \text{volume of primary sludge} \cdot (40\ \text{g/liter}) + \text{volume of secondary sludge} \cdot (10\ \text{g/liter})$$

$$\text{Volume of thickened sludge (gal)} = \frac{5960\ \text{gal} \cdot (40\ \text{g/liter}) + 56{,}050 \cdot (10\ \text{g/liter})}{50\ \text{g/liter}}$$

$$= 15{,}978\ \text{gal}$$

Step 3. Estimate the gallons of thickened sludge that must be processed per day during the 5-day work week:

$$\text{Thickened sludge processing rate (gal/day)} = 15{,}978\ \text{gal/day} \cdot 7\ \text{days/week} \cdot \text{week/5 days}$$

$$= 22{,}370\ \text{gal/day}$$

Step 4. Estimate the tons of dry solids produced per day during the 5-day work week:

$$\text{Dry solids production (tons/day)} = \frac{\text{ton}}{2000\ \text{lb}}\left[\frac{22{,}370\ \text{gal}}{1 \cdot 10^6\ \text{gal/mg}} \cdot \frac{50{,}000\ \text{mg}}{\text{liter}} \cdot \frac{8.34\ \text{lb}}{\text{MG} \cdot (\text{mg/liter})}\right]$$

$$= 4.7\ \text{tons/day}$$

Step 5. Estimate the pounds of dry polymer needed to condition the thickened sludge per day during the 5-day work week:

$$\text{Pounds of dry polymer required (lb/day)} = \text{tons dry solids/day} \cdot 20\ \text{lb/ton}$$

$$= 4.7\ \text{tons/day} \cdot 20\ \text{lb/ton}$$

$$= 94.0\ \text{lb/day}$$

Step 6. Estimate the volume of the polymer day tank assuming that the density of the diluted polymer solution is 8.34 lb/gal:

Volume of polymer day tank (gal)

$$= 94.0\ \text{lb/polymer} \cdot \frac{100\ \text{lb polymer solution}}{0.5\ \text{lb polymer}} \cdot \frac{\text{gal}}{8.34\ \text{lb polymer solution}}$$

$$= 2254.2\ \text{gal (use a 2500-gal day tank)}$$

3.3.8 Selection of conditioning chemicals

Many factors must be considered in selecting the appropriate conditioning chemical to be used at a particular plant. These factors include (1) performance, (2) materials handling, (3) storage requirements, (4) type of dewatering unit, (5) final disposal method, and (6) economics. For example, a facility whose final sludge disposal method is incineration desires the driest cake possible with the least mass and the highest organic matter content. Therefore, polymer conditioning usually is preferred over inorganic chemicals. Polymer conditioning also proves to be the better choice if either storage space is at a minimum or if materials handling is a concern.

There are presently several tests that can be performed to evaluate the effectiveness of conditioning chemicals as well as the optimal dosing level required to achieve a desired dewatered sludge solids content. These tests include (1) jar tests, (2) the filter leaf test, (3) specific resistance or Buchner funnel test, and (4) capillary suction time. The following sections are brief descriptions of these tests. Additional information can be found in the following references [33,38,39,48,80].

3.3.8.1 Jar test. The jar test is used to provide rapid screening of conditioning chemicals when the design engineer has to choose among a wide array of potential products. Taking four to six large beakers of 1-liter capacity and filling them with about 600 ml of sludge is required for this test. Solutions of different types of conditioning chemicals are then prepared and added to the beakers in accordance with the manufacturer's instructions. The beakers are then placed on a stirrer with the rotational speed set at about 75 rev/min. The sludge–conditioning chemical mixture is then mixed for about 30 seconds, after which the operator observes the effectiveness of floc formation and settling.

3.3.8.2 Filter leaf test. The filter leaf test is used for evaluating the effect of conditioning chemicals on the dewaterability of sludge by a vacuum filter. In some cases, this test has been used to size a vacuum filter. The test is performed by assembling a filter leaf apparatus as shown in Fig. 3.92. The filter cloth should be of the same fabric as that intended for actual use in the vacuum filter.

In this test, 2 liters of chemically conditioned sludge are prepared and transferred to a beaker. The temperature of the conditioned sludge is measured, and the filter leaf is then submerged in the mixture to a depth of about 2 in (5 cm) below the surface. A vacuum equivalent to 20 inHg (51 cmHg) is applied to the filter leaf, and timing begins. After 45 seconds, the filter leaf is withdrawn and dried for 90 seconds. This test simulates a 3-minute cycle time, divided into 45 seconds of form time, 90 seconds of drying time, and 45 seconds of discharge time.

The cake thickness is measured, and the cake is scraped into a previously weighed dish. The dish and cake are weighed and transferred to a drying oven. After air drying, the cake should be desiccated and weighed. The dried sludge is then volatilized at 550°C (1022°F) and weighed again to estimate the volatile solids content. For each test, the filter yield may be calculated by Eq. (3.57):

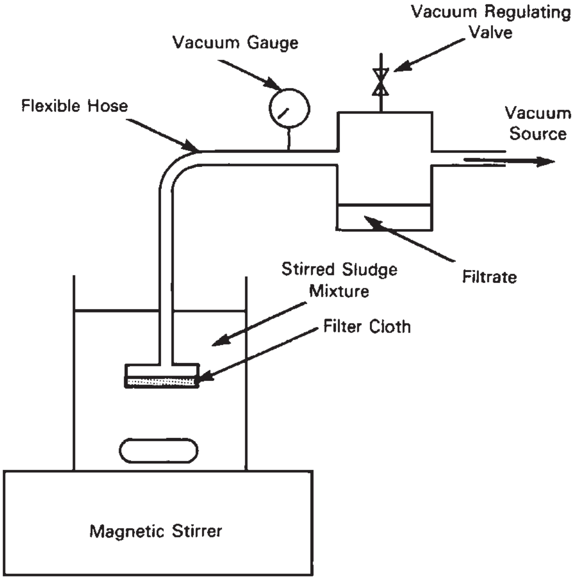


Figure 3.92 Filter leaf apparatus.

$$\text{Yield}\left(\frac{\text{lb of cake}}{\text{ft}^2\cdot\text{h}}\right)=\frac{\text{dry weight of cake (g)}\cdot\dfrac{\text{cycles}}{\text{h}}}{\dfrac{454\text{ g}}{\text{lb}}\cdot\text{filter area (ft}^2\text{)}}\qquad(3.57)$$

3.3.8.3 Specific resistance/Buchner funnel test. The specific resistance test is another method of evaluating the effectiveness of the conditioning chemical/agent. The Buchner funnel test equipment consists of a graduated cylinder, a Buchner funnel, and a vacuum pump, as depicted in Fig. 3.93.

A series of 600-ml conditioned sludge samples are prepared in 1-liter beakers. A Buchner funnel is mounted on top of the graduated cylinder, and the funnel is fitted with a piece of filter paper. For each test, a portion of the conditioned sludge (50–200 ml) is poured into the funnel. After 2 minutes of gravity drainage, the vacuum pump is turned on (at a vacuum equivalent to 15 inHg). At 15-second intervals, the filtrate volume is measured and recorded until the vacuum breaks or additional water cannot be removed. The sludge cake is then removed from the filter and placed in a preweighted dish. The wet weight and dry weight of the sample are measured as part of the evaluation. The time and volume of filtrate data are transformed to prepare a plot similar to the one shown in Fig. 3.94.

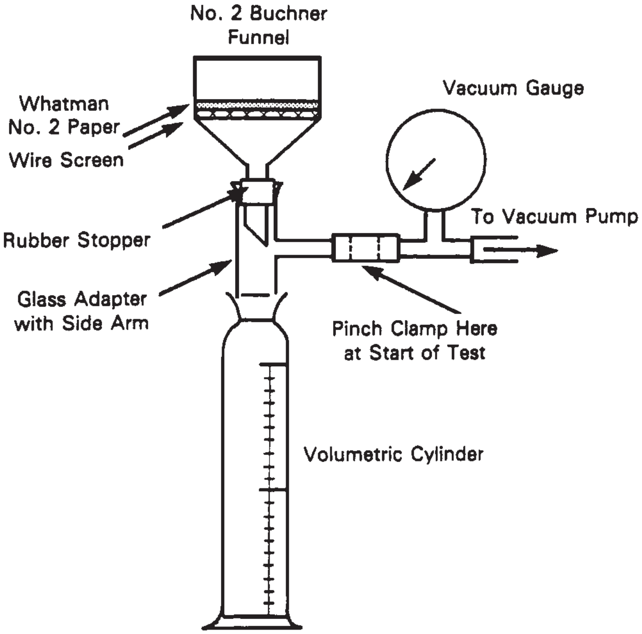


Figure 3.93 Buchner funnel test apparatus.

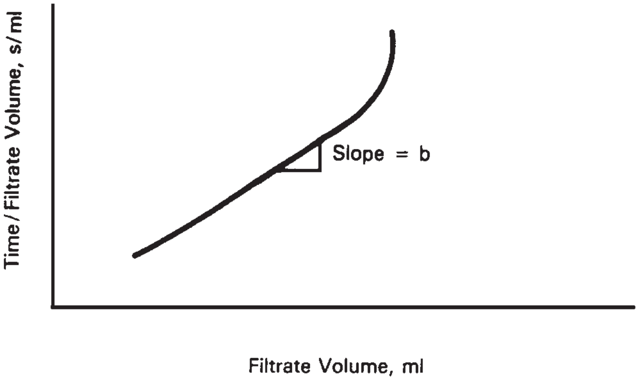


Figure 3.94 Time/filtrate volume versus volume from Buchner funnel test.

The slope of the straight portion of the graph b is used to calculate the specific resistance r of the conditioned sludge using Eq. (3.58):

$$r = \frac{2PA^2b}{\mu W} \tag{3.58}$$

where r = specific resistance, m/kg
P = pressure of filtration, N/m^2
A = area of filter, m^2
B = slope of curve, s/cm^6
μ = viscosity of filtrate, $N \cdot s/m^2$
W = weight of dry solids/volume of filtrate, kg/m^3

The magnitude of specific resistance r can be used to compare the effectiveness of various dosages of conditioning chemicals to improve the dewatering characteristics of sludge. Figure 3.95 illustrates a plot of specific resistance r versus conditioning chemical dose. This plot was constructed with specific resistance data from sludge conditioned with various levels of the same polymer. The optimal chemical dosage is that which produces the lowest specific resistance.

A modification of the Buchner funnel test can be used to simulate the gravity drainage that occurs on a belt filter press. The test uses the apparatus shown in Fig. 3.93 exclusive of the vacuum pump. A piece of the belt material is placed in the Buchner funnel. A sample of conditioned sludge is then placed into the funnel, and the volume of water released is measured at regular intervals. Both time and volume are recorded. The polymer and/or dose that produces the greatest volume of drainage in the shortest time interval is the polymer that should be employed [48].

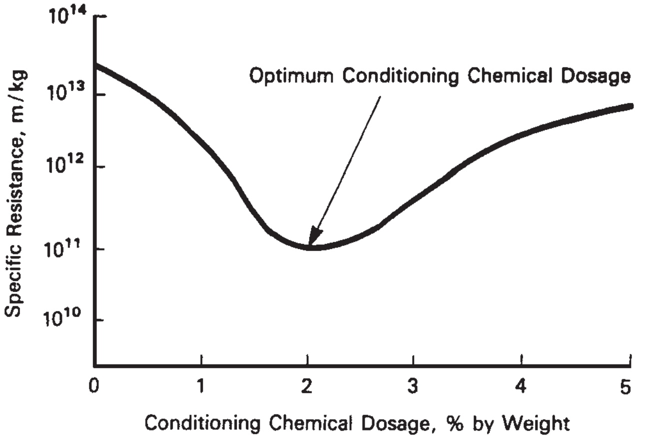


Figure 3.95 Specific resistance versus chemical conditioning dose.

3.3.8.4 Capillary suction time. The capillary suction time (CST) is a rapid test that measures the time required for the liquid portion of the conditioned sludge to travel a fixed distance (typically 1 cm). The apparatus used in this test consists of a timing device, an upper plate containing probes that activate and deactivate the timing device, and a lower plate that holds the filter paper and a metal sampling container (Fig. 3.96).

During the capillary suction time test, a sample of conditioned sludge is placed in the sample container. As water migrates through the paper and reaches the first probe, it activates the timer. When the water reaches the second probe, the timer deactivates. The time interval between timer activation and deactivation is the capillary suction time and is a measure of the dewaterability of the conditioned sludge.

Capillary suction time is plotted versus chemical dosage. The dosage that gives the shortest time to traverse the 1-cm distance is the optimal dosage. Conditioning chemicals and concentrations should be varied until the optimal chemical and dosage are found.

3.3.9 Elutriation

Elutriation refers to the washing of anaerobically digested sludge prior to chemical conditioning. Washing causes a dilution of the bicarbonate alkalinity in the sludge and can reduce the demand for chemicals by as much as 50 percent [38]. Elutriation also can result in removing a substantial amount of ammonia. The washing process normally involves the use of plant effluent as the wash water.

Although effective in removing alkalinity, elutriation also may remove a considerable amount of solids [38]. Since elutriation wash

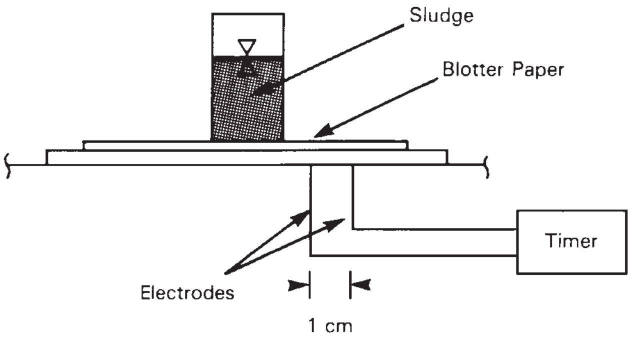


Figure 3.96 Schematic diagram of capillary suction time (CST) apparatus.

water is recycled to the wastewater treatment plant headworks, care must be taken to reduce the amount of solids that are transferred to the wash water. In some cases, the wash water is treated with polymer to remove excess solids prior to being transferred to the plant headworks [38].

3.3.10 Thermal conditioning

Thermal conditioning involves the heating of sludge to temperatures ranging from 250 to 400°F (177–240°C) in a reaction vessel under pressures that range from 250 to 400 psig (1723–2758 kN/m^2) for periods of 15 to 40 minutes [48]. At these temperatures and pressures, thermal conditioning also acts as an effective sludge stabilization process. Two basic thermal conditioning operations currently are employed in sludge treatment: (1) low-pressure oxidation (LPO process) and (2) heat treatment (HT process) (Fig. 3.97).

The principal difference in the two operations is that the low-pressure oxidation process introduces air into the conditioning system, whereas the heat treatment process does not.

During the thermal conditioning process, a portion of the volatile suspended solids is solubilized as a result of the breakdown in the sludge structure [38]. Although this solubilization process does not change the total organic carbon content of the sludge, it does increase the soluble BOD_5 concentration, making treatment of this side stream a major concern for secondary treatment operations. The solubilizing of volatile suspend solids and the resulting soluble BOD_5 generated from the subsequent mechanical dewatering systems may be estimated using Eqs. (3.59) and (3.60), respectively. For illustrations of the approach used to estimate side-stream quality, see Chap. 5.

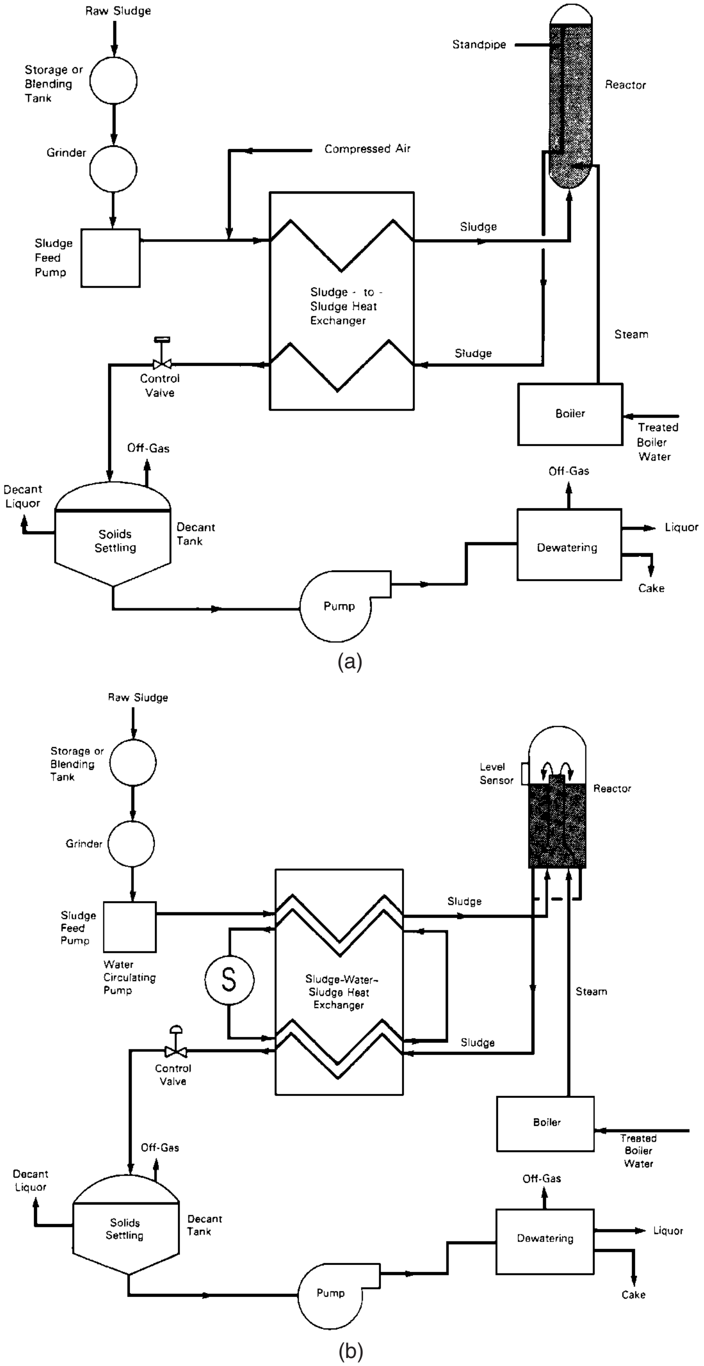


Figure 3.97 Schematic diagrams of thermal conditioning processes: (*a*) low-pressure oxidation; (*b*) heat treatment [38].

$$\mathrm{VSS} = 0.1\mathrm{PS} + 0.4\mathrm{WAS} \tag{3.59}$$

$$\mathrm{BOD}_5 = 0.07\mathrm{PS} + 0.3\mathrm{WAS} \tag{3.60}$$

where VSS = volatile suspended solids solubilized during thermal treatment, kg (lb)
PS = primary sludge added, kg (lb)
WAS = waste-activated sludge added, kg (lb)
BOD_5 = 5-day biochemical oxygen demand produced by VSS solubilized, kg (lb)

In general, the composition of the filtrate, centrate, or decantate generated from the mechanical dewatering of thermally conditioned sludge is difficult to predict. The composition will be a function of the type of sludge, feed volatile solids concentration, reaction time, and temperature [48]. It is recommended that pilot testing be conducted to determine the side-stream characteristics and the optimal treatment strategy.

Although effective in producing a well-conditioned sludge, many wastewater treatment plants that employ thermal conditioning systems have indicated that the process has exceptionally high operation and maintenance costs [48]. The high operating costs result primarily from the cost of fuel for steam generation and chemical addition for boiler water treatment. The maintenance costs involve replacing various parts, washing the heat exchanger and reactor to remove scale, and the labor to perform these tasks. An excess accumulation of scale has been found to occur in wastewater treatment plants that are treating a hard wastewater influent and/or wastewater treatment plants that are treating a significant amount of industrial discharge. The advantages and disadvantages of employing thermal sludge conditioning are listed in Table 3.29.

TABLE 3.29 Advantages/Disadvantages of Thermal Conditioning*

Advantages
Produces sludge with excellent dewatering characteristics.
No chemical addition necessary.
Process sterilizes sludge.
Process is insensitive to sludge composition.
Disadvantage
Process has a high capital cost.
Process requires highly skilled operators.
Process produces an odorous gas that must be treated.
Process produces liquid side stream with high concentrations of ammonia.
Scale formation in heat exchangers, pipes, and reactor require acid washing.

*Adapted from ref. [38,48].

3.4 Dewatering

Dewatering is the removal of water from sludge to achieve an overall volume reduction. Sufficient water is removed during the dewatering process to produce a sludge that is no longer fluid and must be handled and transported as a solid. The principal variables in any dewatering process include

- Solids concentration and volumetric flow rate of the feed sludge stream
- Chemical demand and costs
- Solids concentration and volumetric flow rate of the dewatered sludge

3.4.1 Strategy for dewatering process selection

The most common methods of dewatering sludge include (1) belt press filtration, (2) centrifugation, (3) pressure filtration, (4) vacuum filtration, (5) drying beds, (6) drying lagoons, and (7) heat drying. Although heat drying is included in the list of dewatering methods, it is normally preceded by a mechanical dewatering process.

A dewatering process cannot be evaluated without consideration of the overall sludge-handling system, including prior sludge treatment processes and subsequent disposal practices. Determining the most cost-effective dewatering system is normally an iterative process that should account for the various combinations of unit operations available [39,48]. The strategy involved in selection of a dewatering process consists of the following five stages of analysis:

Stage 1. Initial screening of dewatering concepts

Stage 2. Initial cost evaluation

Stage 3. Laboratory testing

Stage 4. Field-level testing

Stage 5. Final evaluation based on detail design parameters

Collectively, these stages represent a screening procedure in which dewatering processes under consideration are given increasing scrutiny as more detailed cost, operational, and design data are collected [38]. The following sections highlight the important aspects of each of the five stages.

3.4.1.1 Compatibility of dewatering process with plant size. Use of uncomplicated sludge dewatering equipment increases the probability

TABLE 3.30 Compatibility of Dewatering Equipment with Plant Size*

Dewatering process	<1 MG/day	1–10 MG/day	>10 MG/day
Centrifugation		X	X
Belt filter press	X	X	X
Vacuum filter		X	X
Recessed plate/diaphragm press		X	X
Drying bed	X	X	
Sludge lagoon	X	X	

*Adapted from ref. [48].

for successful operation in any size wastewater treatment plant. Complex equipment is especially unsuited for small plants (i.e., those which treat less than 5 MG/day of wastewater) for several reasons, including (1) the amount of operator time available for dewatering equipment maintenance generally decreases as plant size decreases, (2) the number of highly skilled operators is normally less at smaller wastewater treatment plants, and (3) the less complex the dewatering equipment is, the lower are the capital and maintenance costs [39,48].

The choice of the dewatering process to be used is normally a decision of the owner of the facility. Although there are no rules concerning the type of dewatering equipment to be used for a certain size wastewater treatment plant, certain guidelines do exist based on a survey of wastewater treatment plants across the United States. These data are summarized in Table 3.30.

3.4.1.2 Compatibility of dewatering process with subsequent treatment/disposal options. Careful attention must be paid to the methods of final sludge disposal and the sludge moisture content required by the disposal option. A potentially costly situation is for the dewatering equipment to remove more water than is necessary for the selected disposal option. Table 3.31 presents a general guideline for the compatibility of sludge final disposal options with the various methods of dewatering. It should be noted that the information in Table 3.31 must be evaluated on a case-by-case basis.

3.4.2 Dewatering processes

Dewatering processes are classified as either mechanical or air-drying processes. Mechanical systems include (1) belt filter press (belt press filtration), (2) centrifugation, (3) recessed-plate or diaphragm filtration (filter press), and (4) vacuum filtration. Air-drying processes include drying beds and sludge lagoons.

Conditions favoring the use of mechanical dewatering include (1) aesthetics, (2) climate, (3) costs, and (4) site limitations. All mechanical

TABLE 3.31 Dewatering Process Compatibility with Subsequent Treatment/Disposal Options*

Dewatering process	Composting	Land application	Incineration
Centrifuge	X	X	X
Belt filter press	X	X	X
Vacuum filter	X	X	X
Recessed plate/diaphragm filter press	X	X	X
Drying bed	X	X	
Sludge lagoon		X	

X denotes a technically compatible process.
*Adapted from ref. [48].

dewatering equipment employs filtration as the principal mechanism for moisture removal. Because of the frictional losses associated with water movement in mechanical dewatering systems, a significant pressure drop is required for the liquid to flow through the porous medium. The required pressure drop can be achieved in mechanical dewatering systems using one of the following approaches:

- Creating a centrifugal force
- Placing a vacuum on one side of a porous medium
- Raising the pressure on one side of a porous medium

At present, belt filter presses and solid-bowl centrifuges are the mechanical dewatering systems most frequently used in the United States. Vacuum filters, although commonly installed up to the mid-1970s, are rarely selected today. Basket centrifuges have never been a common selection for sludge dewatering. Filter presses, although seldom selected, can be a cost-effective dewatering alternative if an extremely dry cake is required [48].

Drying beds and sludge lagoons are used commonly at small wastewater treatment plants (i.e., less than 5 MG/day) that have land available and in larger plants that have both available land and are located in areas with significant evaporation rates. The following sections briefly discuss each dewatering process, their operational principles, and key advantages and disadvantages.

3.4.3 Belt press filtration

Belt filter presses are designed on the basis of the concept that when sludge is compressed between two tensioned porous belts that are passed over and under various diameter rollers, significant quantities of water are released. The mechanical components of a belt filter press include (1) dewatering belts, (2) rollers and bearings, (3) a belt tracking and tensioning system, (4) controls and drives, and (5) a belt washing

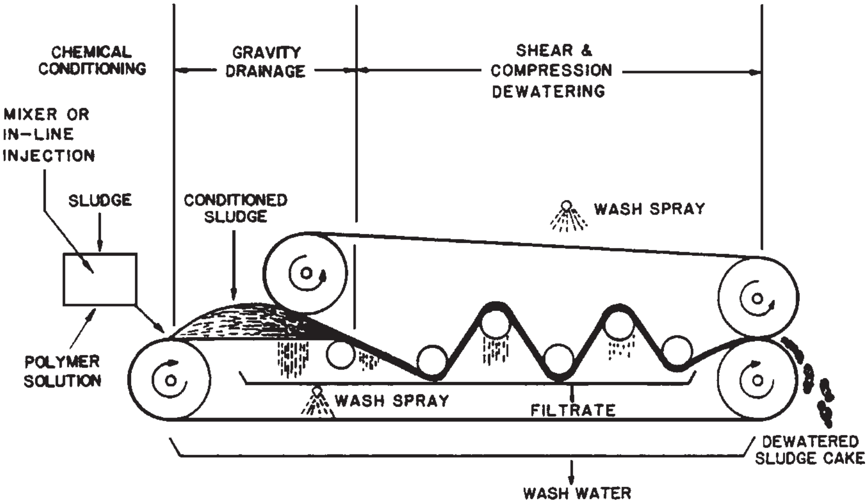


Figure 3.98 Schematic diagram of a belt filter press.

system. For a given belt tension, as roller diameter decreases, an increased pressure is exerted on the sludge, causing more water to be transmitted. Although many different designs of belt filter presses are available, they all contain the following features: (1) polymer conditioning zone, (2) gravity drainage zone, (3) low-pressure zone, and (4) high-pressure zone (Fig. 3.98).

The polymer conditioning zone can be a small tank (70–100 gal) located 2 to 6 ft from the press, a rotating drum attached to the top of the press, or an in-line injector. The gravity drainage zone is a flat or slightly inclined belt that is unique to each belt filter press model. In this section, sludge is dewatered by the gravity drainage of free water. This section typically employs stationary plows to assist in the gravity dewatering (Fig. 3.99). The design engineer should expect a 5 to 10 percent increase in solids concentration in the gravity drainage zone from the original feed [48].

Problems such as sludge squeezing out from between belts and clogging of the belt mesh can occur if the sludge does not drain well in the gravity drainage zone. The free water drainage is a function of sludge type, quality, conditioning, screen mesh, and design of the drainage zone.

The low-pressure zone, also called the *wedge zone,* is the area where the upper and lower belts come together forming the sludge "sandwich." The low-pressure zone is important because it prepares the sludge by forming a firm sludge cake that is able to withstand the shear forces within the high-pressure zone (Fig. 3.100).

In the high-pressure zone, forces are exerted on the sludge by the movement of the upper and lower belts guided by a series of rollers

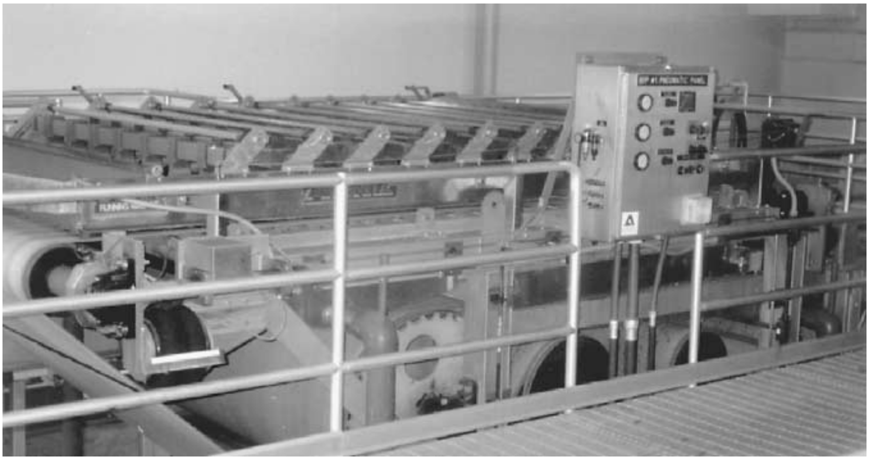


Figure 3.99 A belt filter press.

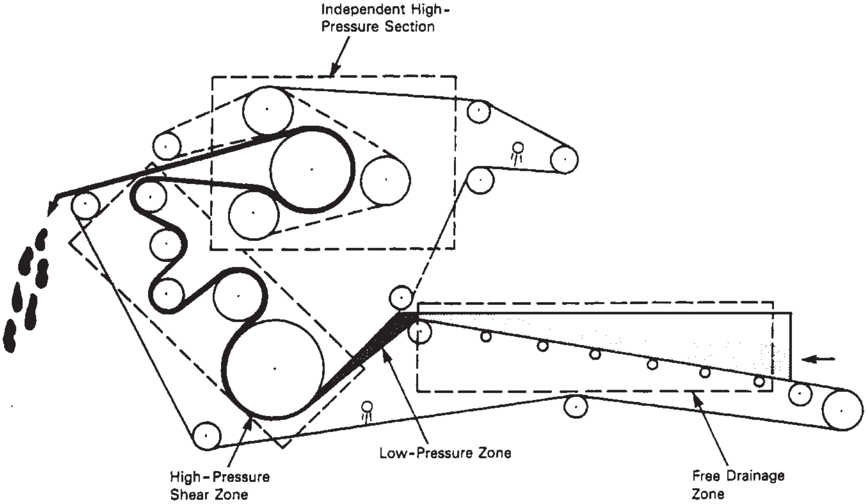


Figure 3.100 Schematic diagram of the pressure zones in a belt filter press.

with decreasing diameters. Some manufacturers have an independent high-pressure zone that uses belts or a hydraulic cylinder to further increase the pressure on the sludge. Photographs of sludge being processed in both the gravity drainage and high-pressure zones are provided in Fig. 3.101.

The dewatering belts usually are woven from monofilament polyester fibers. Various weave combinations, air permeability, and retention capabilities are available from belt manufacturers. These parameters significantly influence belt filter press performance. The specific belt

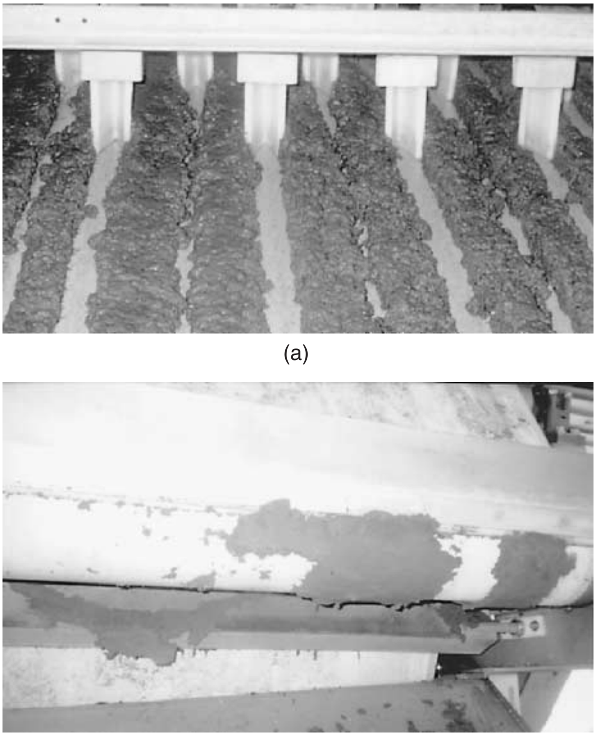


Figure 3.101 (*a*) Sludge after passing through the gravity drainage zone. (*b*) Sludge being discharged from the high-pressure zone.

type recommended by the supplier should be based on the sludge type and past experience.

The belts should be designed for ease of replacement with a minimum of downtime to ensure continuous dewatering. Two different types of belts are available: (1) split and (2) continuous. Split belts are joined together with a splicing device called a *clipper seam*. Split belts are the most common type and can be used on all models of belt filter presses. The continuous or seamless belt can only be used on certain belt filter presses [38,48].

The rollers and bearings are the main mechanical components of a belt filter press. The rollers impart the pressure that causes dewatering of the sludge to occur. The rollers also ensure proper belt support and tension. The tensioning device is used to maintain suitable pressure on the sludge once the roller sizes have been fixed. Roller diameter and shaft sizes are important design parameters and should be evaluated carefully.

Belt filter press controls should be located either on the press itself or on a remote-control panel. The control system should include automatic startup and shutdown capabilities, instrumentation for tracking and tensioning of the belts, pressure gauges, and safety interlocks. In addition, many belt filter press control systems include running-time meters, sludge and polymer pump controls, and other auxiliary devices that allow for a more efficient operation. Automatic shutdown of the belt filter press should occur for any of the following conditions: (1) belt drive failure, (2) sludge conditioning tank failure, (3) belt misalignment, (4) insufficient belt tension, (5) loss of pneumatic or hydraulic pressure, (6) low belt water pressure, (7) emergency stop, (8) high sludge level on the gravity drainage section, and (9) polymer feed pump shutdown.

To ensure effective dewatering, belts must be washed to remove any adhered solids after the dewatered cake has been discharged. The belt washing system should include a high-pressure water pump, a set of spray bars for cleaning both the upper and lower belts, and a spray-cleaning device. The belt is always washed from the side opposite to that which is in contact with the sludge cake (Fig. 3.102). Either potable water or high-quality plant effluent can be used as wash water for the cleaning the belts. However, when plant effluent is used, a highly efficient filtration system must be installed to ensure that the effluent is free of solids that can clog the spray nozzles of the cleaning system. Many of the newer models of belt filter presses are equipped with stainless steel brushes within the spray header to automatically clean the spray nozzles.

The flow rate required for belt washing is normally 50 to 100 percent of the flow rate of sludge. The combined filtrate and belt wash water is

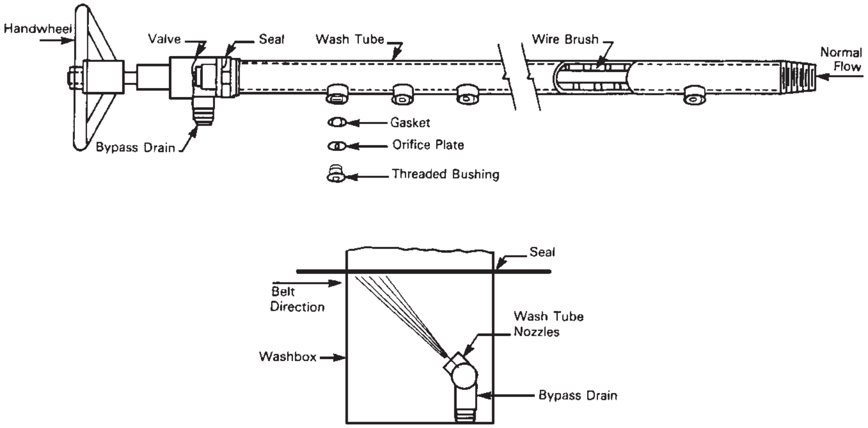


Figure 3.102 Schematic diagram of water spray bar with cleaning brushes [48].

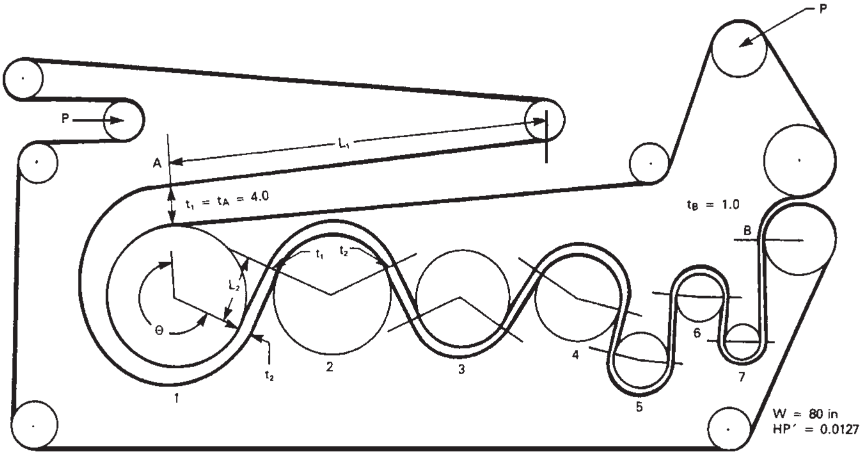
normally 1.0 to 1.5 times the incoming sludge flow. Some belt presses recirculate wash water from the filtrate collection system to wash the belts, but normally secondary effluent or potable water is used. This combined flow of wash water and filtrate typically contains between 500 and 2000 mg/liter of suspended solids and normally is recycled to either the primary or secondary wastewater treatment operations.

Belt press filtration performance is measured by (1) percentage solids of the sludge cake, (2) percentage solids capture, (3) solids and hydraulic loading rates, and (4) required polymer dosage. When dewatering a 50:50 mixture of anaerobically digested primary sludge and waste-activated sludge, a belt filter press typically will produce a cake with a solids content of 18 to 23 percent.

3.4.3.1 Belt filter press theory. The total pressure on the sludge cake within the pressure zones is due to (1) pressure on the belt due to the drive torque, (2) pressure due to belt tension, and (3) pressure due to belt elasticity. Figure 3.103 presents a schematic diagram of the location of the various pressures exerted on the sludge.

To estimate the pressure on the sludge due to the drive torque (e.g., force required to pull the belt through the press), Eq. (3.61) may be used:

$$\text{psi}_1 = \frac{2F_1}{D} = \frac{1700 \cdot \text{hp}}{DV} \tag{3.61}$$



Roll	1	2	3	4	5	6	7
Diameter, in	36	30	24	21	13	10	8
L_1, in	94	16	15	16	11	17	10
L_2, in	16	15	16	11	17	10	24
Θ	244°	130°	114°	160°	185°	180°	180°
$\frac{\pi D\Theta}{360}$	76.6	34.0	23.9	29.3	21.0	15.7	12.6

Figure 3.103 Schematic diagram of high-pressure filtration zones.

where psi_1 = maximum pressure (lb/in^2) on the sludge cake due to force F_1
F_1 = pounds of force due to drive torque per inch of belt width
D = roller diameter, in
hp = drive horsepower per inch of belt width per belt
V = belt speed, ft/min

Equation (3.62) may be used to estimate the pressure on the sludge cake due to belt tension (applicable to belt presses that use pneumatic or hydraulic cylinders to adjust the tension of the belts), whereas Eq. (3.63) can be employed to estimate the pressure on the sludge due to belt elasticity.

$$\text{psi}_2 = \frac{2F_2}{D} = 2P\cos\left(\frac{2a}{DWY}\right) \tag{3.62}$$

where psi_2 = maximum pressure (lb/in^2) on the sludge cake due to force F_2
F_2 = pounds of force due to take up tension per inch of belt width (required to prevent slack in belt and to provide traction for the drive rollers)
P = resulting force from tensioning roller actuator
a = angle between belt force resultant and actuator cylinder axis
D = roller diameter, in
W = active belt width, in
Y = belt wrap angle at take-up roller

$$\text{psi}_3 = \frac{2F_3}{D} \tag{3.63}$$

where psi_3 = maximum pressure (lb/in^2) on the sludge cake due to force F_3
F_3 = pounds of force due to belt elasticity per inch of belt width = $2eE/D$
D = roller diameter, in
E = modulus of elasticity of the belt (i.e., stress/strain before yield point)
e = belt strain, Δ/L_1
Δ = belt stretch ($L_0 - L_i$)
L_1 = tangent length of belt entering roller
L_0 = length of outer belt around roller between tangent points on adjacent rollers
L_i = length of inner belt around roller between tangent points on adjacent rollers.

The total pressure on the cake at any roller may be estimated by summing Eqs. (3.61) to (3.63), which yields Eq. (3.64):

$$\text{Total pressure on sludge cake (psi)} = \text{psi}_1 + \text{psi}_2 + \text{psi}_3 = \frac{2\,(F_1 + F_2 + F_3)}{D} \qquad (3.64)$$

Equation (3.64) can be used to estimate the total pressure on the sludge cake at each successive roller to ensure that there is a gradually increasing pressure. These equations also can be used to determine roller diameter and bearing size requirements. The results of these calculations can then be compared with the manufacturer's specifications. Example 3.25 illustrates the use of these equations for belt filter press design.

Example 3.25 A sludge dewatering consultant to the Lodwar County Water Reclamation Plant (see Example 3.24) has recommended that the facility employ a belt filter press to achieve effective dewatering. If the belt filter press is to employ a belt of 80 in in width, estimate the pressure on the thickened sludge cake at the first roller due only to the drive torque (F_1) if the belt speed is held constant at 10 ft/min. Assume that the first roller has a diameter of 30 in and that the drive horsepower is 0.015 hp/in of belt width.

solution Estimate the pressure on the sludge cake over the first roller due to the drive torque using Eq. (3.61):

$$\text{psi}_1 = \frac{1700 \cdot \text{hp}}{DV} = \frac{1700 \cdot (0.015\ \text{hp/in})}{30\ \text{in} \cdot 10\ \text{ft/min}} = 0.085\ \text{psi}$$

NOTE: To estimate the total pressure on the sludge cake over each roller, the pressure due to take-up tension and belt elasticity must be taken into account.

3.4.3.2 Belt filter press performance. In most cases, cationic polymers are used for conditioning sludge for belt filter press dewatering. To provide flexibility in system operation, several polymer feed points should be provided since sludge characteristics can change, and sometimes the sludge will dewater better if it has a longer contact time with the polymer and, in other cases, when the contact time is shorter.

In addition to the changing sludge characteristics, odors can be a problem with belt filter press operation. Odors may be controlled with the use of properly designed ventilation systems and/or the use of chemicals such as potassium permanganate or hydrogen peroxide. If chemicals are used to manage odors, they are normally fed directly into the suction side of the sludge transfer pump so that the pump

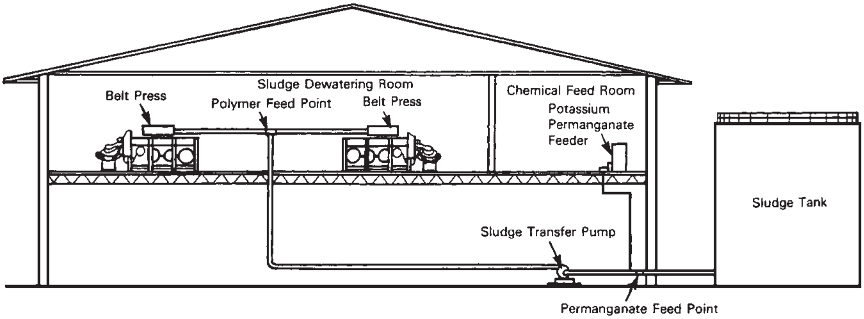


Figure 3.104 Belt filter press installation equipped with a chemical odor control system.

itself can act as a mixer. A typical chemical odor control system is illustrated in Fig. 3.104.

Belt speed is an important operational parameter that affects cake solids content, polymer dosage, solids recovery, and hydraulic capacity for the belt filter press system. In general, lower belt speeds result in a drier cake. At a given belt speed, increased polymer dosages result in a higher cake solids content, whereas, with an adequate polymer dose, solids recovery increases with lower belt speeds [48]. Finally, hydraulic capacity (i.e., sludge throughput) increases at higher belt speeds; however, the solids capture efficiency under these operational conditions will decrease.

Belt tension also has an effect on cake solids content, maximum solids loading, and solids capture efficiency. In general, a higher belt tension produces a drier cake but causes a lower solids capture. A drawback of using a higher belt tension is increased belt wear. The maximum belt tension that will not cause solids loss from the sides of the belts should be used [38,48].

Finally, failure of the chemical conditioning process to adjust to changing sludge characteristics can cause operational problems. If sludge is underconditioned, improper drainage occurs, resulting in either extrusion of inadequately drained solids from the compression section or uncontrolled overflow of sludge from the drainage section. Conversely, overconditioned sludge drains so rapidly that it does not distribute properly across the belt. Vanes and distribution weirs included in the gravity drainage section will help alleviate this problem.

3.4.4 Centrifugation

Centrifugal dewatering of sludge uses the force developed by a rotating cylindrical drum or bowl to achieve liquid-solids separation. The centrifuge is essentially a high-energy settling unit. The centrifugal acceleration force developed by a centrifuge can be estimated using Eq. (3.65):

$$G = \frac{(2\pi N)^2 R}{60} \tag{3.65}$$

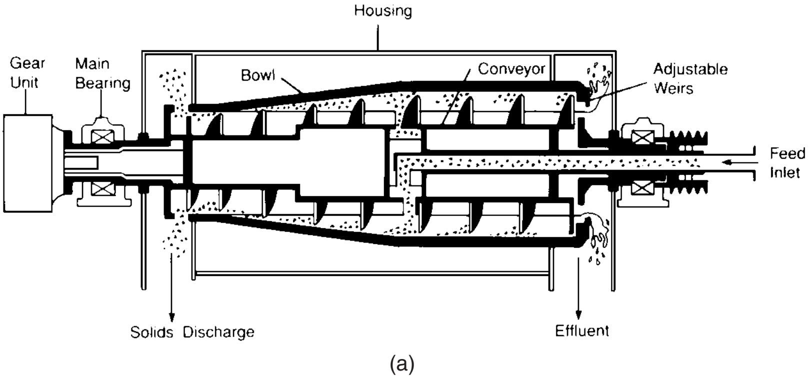
where G = acceleration force, N/m^2
N = rotational speed of centrifuge, rev/s
R = radius of rotating body of liquid, m

In the centrifugation process, density differences that exist between the sludge solids and liquid are used to separate the two media from one another at high rotational speeds. The solids cake and the liquid centrate are then discharged separately from the unit.

Although three types of centrifuges were described for municipal sludge thickening, e.g., imperforate basket centrifuge, solid-bowl centrifuge, and disk-nozzle centrifuge (see Sec. 3.1), the solid-bowl centrifuge has become the industry standard for municipal sludge dewatering. Moreover, because of its dewatering efficiency and ease of operation, the high-speed countercurrent solid-bowl centrifuge design has become the preferred solid-bowl design for continuous dewatering of municipal sludge, with other designs employed infrequently. Because of wide application of the countercurrent solid-bowl centrifuge in municipal sludge dewatering applications, its design and operational characteristics will be the focus of this section. Readers interested in other types of centrifuge systems applicable to sludge dewatering are referred to the following references [38,39,48].

3.4.4.1 Solid-bowl centrifuge. The countercurrent high-speed solid-bowl centrifuge consists of a rotating horizontal cylindrical bowl containing a screw-type conveyor or scroll that rotates also, but at a slightly lower or higher speed than the bowl. The rotating bowl or scroll is supported between two sets of bearings and, at one end, necks down to a conical section that acts as a dewatering beach. Sludge enters the solid-bowl centrifuge through a stationary feed pipe extending into the hollow shaft of the rotating conveyor and is distributed through ports in the hollow shaft into a pool within the rotating bowl (Fig. 3.105).

The helical-screw conveyor fits inside the bowl with a small clearance between its outer edge and the inner surface of the bowl. The conveyor rotates at a slightly lower or higher speed than the bowl. This difference in speed (measured in revolutions per minute) is known as the *differential speed* and allows the solids to be conveyed from the zone of the stationary feed pipe to the dewatering beach (Fig. 3.106). In the countercurrent operational mode, influent sludge is introduced



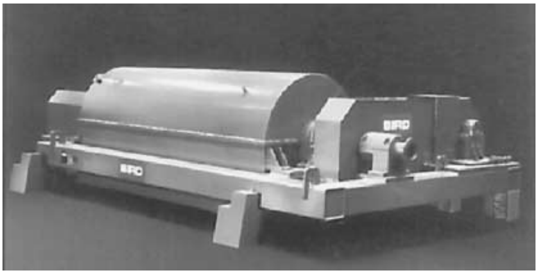
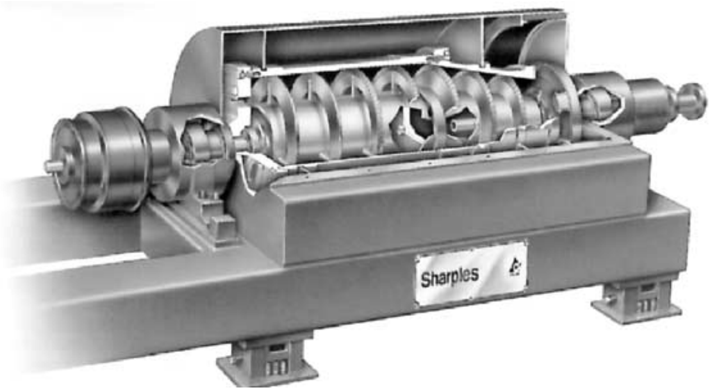


Figure 3.105 (*a*) Schematic diagram of a high-speed solid-bowl centrifuge. (*Courtesy of Baker Process Inc.*) (*b*) Photograph of a solid-bowl centrifuge. (*Courtesy of Baker Process Inc.*)

through the feed opening at or near the junction of the cylindrical and conical section of the bowl. The solids move to the conical end of the device, while the centrate flows to the opposite end.

Under the influence of centrifugal force, the sludge solids are pushed against the bowl wall. The solids are then moved gradually by the rotating conveyor along the bowl wall up the dewatering beach. From the dewatering beach, the dewatered solids are dropped into a sludge cake discharge hopper.

3.4.4.2 Centrifuge design and scale-up. The centrifuge has three basic functions that are not entirely compatible. The first is clarification or removal of solids from suspension. The second is consolidation of the solids particles against the bowl wall, and the third is conveying the dewatered solids out of the bowl. For a given magnitude of acceleration (i.e., G force), the effectiveness of clarification and solids concentration will be a function of the pool volume (a large volume favoring clarification and a small volume favoring solids concentration). The clarification function of any centrifuge is characterized by its sigma (Σ) value, which is described by Eq. (3.66):



(a)

(b)

Figure 3.106 (*a*) Schematic diagram of solid-bowl centrifuge components. (*Courtesy of Alfa Laval.*) (*b*) Series of solid-bowl centrifuges used at large municipal wastewater treatment plant. (*Courtesy of Alfa Laval.*)

$$\Sigma = 2\pi L\,(w^2/g)\,(0.75r_1^{\,2} + 0.25r_2^{\,2}) \qquad (3.66)$$

where Σ = theoretical hydraulic capacity (sigma factor), m^2

L = effective clarifying length of bowl (inlet to outlet), m

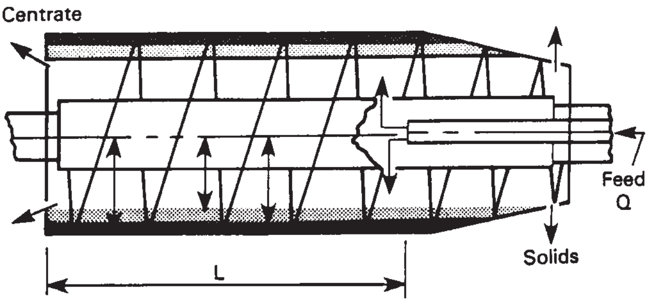
w = angular velocity of centrifuge bowl, rad/s

g = acceleration due to gravity, 9.81 m^2/s

r_2 = radius from the centrifuge centerline to the liquid surface in the centrifuge bowl, m

r_1 = radius from centrifuge centerline into the inside wall of the centrifuge bowl, m

The sigma value may be used for scaling from one size centrifuge to another size when clarification is limiting (i.e., the resulting sludge solids concentration is adequate) through the use of Eq. (3.67). Figure 3.107 illustrates use of the sigma factor in evaluating two centrifuge designs.



$$\Sigma = 2\pi L \frac{\omega^2}{q}(0.75r_1^2 + 0.25r_2^2)$$

Test Centrifuge A		Test Centrifuge B	
L	= 0.861 m	L	= 1.625 m
r_1	= 0.178 m	r_1	= 0.318 m
r_2	= 0.143 m	r_2	= 0.254 m
ω	= 366 rad/s	ω	= 262 rad/s
Σ_A	= 2,134 m²	Σ_B	= 6,574 m²
Q_A	= 9 m³/h	Q_B	= ?

$$Q_B = \frac{\Sigma_B}{\Sigma_A} Q_A$$

$$= 27.7 \text{ m}^3/\text{h}$$

Figure 3.107 Sigma scale-up procedure for solid-bowl centrifuges.

$$\frac{Q_A}{\Sigma_A} = \frac{Q_B}{\Sigma_B} \tag{3.67}$$

where Q_A = centrate flow rate through centrifuge *A*, m³/h
Σ_A = sigma value for centrifuge *A*
Q_B = centrate flow rate through centrifuge *B*, m³/h
Σ_B = sigma value for centrifuge *B*

When clarification of centrate is satisfactory and solids concentrating capacity (i.e., dewatering) is limiting, a similar approach is used. The solids concentrating capacity of a centrifuge is designated as the beta (β) factor, and it is used like sigma. The solids capacity of a solid-bowl centrifuge is reached at the point where the compacted sludge volume in the pool just interferes with the solids-liquid separation and the solids recovery declines. The total pool volume V and the total volume of sludge in the pool V_s are given by Eqs. (3.68) and (3.69), respectively.

$$V = \pi (r_1^2 - r_2^2) L \tag{3.68}$$

$$V_s = \pi (r_1^2 - r_3^2) L \tag{3.69}$$

where V = pool volume (centrate plus compacted sludge), m^3
V_s = compacted sludge volume, m^3
L = effective clarifying length of bowl (inlet to outlet), m
r_2 = radius from the centrifuge centerline to the liquid surface in the centrifuge bowl, m
r_1 = radius from centrifuge centerline into the inside wall of the centrifuge bowl, m
r_3 = radius from the centrifuge centerline to the sludge surface in the centrifuge bowl, m

If there is no slippage of sludge solids in the bowl, then the particle travel time can be calculated using Eq. (3.70):

$$T = \frac{L}{\Delta w S N} \tag{3.70}$$

where T = particle travel time, s
L = length of cylinder, m
Δw = differential speed, rad/s
S = spacing between conveyor blades, m
N = number of leads

The volume of solids contained in the centrifuge under steady-state conditions can be estimated using Eq. (3.71):

$$V_s = \left(\frac{Q_s}{Y_c}\right) T \qquad \text{or} \qquad V_s = \left(\frac{Q_s}{Y_c}\right) \cdot \left(\frac{L}{\Delta w S N}\right) \tag{3.71}$$

where Q_s = sludge solids flow into centrifuge, kg TSS/h
Y_c = specific weight of compacted sludge solids, kg/m^3
Q_s/Y_c = volumetric flow rate of solids discharged by the centrifuge, m^3/h

The ratio of sludge volume V_s to total pool volume V is given by Eq. (3.72). Equation (3.72) defines the beta factor (β), which is used to scale centrifuges that are limited by the system's ability to concentrate solids [Eq. (3.73)]. The relationship between the β factors and the scaling of centrifuges is illustrated in Example 3.26.

$$\begin{aligned}\frac{V_s}{V} &= \left(\frac{Q_s}{Y_c}\right) \frac{1}{2\Delta w S N r_1 (r_1 - r_2)} \\ &= \left(\frac{Q_s}{Y_c}\right) \frac{1}{\beta} \end{aligned} \tag{3.72}$$

where β = beta factor for scaling centrifuges that are solids capacity limited $[2\Delta w S N r_1(r_1 - r_2)]$.

$$\frac{Q_A}{\beta_A} = \frac{Q_B}{\beta_B} \tag{3.73}$$

where Q_A = sludge flow rate through centrifuge *A*, m^3/h
β_A = beta value value for centrifuge *A*
Q_B = sludge flow rate through centrifuge *B*, m^3/h
β_B = beta value for centrifuge *B*

Example 3.26 The Turkana County Sewage Treatment Works desires to purchase centrifugal dewatering equipment to replace its drying beds. To estimate performance of the full-scale centrifuge system, data from a laboratory-scale system have been furnished to the plant superintendent. If the 0.6-m laboratory centrifuge treats 4.5 m^3/h of thickened sludge, find the treatment capacity for a 3.0-m centrifuge system if at the desired operating conditions of the full-scale system r_1 is expected to be 0.38 m and r_2 is expected to be 0.24 m. Assume that the differential speed Δw, spacing between conveyor blades *S*, and number of leads *N* are identical in the laboratory- and full-scale systems. From dewatering tests, the laboratory scale system was found to be solids limited at the following operating conditions:

1. r_1: 0.18 m
2. r_2: 0.14 m
3. r_3: 0.16 m

solution

Step 1. Find the pool volume and compacted sludge volume for the laboratory-scale centrifuge:

$$\begin{aligned}\text{Pool volume } (m^3) &= \pi\,(r_1^2 - r_2^2)\,L \\ &= \pi\,(0.18\ m^2 - 0.14\ m^2)\ 0.6\ m \\ &= 0.067\ m^3\end{aligned}$$

$$\begin{aligned}\text{Compacted sludge volume } (m^3) &= \pi\,(r_1^2 - r_3^2)\,L \\ &= \pi\,(0.18\ m^2 - 0.16\ m^2)\ 0.6\ m \\ &= 0.0128\ m^3\end{aligned}$$

Step 2. Find the beta factor (β) for the laboratory-scale centrifuge using Eq. (3.72):

$$\frac{V_s}{V} = \frac{Q_A}{\beta_A} \qquad \text{or} \qquad \beta_A = \frac{Q_A V}{V_s} = \frac{4.5\ m^3/h \cdot 0.067\ m^3}{0.0128\ m^3} = 23.51\ m^3/h$$

Step 3. Find the term ΔwSN for the laboratory-scale system:

$$\beta_A = 2\Delta wSNr_1\,(r_1 - r_2)$$

$$23.51\ \text{m}^3/\text{h} = 2\Delta wSN\ 0.18\ \text{m}\ (0.18\ \text{m} - 0.14\ \text{m})$$

or

$$\Delta wSN = 1632.6$$

Step 4. Find the beta factor (β) for the full-scale centrifuge:

$$\begin{aligned}\beta_B &= 2\Delta wSNr_1\,(r_1 - r_2)\\ &= 2 \cdot 1632.6 \cdot 0.38\ \text{m}\ (0.38\ \text{m} - 0.24\ \text{m})\\ &= 173.71\ \text{m}^3/\text{h}\end{aligned}$$

Step 5. Find the treatment capacity for the full-scale centrifuge using Eq. (3.73):

$$\frac{Q_A}{\beta_A} = \frac{Q_B}{\beta_B}$$

or

$$Q_B = \frac{Q_A}{\beta_A}\beta_B = \frac{4.5\ \text{m}^3/\text{h}}{23.51\ \text{m}^3/\text{h}} \cdot \frac{173.71\ \text{m}^3}{\text{h}}$$

$$= 33.25\ \text{m}^3/\text{h}$$

Since the feed solids are split between the centrate and the dewatered cake during the centrifugation process, it is necessary to use a recovery formula to determine the efficiency of solids capture. This formula, which is based a mass-balance approach, was introduced during the discussion of solids thickening and is repeated here for convenience. The control volume for analysis of any centrifuge operation is depicted in Fig. 3.108.

Using the control volume in Fig. 3.108, the mass and volume balances can be described by Eqs. (3.74) and (3.75), respectively:

$$M_T = M_C + M_D \qquad (3.74)$$

$$Q_T = Q_C + Q_D \qquad (3.75)$$

where M_T = mass of thickened sludge entering the centrifuge, lb/h
M_C = mass of sludge leaving centrifuge in the centrate, lb/h
M_D = mass of sludge leaving centrifuge as dewatered sludge flow, lb/h

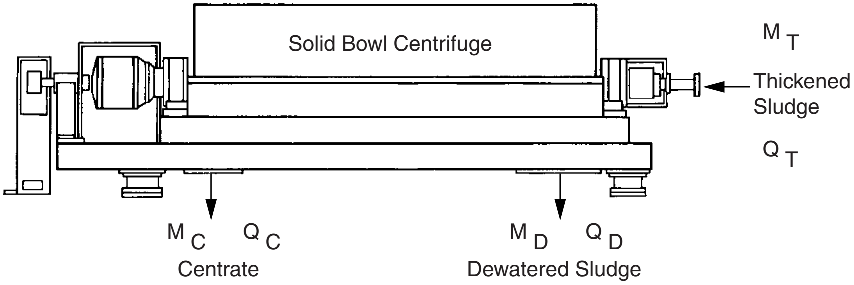


Figure 3.108 Control volume for analysis of centrifuge operation. M_T = mass of thickened sludge entering the centrifuge, lb/h; M_C = mass of sludge leaving centrifuge in the centrate, lb/h; M_D = mass of sludge leaving centrifuge as dewatered sludge flow, lb/h; Q_T = volumetric flow rate of thickened sludge entering centrifuge, gal/h; Q_C = volumetric flow rate of centrate, gal/h; Q_D = volumetric flow rate of dewatered sludge flow, gal/h.

Q_T = volumetric flow rate of thickened sludge entering centrifuge, gal/h
Q_C = volumetric flow rate of centrate, gal/h
Q_D = volumetric flow rate of dewatered sludge flow, gal/h

Since the specific gravity of each flow stream can be assumed to equal 1.0 without introducing significant error, the mass of solids in each flow can be estimated using Eq. (3.76):

$$\text{Mass flow rate of solids} = \text{volumetric flow rate} \cdot \text{concentration}$$

$$M_X = Q_X C_X \tag{3.76}$$

where M_X = mass flow rate of solids in flow X, lb/h
Q_X = volumetric flow rate of flow X, gal/h
C_X = concentration of solids in flow X, percentage, or mg/liter

Combining Eqs. (3.74), (3.75), and (3.76) results in Eqs. (3.77) and (3.78), which are the mass-balance equations that can be employed in analyzing the solids capture efficiency of any centrifuge system:

$$\frac{M_D}{M_T} = \frac{C_D}{C_T} \; \frac{C_T - C_C}{C_D - C_C} \tag{3.77}$$

$$\frac{Q_C}{Q_T} = \frac{C_D - C_T}{C_D - C_C} \tag{3.78}$$

where M_D/M_T = solids capture efficiency decimal
Q_C/Q_T = fraction of influent flow that appears as centrate (or return flow)

C_T = concentration of solids in influent thickened sludge flow, % or mg/liter

C_D = concentration of solids in effluent dewatered sludge flow, % or mg/liter

C_C = concentration of solids in centrate flow, % or mg/liter

Q_c = volumetric flow rate of centrate, gal/h

Q_T = volumetric flow rate of influent thickened sludge, gal/h

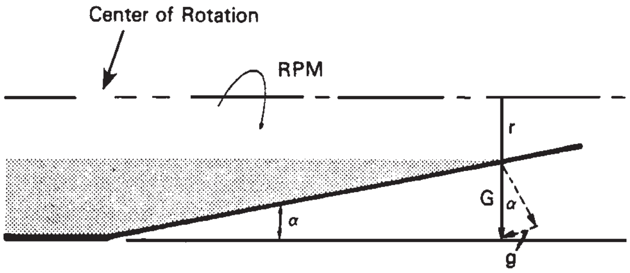
For a constant sludge feed concentration C_T, the solids capture efficiency (M_D/M_T) increases as the concentration of solids in the centrate C_C decreases. It should be noted that in all cases, bench- and/or pilot-scale tests are recommended to establish the acceptable range for process variables.

3.4.4.3 Centrifuge operation and control. The solid-bowl centrifuge configuration is cylindrical and conical in shape, although the proportions of each section will vary depending on manufacturer and application. In general, increasing the bowl diameter will increase both the capacity of the solids conveyance and clarification. An increase in the bowl length improves only clarification capacity, while the angle and length of the conical section, which acts as the dewatering beach, has an important effect on solids concentration of the dewatered sludge.

The beach angle is usually maintained at 8 to 10 degrees to help prevent slippage of the conveyed solids. As the solids emerge from the pool, the buoyancy effect is lost, and it becomes more difficult to convey fine, hydrous, and soft solids (e.g., waste-activated sludge). Shallow beach angles, deep pools, and conveyor design help to push the solids up to the beach and eliminate slippage problems. Although a shallow beach angle increases conveyor capacity and improves centrate quality, a wetter sludge cake is produced due to the loss of beach drainage area. Conversely, a steep beach angle produces a drier cake but at the expense of centrate quality and conveyor capacity. With a deep beach angle, there is a higher resistive force to conveyance of solids (Fig. 3.109).

In general, increasing the differential speed between the bowl and the scroll results in a wetter sludge cake, poorer solids recovery, and higher sludge throughput [48]. Conversely, reducing the differential speed produces a drier cake, increases solids capture, and decreases the overall sludge throughput.

Operating at too low a differential speed can cause the cake in front of the scroll conveyor blade to increase to a point that it has an impact on the clarified liquid (centrate) area. This may result in the skimming



G = Centrifugal Force

g = G sin α = Slippage Force

Figure 3.109 Effect of beach angle on sludge movement in a centrifuge.

of some fine particles from the top of the cake to the centrate, thus lowering the solids capture efficiency. Too low a differential speed also can result in plugging of the centrifuge if solids are removed at a slower rate than they are fed into the system [39,48].

Bowl speed is normally not varied once the unit is installed. Solid-bowl centrifuges operate at speeds equivalent to 600 to 3000 times the force of gravity and are categorized as low- or high-*G* centrifuges. Low-*G* units have operating speeds equivalent to 600 to 1800 *G* forces, and high-*G* units operate at 1800 to 3000 *G*s. The acceleration force is directly proportional to the bowl diameter and the square of the bowl speed. Therefore, the *G* force takes into account both the bowl speed and diameter and is a better method of describing the centrifuge than the speed alone.

There has been considerable debate over the benefits of a low-*G* versus a high-*G* centrifuge. Manufacturers of low-*G* centrifuges claim that their product consumes less energy, has a lower noise level, and requires less maintenance than high-*G* systems. Conversely, manufacturers of high-*G* centrifuges claim that their product requires less polymer and achieves a high solids throughput relative to low-*G* systems [39,48]. Resolution of whether or not low-*G* centrifuges have a lower total annual cost than high-*G* centrifuges can only be determined after side-by-side tests are conducted.

Finally, the ability to be used either for thickening or dewatering is an advantage of the solid-bowl centrifuge system. For example, a solid-bowl centrifuge can be used to thicken sludge ahead of a filter press, reducing chemical use and increasing solids throughput. During periods of downtime of the filter press, the solid-bowl centrifuge can be operated as a dewatering device. Another advantage of the solid-bowl centrifuge for larger plants is the ability to dewater a large feed solids flow rate. Larger centrifuges are capable of handling

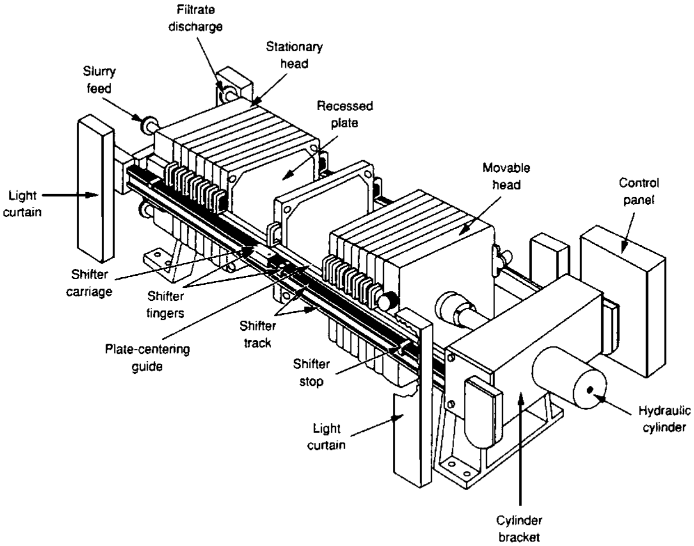
300 to 800 gal/min of influent sludge flow depending on sludge characteristics [39]. A solid-bowl centrifuge typically will dewater a 50:50 mixture of anaerobically digester primary and secondary sludge to a cake having a solids content in the range of 15 to 21 percent [48].

3.4.5 Pressure filtration

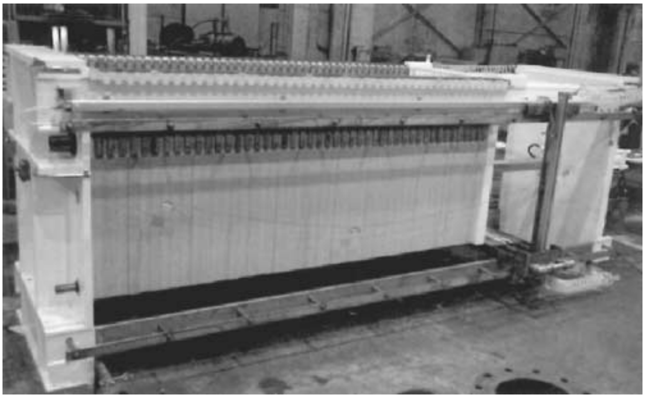
The two types of filter presses that are commonly available to dewater sludge are the fixed-volume recessed-plate filter press and the variable-volume recessed-plate filter press (also referred to as the *diaphragm filter press*). In general, recessed-plate filter presses consist of a series of plates, each equipped with a recessed section that forms the volume into which the sludge is pumped for dewatering. Filter media are placed against each wall that retain the sludge solids while permitting the passage of filtrate. Sludge is pumped under high pressure into the volume between the two plates. The filtrate passes through the cake and filter media and out of the press through special ports on the filtrate side of the media.

In the fixed-volume recessed-plate filter press, liquid sludge is pumped into a volume between two filter cloths held in place by a rigid framework (Fig. 3.110). Sludge pumping is at relatively high pressures (up to 250 psi, or 1730 kPa). Industry practice has historically separated the operation of recessed-plate filter presses into two categories: (1) low-pressure units (operation pressures of 50–125 psi, 350–864 kPa) and (2) high-pressure units (150–250 psi, 1040–1730 kPa). As a result of the high pressure, a substantial portion of the water in the feed sludge passes through the filter cloth and drains from the press. The production of filtrate eventually ceases, indicating that the void volume between the filter cloths is filled with sludge solids and residual moisture. Under normal operating conditions, the sludge pumps will continue to discharge thickened sludge to the filter press until the pressure within the recessed volume can be maintained at the pumping pressure for a prescribed length of time. Once this occurs, sludge pumping is stopped and the plates are squeezed together by the hydraulic system, resulting in the removal of additional moisture. The press is then opened to discharge the dewatered cake prior to initiation of a new cycle.

Periodic washing of the filter cloth is required because the pressure tends to bind sludge solids to the cloth. Although polymers are being employed increasingly for sludge conditioning prior to filter press dewatering, ferric chloride and lime addition are still common. If ferric chloride–lime conditioning is used in recessed-plate filter press dewatering, lime scale must be removed periodically through acid washing. The fixed-volume recessed-plate filter press typically will dewater a 50:50 mixture of digested primary and secondary sludge to



(a)



(b)

Figure 3.110 (*a*) Schematic diagram of a fixed-volume recessed filter plate. (*Courtesy of EIMCO Process Equipment Company.*) (*b*) Photograph of a fixed-volume recessed filter plate system. (*Courtesy of EIMCO Process Equipment Company.*)

a cake having a solids content in the range of 35 to 42 percent [48]. The solids fill and discharge processes for a fixed-volume recessed-plate filter press are illustrated in Fig. 3.111.

In a variable-volume recessed-plate or diaphragm filter press system, sludge is pumped into the press at a low pressure until the volume of the press has been filled with a loosely compacted cake. The sludge pumping is then stopped, and the diaphragm is inflated for a preset time. Although most of the water removal occurs when the

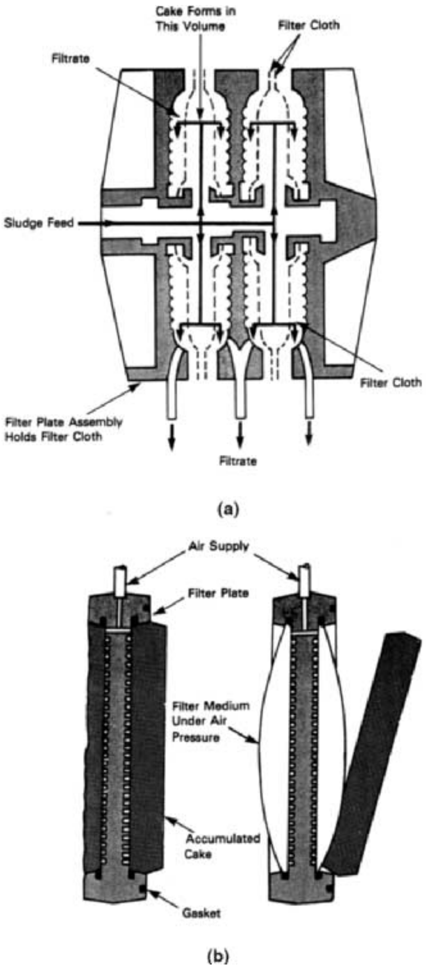


Figure 3.111 Schematic diagram of a fixed-volume recessed filter: (*a*) plate fill; (*b*) discharge mode.

sludge is being pumped into the press, a significant quantity of water is removed after the diaphragm is inflated (Fig. 3.112).

The diaphragm is inflated by pumping either air or water into the diaphragm at pressures in the range of 215 to 285 psi (1480–1965 kPa) [38,48]. After a preset time has elapsed, the diaphragm is deflated, and the press opens, allowing the cake to drop out. The filter cloth is washed periodically by permanent spray nozzles. A diaphragm press typically will dewater a 50:50 mixture of digested primary and secondary sludge to a cake having a solids content in the range of 38 to 47 percent.

The diaphragm press offers several advantages over the fixed-volume recessed-plate press. First, the diaphragm press produces a

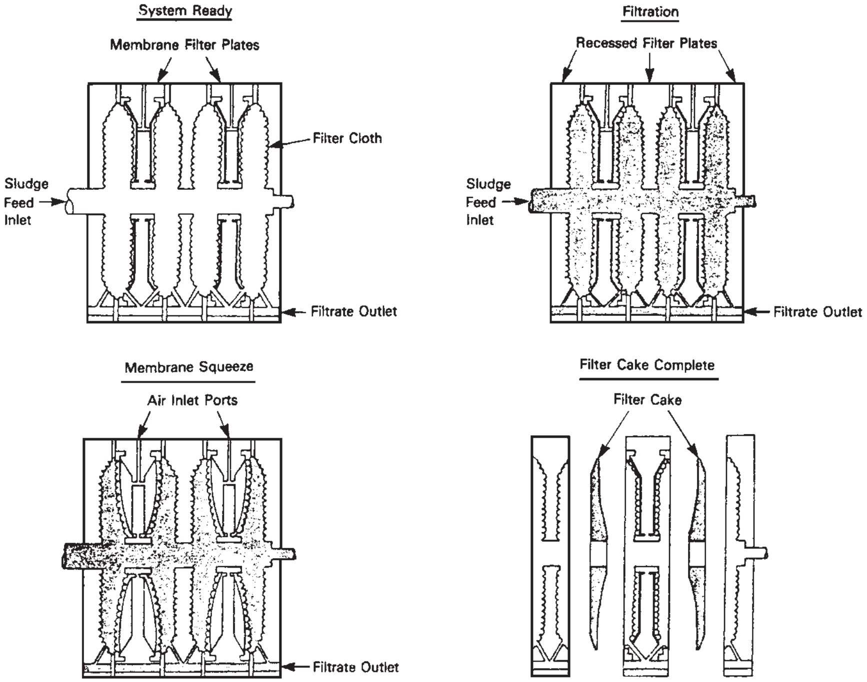


Figure 3.112 Schematic diagram of diaphragm filter plate fill and discharge modes.

drier sludge cake with a relatively uniform moisture content compared with the fixed-volume system. Second, the diaphragm press has an overall shorter cycle time and therefore a higher production throughput. The primary reason for the shorter cycle time for the variable-volume filter press is that the diaphragm press creates a more effective and uniform pressure on the sludge cake than what occurs when liquid sludge is pumped continuously into the chamber (as is the case with the fixed-volume press). Finally, the diaphragm press has a lower operation and maintenance requirement for the sludge feed pumps and has the ability to dewater marginally conditioned sludge [48]. The principal disadvantage of the diaphragm press is its higher initial cost, which can be as much as two to three times the cost of a fixed-volume recessed-plate system with the same daily throughput.

For both types of filter presses, system performance is affected by various operational parameters including the solids content in the feed sludge, chemical conditioning dosage, cake solids content, total cycle time, solids capture, and desired yield ($\mathrm{lb/ft^2 \cdot h}$ or $\mathrm{kg/m^2 \cdot h}$). To properly operate the filter press, the relationships between operational parameters must be thoroughly understood. For example, as the feed solids content increases, the required conditioning chemical dosage and total cycle time decrease, and the filter yield increases [38].

Control of filter presses may be manual, semiautomatic, or fully automatic. Labor requirements for operation will vary depending on the degree of instrumentation used for control. Despite automation, operator attention is normally required during the discharge cycle to ensure separation of the solids from the media.

3.4.5.1 Filter press design approach. The most important parameters for the design and operation of filter presses are (1) cake solids concentration, (2) throughput rate, and (3) recovery fraction (i.e., fraction of solids delivered to the unit that exits as sludge cake).

The final cake solids concentration achievable by the filter press for a particular sludge will have an impact on the cost of the downstream operations and often determine the need for additional upstream operations such as thickening. With filter presses, high solids contents in the cake are normally achievable if the cycle time is increased. An increased cycle time, however, means a reduced rate of throughput.

With regard to solids recovery, filter presses that do not recover a substantial quantity of solids can experience an increased need for medium washing and cause an accumulation of fines in some other part of the wastewater treatment plant. Solids losses above 2 to 3 percent of feed solids usually are traced to a torn medium or the result of sludge adhering to the medium that is subsequently washed off and recycled. Recovery of solids in excess of 95 percent is an important design objective and is necessary to prevent both the excessive recycling of solids and the potential negative impact on other aspects of plant operation.

To estimate the performance of a filter press, the ease of separating the water phase from the solids is normally estimated using the specific resistance and/or capillary suction time (CST) test. The specific resistance of sludge r is a measure of the filterability of sludge and is numerically equal to the pressure difference required to produce a rate of filtrate (or centrate) flow of unit viscosity through a unit weight of sludge cake. Specific resistance r can be estimated using the fundamental filtration equation [Eq. (3.79)]:

$$\frac{dV}{dt} = \frac{PA^2}{\mu\,(rcV + R_m A)} \tag{3.79}$$

where V = volume of filtrate
t = time of processing (also called *cycle time*)
P = pressure (or vacuum)

A = filtration area
μ = filtrate viscosity
r = specific resistance
R_m = initial resistance of filter medium
c = weight of solids per unit volume of filtrate

Equation (3.79) normally is not used directly to estimate specific resistance. Rather, Eq. (3.79) should be integrated over a cycle time and the terms rearranged to generate Eq. (3.80). It should be noted that Eq. (3.80) is a linear equation of t/V (cycle time/filtrate volume) versus V (filtrate volume). Figure 3.113 depicts a typical t/V versus V curve.

$$\frac{t}{V} = \left(\frac{\mu rc}{2PA^2}\right)V + \left(\frac{\mu R_m}{PA}\right) \tag{3.80}$$

If sufficient filtration data are available to plot the left-hand side of Eq. (3.80) i.e., t/V, versus the volume of filtrate produced, i.e., V, the numerical value of the slope b of the curve can be used to estimate the specific resistance r using Eq. (3.81):

$$r = \frac{2bPA^2}{\mu c} \tag{3.81}$$

where b = slope of t/V versus V line.

The pressure P, filter area A, and filtrate viscosity μ are known quantities that are established prior to the initiation of data collection. The only unknown term in Eq. (3.81) is the weight of solids per volume of filtrate c, which may be estimated after the data-collecting effort is completed using Eq. (3.82):

$$c = \frac{1}{C_i/(100 - C_i) - C_f/(100 - C_f)} \tag{3.82}$$

where c = weight of solids per unit volume of filtrate, g/ml
C_i = initial moisture content of conditioned sludge, %
C_f = final moisture content of sludge cake, %

Although the data required for plotting a t/V versus V curve [Eq. (3.80)] may be obtained from pilot- or full-scale dewatering tests, a more cost-effective approach to obtaining these data is through the use of the Buchner funnel laboratory filtration test [48]. Example 3.27 illustrates use of laboratory filtration test in evaluating the effectiveness of a polymer-conditioning system to improve a filter press operation.

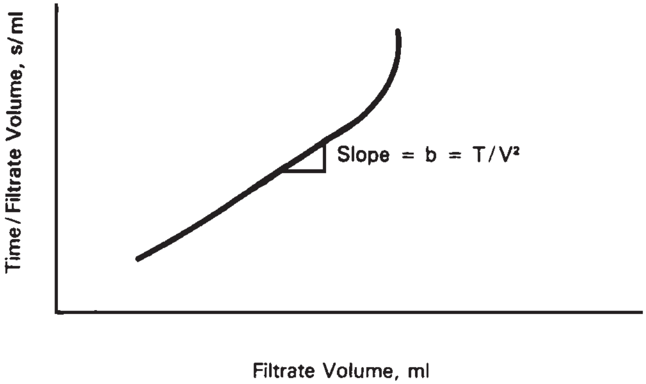


Figure 3.113 Typical behavior of t/V versus filtrate volume V dewatering data.

Example 3.27 Moore County Wastewater Treatment Facility is considering the use of a polymer to improve its filter-press dewatering operation. As the design engineer, you have narrowed down the choice to two polymers, each available from separate manufacturers. Since both manufacturers claim that their product yields superior sludge dewaterability, you have provided a contract laboratory a sample of your sludge and samples of each polymer and asked it to conduct a Buchner funnel test on the conditioned sludge. The following are the laboratory results from the two tests:

Polymer A:

Filter area: 200 cm^2

Pressure: 15 psi (1055 g_f/cm^2)

C_i (initial moisture content of conditioned sludge): 95 percent

C_f (final moisture content of dewatered sludge): 70 percent

μ (viscosity at 20°C): 0.01 poise [g/cm · s)]

Time	Filtrate volume V (ml)	t/V
5	70	0.071
10	110	0.091
15	145	0.103
30	170	0.176
40	180	0.220

NOTE: (lb/in^2) = (70.31 g_f/cm^2), where g_f = gram (force).

Polymer B:

Filter area: 200 cm^2

Pressure: 15 psi (1055 g_f/cm^2)

C_i (initial moisture content of conditioned sludge): 97 percent

C_f (final moisture content of dewatered sludge): 72 percent

μ (viscosity at 20°C): 0.01 poise [g/cm · s)]

Time	Filtrate volume V (ml)	t/V
5	60	0.083
10	95	0.105
15	115	0.130
30	130	0.231
40	140	0.286

Assuming that the dosage and costs of the polymers are similar, which polymer provides the better conditioning for filter press operation?

solution

Step 1. Plot t/V versus V data and obtain the slope b from the linear portion of the curves on the next page.

Step 2. Estimate the weight of solids per volume of filtrate c for each test using Eq. (3.82).

Polymer A:

$$c = \frac{1}{C_i/(100 - C_i) - C_f/(100 - C_f)} = \frac{1}{95/(100 - 95) - 70/(100 - 70)}$$

$$= 0.059 \text{ g/ml}$$

Polymer B:

$$c = \frac{1}{C_i/(100 - C_i) - C_f/(100 - C_f)} = \frac{1}{97/(100 - 97) - 72/(100 - 72)}$$

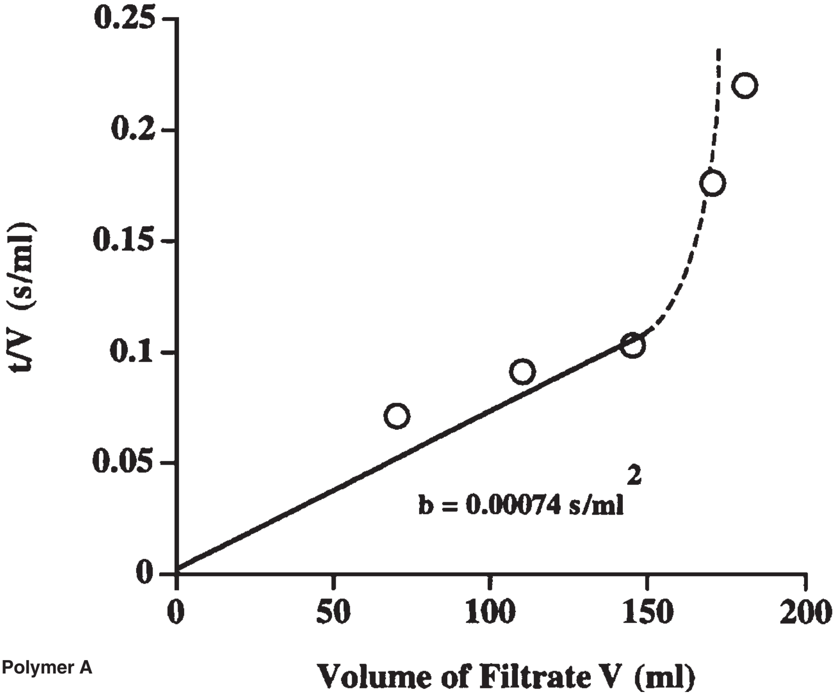
$$= 0.0336 \text{ g/ml}$$

Step 3. Estimate the specific resistance of each conditioned sample using Eq. (3.81).

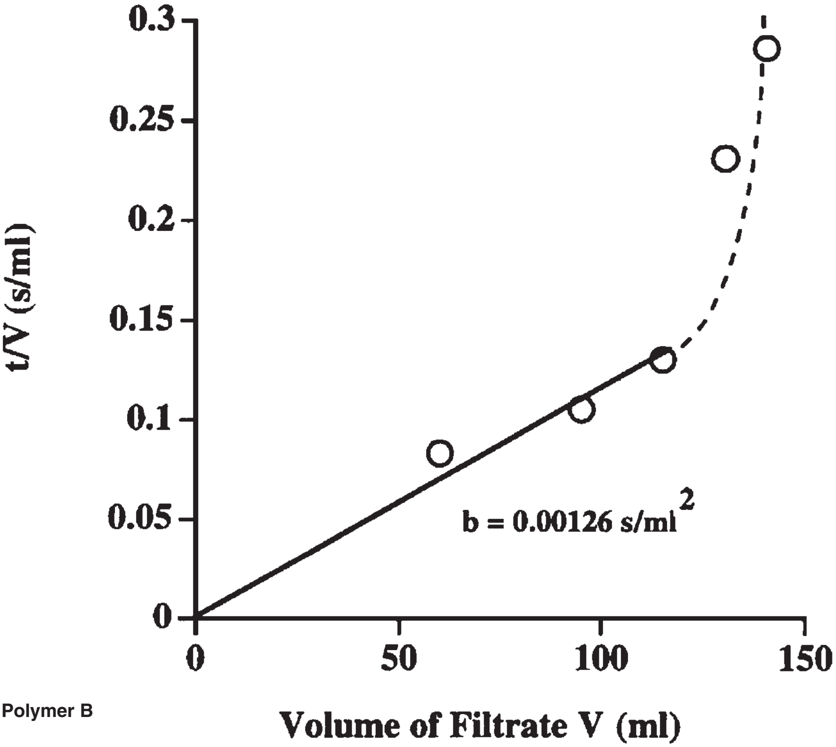
Polymer A:

$$r = \frac{2bPA^2}{\mu c} = \frac{2\,(0.00074 \text{ s/cm}^6)\,(1055 \text{ g}_\text{f}/\text{cm}^2)\,(200 \text{ cm}^2)^2}{(0.01 \text{ g/cm} \cdot \text{s})\,(0.059 \text{ g/cm}^3)}$$

$$= \frac{10.6 \cdot 10^7 \text{ g}_\text{f} \cdot \text{s}^2}{\text{gm}^2}$$



Polymer A



Polymer B

3.200

Polymer B:

$$r = \frac{2bPA^2}{\mu c} = \frac{2\,(0.00126\ \mathrm{s/cm^6})\,(1055\ \mathrm{g_f/cm^2})\,(200\ \mathrm{cm^2})^2}{(0.01\ \mathrm{g/cm\cdot s})\,(0.0336\ \mathrm{g/cm^3})}$$

$$= \frac{31.7\cdot 10^7\ \mathrm{g_f\cdot s^2}}{\mathrm{g\,m^2}}$$

Step 4. Choose the polymer with the better performance based on specific resistance estimates. Conditioned sludge yielding a lower specific resistance means that a particular conditioning chemical is more effective in facilitating dewatering of the sludge. Since polymer A has a lower specific resistance than polymer B, polymer A should be chosen by the wastewater treatment plant, assuming that all other factors are equal.

Although it is possible to use specific resistance data to estimate the dewatering equipment size and solids loading, this application of the data is not recommended because of the dissimilarities between the specific resistance test and the actual filtration operations. For example, the Buchner funnel uses filter paper as the filtration medium as opposed to the actual filter medium used in dewatering equipment. The process for sizing actual dewatering equipment should consist of working with equipment vendors, contacting other treatment plants using the same equipment, and evaluating the dewaterability of the sludge using pilot- or full-scale equipment.

3.4.6 Vacuum filtration

The most common approach for mechanically dewatering sludge in the United States up until the mid-1970s was vacuum filtration. For various reasons, including high energy costs and maintenance problems, this dewatering technology is not as popular as it once was. However, many units are still in operation today.

A vacuum filtration system consists of a horizontal cylindrical drum that rotates partially submerged (ca. 25 percent submergence) in a vat of sludge. The filter drum is divided into multiple compartments that are connected to a rotary valve. Bridge blocks in the valve divide the drum compartments into three distinct sections, including (1) a cake formation zone, (2) a cake drying zone, and (3) a cake discharge zone (Fig. 3.114).

As the drum rotates, each section is carried through the submergence or cake forming zone to the vacuum drying zone. The vacuum or cake drying zone represents from 40 to 60 percent of the drum surface and ends at the point where the internal vacuum is shut off. When the drum position reaches this location, the sludge cake and the

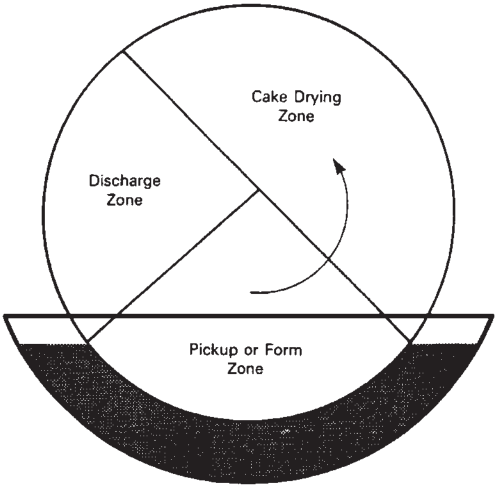


Figure 3.114 Operating zones in a rotary vacuum filter.

drum section will then enter the cake discharge zone, where the sludge cake is removed from the medium (Fig. 3.115).

There are basically two types of belt media used in vacuum filtration: (1) coil springs and (2) fiber cloth (woven cloth or metal belt). Figure 3.116*a* presents a cross-sectional view of a coil-spring, belt-type vacuum filter. This vacuum filter type uses two layers of stainless steel coils arranged around the drum. After the dewatering cycle, the two layers of springs leave the drum and are separated from each other. Through separation of the two layers, the cake is lifted off the lower layer of springs and can be discharged from the upper layer. After cake discharge, the coils are spray washed and returned to the drum just before the drum enters the sludge vat.

Figure 3.116*b* illustrates a cross section of a fiber-cloth, belt-type rotary vacuum filter. The filter medium in this system leaves the drum surface at the end of the drying zone and passes over a small-diameter discharge roller that facilitates cake discharge. Washing of the medium occurs after cake discharge and before the belt returns to the drum for another cycle.

The fiber-cloth belt-type vacuum filter normally has a small-diameter curved bar between the point where the belt leaves the drum and the discharge roll. This bar aids in maintaining belt dimensional stability. Corrective measures, such as addition of scraper blades, use of excess chemical conditioner, or addition of fly ash, are sometimes required to obtain cake release from the cloth medium. Corrective measures are particularly important at wastewater treatment plants that produce sludge that is greasy and/or contains a large percentage of waste-activated sludge [39].

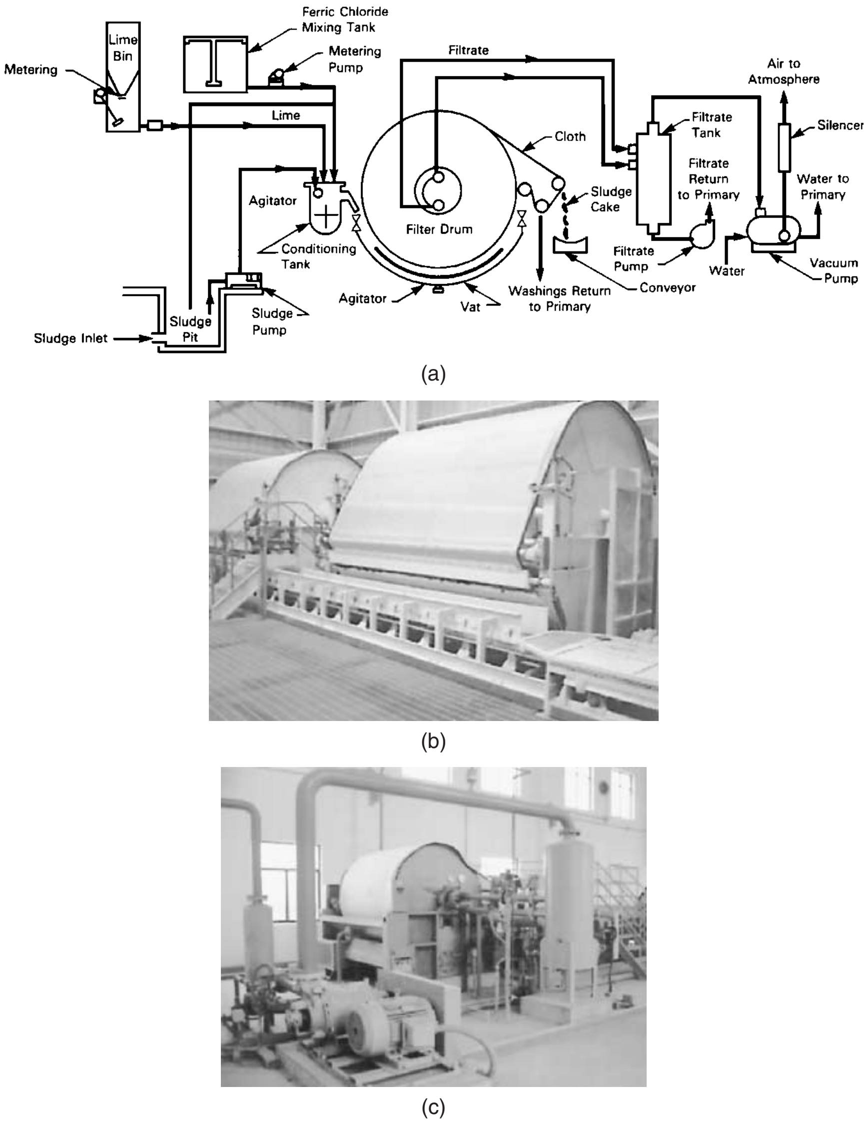


Figure 3.115 (*a*) Schematic diagram of a rotary vacuum filtration system. (*b*) Photograph of a large vacuum filtration system. (*Courtesy of WesTech Engineering.*) (*c*) Photograph of a small custom-built vacuum filtration system. (*Courtesy of EIMCO Process Equipment Co.*)

The performance of vacuum filters may be measured by several criteria including (1) yield, (2) efficiency of solids removal, and (3) cake characteristics. Yield, which is the most common measure of vacuum filter performance, is expressed in terms of mass of dry solids in the cake discharged from the filter per unit filter area per hour ($\text{lb/ft}^2 \cdot \text{h}$

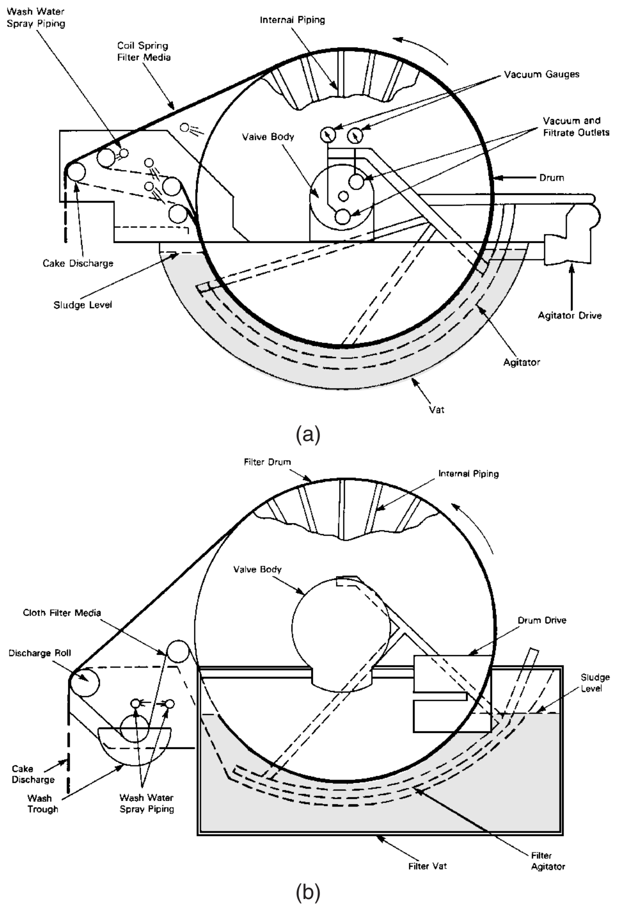


Figure 3.116 (*a*) Cross-sectional view of a coil-spring belt-type rotary vacuum filter. (*b*) Schematic diagram of a fiber-cloth belt-type rotary vacuum filter.

or $kg/m^2 \cdot h$). A typical range of vacuum filter yields for dewatering anaerobically digested primary and secondary sludge is 3.5 to 6.0 $lb/ft^2 \cdot h$ (17–29 $kg/m^2 \cdot h$) [48]. Vacuum filters typically produce a sludge cake with a solids content of between 15 and 20 percent. Despite their capability, vacuum filters have lost significant appeal for sludge dewatering. Some of the reasons cited for the reduced use of this dewatering technology are listed in Table 3.32.

3.4.7 Screw presses

Recent advances in mechanical dewatering systems include the development and application of screw presses. Advantages cited for the use

TABLE 3.32 Principal Disadvantages of Using Vacuum Filtration*

- Consumes a large amount of energy per pound of dewatered sludge produced.
- Vacuum pumps are noisy.
- Lime and ferric chloride conditioning can cause considerable maintenance/cleaning problems.
- The use of lime for conditioning can produce strong ammonia odors.
- Requires at least 3 percent solids content for influent sludge to achieve adequate cake formation and discharge.

*Adapted from refs. [33,38].

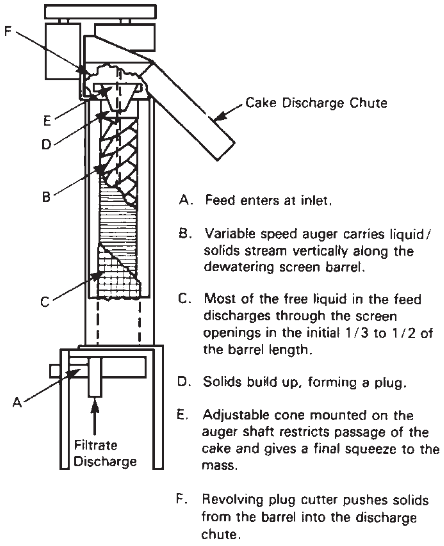


Figure 3.117 Vertical screw press dewatering system. (*Courtesy of Som-A-Press, Inc.*)

of screw presses include low operational and maintenance costs, low capital costs, and smaller area requirements than other types of mechanical dewatering systems [33,48]. Figure 3.117 is a schematic diagram of a vertical screw press system. The system consists of a vertical rotating screw enclosed by dual stainless steel screens. The screens and helical screw are encased in a stainless steel housing with a removable cover on each side.

Small perforations in the inner screen allow only water to escape. The outer screen has larger holes and easily collects the pressate, which sprays inside the housing and drains into a receptacle. Brushes are located along the edge of the screw to sweep the cake that accumulates on the screen, allowing a clear opening for the pressate to escape.

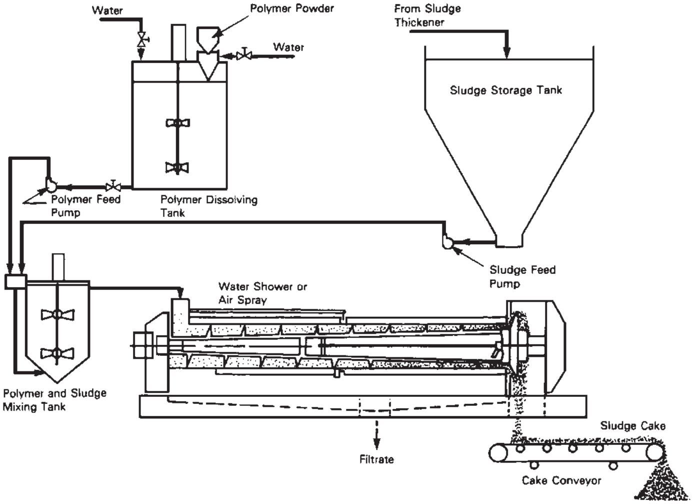


Figure 3.118 Horizontal screw press dewatering system. (*Courtesy of Hoilin Iron Works, Inc.*)

In the vertical screw press design, the feed sludge enters at the bottom of the screw press. As the pressate drains, the cake becomes progressively drier and is pushed to the top, where it is discharged. A backpressure system is located below the discharge chute and gives the cake a final squeeze before discharge.

In the horizontal screw press design, thickened and conditioned sludge receives a gradually increasing pressure as it progresses through the screw press. In the horizontal screw press design, the maximum pressure prior to sludge discharge typically exceeds 150 psi (Fig. 3.118). In trial tests using a sludge with an average solids content of 4.5 percent, this type of mechanical dewatering system is capable of producing a sludge cake having a solids content of 17.5 percent with an average polymer dose of 16 lb of polymer per ton of sludge solids (16 lb/ton) [48].

In some horizontal screw press systems, conditioning, thickening, and dewatering are combined in a single-unit operation. In the combination system depicted in Fig. 3.119, the magnitude and rate of pressure imparted on the sludge cake can be controlled by selection of different tapers on the screw shaft and by customizing the compression ratio for each application.

In all screw press operations, proper sludge conditioning is essential for producing a consistent dewatered product. Although a slow screw rotation will yield a drier cake, operation of the screw press at slow speeds will decrease the solids throughput. To establish the relationship between polymer dosage, solids throughput, and cake dryness, laboratory/pilot tests are recommended [48].

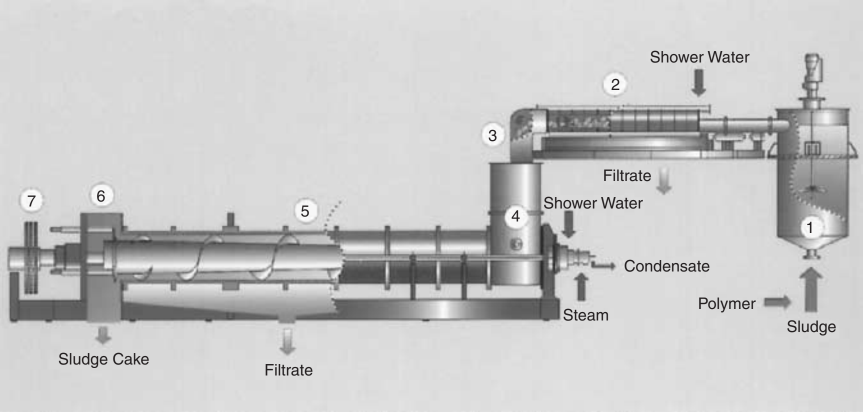


Figure 3.119 Horizontal screw press thickening/dewatering system. (1) Flocculation tank. (2) Rotary screen thickener (RST). (3) Chute for smooth sludge transition and even feed distribution. (4) Head box equipped with a level transmitter. (5) Screw press. (6) Cake discharge box. (7) Variable speed drive. (*Courtesy of EIMCO Process Equipment Company.*)

Although operation and maintenance costs are significantly lower for screw presses relative to other mechanical systems, these units typically have low sludge flow capacities (typically 50 gal/min or less), high polymer dosage requirements (typically 20 lb polymer per ton of solids or more), and low sludge cake solids content (12–25 percent) [48]. The low capacity of the presses may be a deterrent to some wastewater treatment plants because of the need to purchase multiple units.

3.4.8 Air-drying processes

Air-drying processes refer to those unit operations that dewater sludge through natural evaporation, gravity drainage, or induced drainage. Although there may be some mechanical assistance, such as turning and mixing the sludge, the movement of water is controlled primarily by natural forces. The prevalent air-drying processes currently employed for sludge dewatering include (1) sand drying beds, (2) paved drying beds, (3) vacuum-assisted drying beds, (4) wedgewire beds, and (5) sludge drying lagoons [30,38,48].

Air-drying processes are less complex, easier to operate, and require less operational energy than mechanical dewatering systems. However, the systems require larger land areas and typically are more labor-intensive. These factors suggest that these systems are best suited for small to moderately sized wastewater treatment plants (i.e., those treating less than 2 MG/day of wastewater). However, air-drying dewatering systems should be considered for larger facilities located in arid and semiarid climates when land is available.

3.4.9 Sand drying beds

Sand drying beds have been used successfully for sludge dewatering for many years and remain the principal method of dewatering used in the United States [5,30,38,71]. Dewatering on the sand drying bed occurs through both gravity drainage and evaporation. In areas of high precipitation, covered sand drying beds have been employed successfully for dewatering [30,38].

Sand drying beds consist of a sand layer having a minimum depth of 12 in (30 cm). In some cases, sand depths as deep as 18 in (46 cm) have been used to extend the life of the bed. To minimize erosion of sand, the beds are equipped with splash blocks positioned at every sludge entry point (Fig. 3.120). The sand rests on a gravel layer having a depth ranging from 8 to 18 in (20–46 cm) with gravel sizes ranging from 3 to 25 mm (0.1–1.0 in). The gravel supports the weight of sludge and sand as well as provides for the structural support for a system of underdrains. The underdrains, which are usually plastic pipe or clay tile laid with open joints, are used for collection of drainage water. The main underdrain pipes should be at least 4 in (10 cm) in diameter and should be laid with a slope of at least 1 percent to ensure rapid removal of moisture [33,48]. The collected drainage water is normally returned to the headworks of the wastewater treatment plant. With mechanical sludge removal, greater gravel depth may be required to structurally protect the underdrain network. A thinner layer of coarser stone, overlain by a permeable

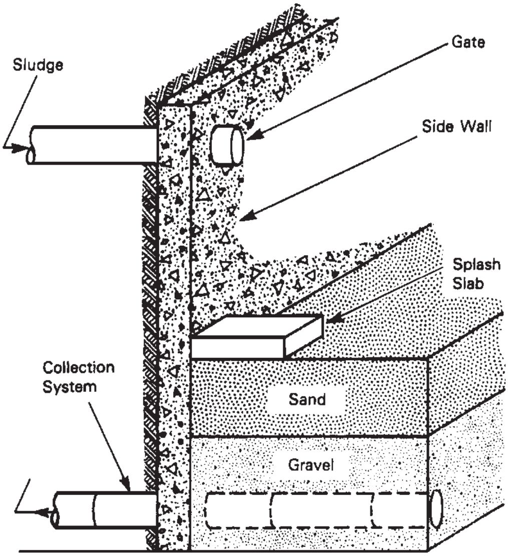


Figure 3.120 Schematic diagram of sand drying bed construction [48].

geotextile membrane, can be used in sand drying beds if sludge is removed using manual means or light equipment.

The sidewalls of sand drying beds may be constructed of reinforced concrete, treated timber planks, or concrete planks. The plank-type construction has the advantage of allowing adjustments of the total depth of the bed, a feature that is useful if modifications to the operation of the bed are made (e.g., freeze-assisted or reed-enhanced operation).

After sludge has dried to a desired moisture content, it is removed by manual or mechanical means. Since sand is unavoidably removed every time dewatered sludge is removed from the bed, new sand must be added. Preferred characteristics of the sand are (1) clean, hard particles (no clay, silt, or organic matter), (2) an effective size of 0.01–0.03 in (0.3–0.75 mm), and (3) a uniformity coefficient of less than 3.5 [30,48].

The design width of the bed depends primarily on the removal method. Small to moderate-sized facilities with hand or semimechanical dewatered sludge removal systems are about 6 m (20 ft) wide. Greater widths are required when mechanical sludge removal methods are employed. In these cases, sand beds having widths in excess of 60 m (200 ft) have been constructed. Uniform sludge distribution on the bed can be difficult, particularly when polymers are used for conditioning. In these cases, the bed length should not exceed 15 to 25 m (50–75 ft), and multiple distribution points should be incorporated into the design [38,48].

3.4.9.1 Sand drying bed design considerations. A sand drying bed's performance will depend on (1) required solids concentration in the dewatered sludge, (2) solids concentration in the applied sludge, (3) type of sludge applied (e.g., stabilized, thickened, conditioned, etc.), and (4) drainage and evaporation rates. The final sludge moisture content required before solids removal depends on the technical or regulatory requirements for final sludge disposal or use. If no special requirements apply, the sludge cake typically is "liftable" at about 25 percent solids and can be recovered without excessive sand loss.

The critical design parameter for construction of sand drying beds is the surface area required to achieve the final sludge solids content within a specified time. Currently accepted loading criteria for sand drying beds are provided in Table 3.33. The upper end of these ranges applies to warm, dry climates and to sludges that drain readily. The solids loading rates for digested primary plus waste-activated sludge discharged to uncovered drying beds range from 60 to 100 $kg/m^2 \cdot yr$ ($12–20\ lb/ft^2 \cdot yr$). These values can be increased to 85 to 140 $kg/m^2 \cdot yr$ ($17–28\ lb/ft^2 \cdot yr$) for covered drying beds.

TABLE 3.33 Loading Criteria for Anaerobically Digested, Nonconditioned Sludge on an Uncovered Sand Drying Bed*

Digested sludge type	Solids loading rate† ($kg/m^2 \cdot yr$)
Primary	100–200
Primary plus low-rate trickling filter	100–160
Primary plus waste-activated sludge	60–100

*Adapted from ref. [48].
†$lb/ft^2 \cdot yr = 0.2048\ kg/m^2 \cdot yr$.

The amount of water that can be removed by drainage in a sand drying bed is strongly influenced by the type of sludge applied. Drainage may account for 25 percent of the water removed when discharging anaerobically digested plus waste-activated sludge to the drying bed and 75 percent for a well-conditioned sludge. The drainage process typically is complete within 3 to 5 days following sludge loading. While the gravity drainage rate depends on the sludge type, the rate of evaporation is more a function of local climatic conditions and the sludge surface characteristics. Seasonal evaporation rates can be estimated from local pan or lake-evaporation values. However, since the crust that forms on the sludge surface inhibits evaporation, the pan evaporation values must be adjusted when used to design sand drying beds. By accounting for the feed sludge solids content, local climatic conditions and desired final dewatered sludge solids content, the minimum drying time for a sludge layer may be estimated using Eq. (3.83):

$$t_d = \frac{y_0 \left(1 - \dfrac{s_0}{s_f}\right)(1 - D)}{k_e E_v} \tag{3.83}$$

where t_d = dewatering time for a single sludge application, months
y_0 = initial depth of applied sludge layer, in (cm)
s_0 = initial dry solids content (%) as a decimal
s_f = final dry solids content (%) as a decimal
D = fraction of water removed by drainage (decimal)
E_v = average pan evaporation rate during time t_d, in/mo (cm/month)
k_e = reduction factor (% as a decimal) for sludge evaporation relative to free water surface (typically 0.6, or measured value)

If the sand drying bed is only operated for part of the year (e.g., no application of sludge during winter months), the maximum number of sludge applications that can be made to the sand drying bed during the year may be estimated by Eq. (3.84):

$$N = \frac{n_v}{t_d} = \frac{n_v k_e E_{vn}}{y_0 \left(1 - \dfrac{s_0}{s_f}\right)(1 - D)} \tag{3.84}$$

where N = number of sludge applications
n_v = length of operating season or an increment if evaporation is significantly different, months
t_d = dewatering time for a single sludge application, months
y_0 = initial depth of applied sludge layer, in (cm)
s_0 = initial dry solids content (%) as a decimal
s_f = final dry solids content (%) as a decimal
D = fraction of water removed by drainage (decimal)
E_{vn} = average pan evaporation rate during period n_v, in/mo (cm/mo)
k_e = reduction factor (% as a decimal) for sludge evaporation relative to free water surface (typically 0.6, or measured value)

Assuming a typical sludge specific gravity of 1.04, the design solids loading rate to the sand drying bed can be estimated by combining Eqs. (3.83) and (3.84) to produce Eq. (3.85):

$$L = 10.4 s_0 \frac{n_v k_e E_{vn}}{\left(1 - \dfrac{s_0}{s_f}\right)(1 - D)} \qquad \text{(SI unit)} \tag{3.85a}$$

$$L = 5.41 s_0 \frac{n_v k_e E_{vn}}{\left(1 - \dfrac{s_0}{s_f}\right)(1 - D)} \qquad \text{(US customary units)} \tag{3.85b}$$

where L = sludge loading during period n_v, kg/m^2 (lb/ft^2)
10.4 = metric conversion factor assuming a sludge with a specific gravity of 1.04
5.41 = standard U.S. conversion factor assuming a sludge with a specific gravity of 1.04
n_v = length of operating season or an increment if evaporation is significantly different, months
s_0 = initial dry solids content (%) as a decimal
s_f = final dry solids content (%) as a decimal
D = fraction of water removed by drainage, decimal
E_{vn} = average pan evaporation rate during period n_v, in/mo (cm/mo)
k_e = reduction factor (% as a decimal) for sludge evaporation relative to free water surface (typically 0.6, or measured value)

To estimate the minimum bed area, the annual sludge solids production rate at the treatment plant (lb/yr, kg/yr) is divided by the theoretical annual solids loading rate estimated from Eq. (3.85). Once the minimum bed area is estimated, the area is multiplied by a safety factor (typically 1.5 or greater) to account for system expansion and/or the use of the beds for temporary sludge storage [30]. The total bed area is typically subdivided into multiple cells. It is convenient to size the cells so that one or two cells are adequate to contain the total volume of the sludge digesters. The basic approach for sizing sand drying beds is illustrated in Example 3.28.

Example 3.28 The Odongo County Water Reclamation Plant is currently producing 27,000 gal/day of thickened sludge having a solids content of 3.5 percent. If a conventional sand drying bed is to be employed for dewatering, estimate the minimum area needed if climatic conditions will permit use of the drying beds for only 9 months per year. Assume that the following conditions apply:

1. Pan evaporation rate: 4.2 in/month
2. Moisture removed by drainage: 22 percent
3. Final sludge solids content: 26 percent
4. Reduction factor k_e: 0.6
5. Safety factor used for design: 1.8

solution

Step 1. Estimate the annual solids loading to the drying beds:

Annual solids loading (lb/year)

$$= 365 \text{ days/year} \cdot \left[\frac{27{,}000 \text{ gal/day}}{1 \cdot 10^6 \text{ gal/MG}} \cdot \frac{35{,}000 \text{ mg}}{\text{liter}} \cdot \frac{8.34 \text{ lb}}{\text{MG} \cdot (\text{mg/liter})}\right]$$

$$= 2{,}876{,}674 \text{ lb/year}$$

Step 2. Estimate the sludge loading during the drying period using Eq. (3.85*b*):

$$L = 5.41 s_o \frac{n_v K_e E_{vn}}{[1 - (s_o/S_f)]\,(1 - D)}$$

$$= 5.41 \cdot 0.035 \cdot \frac{9 \cdot 0.6 \cdot 4.2 \text{ in/month}}{[1 - (0.035/0.26)]\,(1 - 0.22)}$$

$$= 6.36 \text{ lb/ft}^2$$

Step 3. Find the minimum total area required per year for the sand drying bed using the estimated sludge loading:

$$\text{Minimum area (ft}^2\text{/year)} = \frac{\text{annual solids loading (lb/year)}}{\text{sludge loading (lb/ft}^2\text{)}}$$

$$= \frac{2{,}876{,}674.5 \text{ lb/year}}{6.36 \text{ lb/ft}^2}$$

$$= 452{,}153.7 \text{ ft/year (10.38 acres/year)}$$

Step 4. Estimate the total design area using a safety factor of 1.8:

$$\text{Total area (ft}^2\text{/year)} = 10.38 \text{ acres/year} \cdot 1.8 = 18.7 \text{ acres/year}$$

The final depth of the dewatered sludge cake may be estimated by using Eq. (3.86). Since thin layers of sludge will dry faster than a thick layer, it is sometimes tempting to use excessively low loading rates despite the fact that the annual solids loading rate is independent of the depth of the individual layers applied. Moreover, applying thin layers of sludge has several operational disadvantages, including (1) more frequent operation and maintenance, (2) greater sand loss from the bed, and (3) increased costs.

$$y_f = y_0 \frac{s_0}{s_f} \tag{3.86}$$

where y_f = final depth of applied sludge cake, in (cm)
y_0 = initial depth of applied sludge layer, in (cm)
s_0 = initial dry solids content, %
s_f = final dry solids content, %

3.4.9.2 Operation and maintenance. Sludge can be applied to each cell of a sand drying bed using a valved pipe (plug valves) or from an open channel with gate controls along the perimeter of the bed. The open channel is easier to clean but presents more operational problems in cold weather. The valves in a pipe network should be protected from freezing in cold climates because a frozen valve may prevent the moisture in the pipes from draining completely.

Feed sludge typically is applied in a layer having a thickness of 8 to 12 in (20–30 cm) across the entire bed and allowed to drain and dry until the resulting cake begins to crack (*ca.* 25 percent solids concentration). The final sludge depth will be a function of total bed area, the number of beds, climate, digester storage capacity (if available), and the desired cake solids content [30].

Bed maintenance involves the periodic replacement of sand lost during sludge removal, leveling and scarification of the sand surface prior to dosing, and removal of vegetation. Odors should not be a problem with well-stabilized sludges applied to the sand drying beds. However, if odor control is necessary, calcium hypochlorite, potassium permanganate, or ferrous chloride may be added to the sludge during sludge discharge to the drying bed.

Any chemical conditioners (e.g., polymers) added to sludge to enhance bed dewatering performance should be introduced continuously during the pumping operation at points in the system that will ensure proper mixing [48]. Multiple dosage points for polymers should be constructed into the system to optimize the conditioning process. These dosage points, at a minimum, should be located ahead of the pump suction, at the pump discharge, and ahead of the discharge point to the bed. It may not be necessary to use all dosage points, but the multiple array will allow optimization after operation commences.

Two modifications of the sand drying bed are (1) reed-enhanced sand drying bed and (2) freeze-assisted sand drying bed. The reed-enhanced sand drying bed uses the common reed *Phragmites* to improve dewatering, stabilization, and nutrient removal in digested sludge discharged to the system [30]. The plants facilitate oxygen transport to their root systems that harbor a diverse consortia of aerobic bacteria that metabolize the organic matter in the sludge while simultaneously mobilizing nutrients that promote vigorous plant growth. In addition to enhancing organic matter and nutrient removal, the plant's dense root system opens moisture drainage channels, while its leaf system is responsible for transpiring significant amounts of water [30].

Freezing-assisted drying beds take advantage of the structural changes that occur in frozen sludge to enhance moisture drainage. Freezing sludge changes both the structure of the sludge-water mixture and the characteristics of the solids particles. During the freezing process, solids tend to be compressed into large, discrete conglomerates surrounded by frozen water. When thawing commences, drainage occurs instantaneously through the large pores and channels created by the frozen water. The potential of using freeze-assisted dewatering depends on local climatic conditions [29].

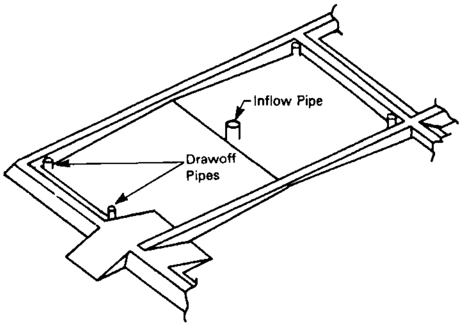
In addition to enhancing sludge dewatering, the reed-enhanced and freeze-assisted sand drying beds may be used for long-term storage of sludge solids. The design approach for both these systems is described in Chap. 5.

3.4.10 Paved drying beds

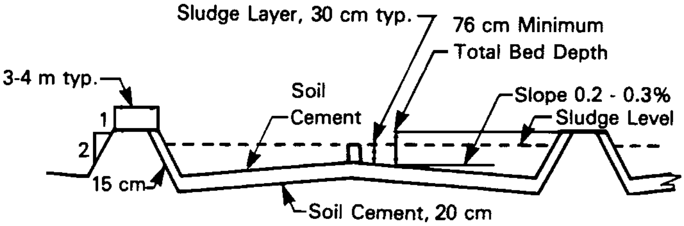
Until recently, paved sludge drying beds were constructed with an asphalt or concrete pavement placed over a porous gravel subbase. Unpaved areas (typically constructed sand drains) were positioned around the perimeter or along the center of the bed to collect and convey drainage water. The principal advantage of this approach to drying bed design was the ability to use relatively heavy equipment (e.g., front-end loaders) for dewatered sludge removal [38]. Reported field data have indicated that the pavement inhibits drainage, so the total bed area has to be significantly greater than that of conventional sand drying beds to achieve the same level of dewatering within a comparable time period [48]. Recent improvements in the paved sludge drying bed process include (1) use of decanting structures to remove supernatant (Fig. 3.121) and (2) use of a tractor-mounted horizontal auger (or other device) to regularly break up the surface crust that inhibits moisture evaporation (Fig. 3.122). The final evaporation-drying period will depend on the climatic conditions and the regular use of the auger/aerator equipment. Dewatered sludge having a solids content in the range of 40 to 50 percent can be achieved within 30 to 40 days in arid climates using these systems [48]. Moreover, it should be noted that since moisture evaporation and decanting are the principal water-removal processes, paved beds have been constructed with and without subsurface drains.

3.4.10.1 Design considerations. Paved beds may be used in any location, but since evaporation provides the major mechanism for water removal, they work best in warm, arid, and semiarid climates. Like sludge drying lagoons, the design solids loading for paved sludge drying beds is a function of the potential moisture evaporation and precipitation in the local area. In many paved drying bed operations, it is possible to decant sludge supernatant that can account for 20 to 30 percent of the water removal. Moreover, if the sludge has particularly good settling characteristics, it may be possible to use several fill and draw cycles to remove moisture prior to actively facilitating evaporative moisture removal by mechanical aeration [30,48].

The rate of evaporation may be determined through small-scale pilot studies, or it can be estimated to be a fraction of the pan evaporation rate in the local area. To estimate the mass of water required to be removed from the sludge to achieve a desired final moisture content, the initial water content and amount of water removed through the decantation process must be estimated using Eqs. (3.87) and (3.88), respectively:



(a)



Cross-Section View

(b)

(c)

Figure 3.121 (*a*) Schematic diagram of a paved drying bed designed for decanting and enhanced evaporation, plan view. (*b*) Schematic diagram of a paved drying bed designed for decanting and enhanced evaporation, side view. (*c*) Photograph of a paved drying bed designed for decanting and enhanced evaporation.

$$W_0 = 1.04S\left(\frac{1 - s_0}{s_0}\right) \tag{3.87}$$

where W_0 = total water content in applied sludge, lb/year (kg/yr)

1.04 = assumed specific gravity of the sludge

S = annual sludge production, dry solids, lb (kg)

s_0 = dry solids in applied sludge, percent as a decimal

Figure 3.122 A tractor-mounted auger/aerator.

$$W_D = 1.04S\left(\frac{1 - s_d}{s_d}\right) \tag{3.88}$$

where W_D = total water remaining after decantation, lb/yr (kg/yr)
1.04 = assumed specific gravity of the sludge
S = annual sludge production, dry solids, lb (kg)
s_d = dry solids in sludge after decantation, percent as a decimal

Since paved sludge drying beds typically are uncovered, precipitation must be taken into account when evaluating the evaporation rate required to achieve a final desired sludge moisture content. Equations (3.89*a*) and (3.89*b*) may be used to estimate the required evaporative moisture removal rates given an annual precipitation rate (ft/year, m/year):

$$W_E = W_D - 1.04S\left(\frac{1 - s_e}{s_e}\right) + PA1000 \qquad \text{(SI units)} \tag{3.89a}$$

$$W_E = W_D - 1.04S\left(\frac{1 - s_e}{s_e}\right) + PA62.4 \qquad \text{(U.S. customary units)} \tag{3.89b}$$

where W_E = total water to be evaporated after decantation, lb/yr (kg/yr)
1.04 = assumed specific gravity of the sludge
S = annual sludge production, dry solids, lb (kg)
s_e = dry solids required in sludge after evaporation, percent as a decimal

P = annual precipitation, ft (m)
A = bed area, ft^2 (m^2)
1000 = density of water in SI units (1000 kg/m^3)
62.4 = density of water in U.S. customary units (62.4 lb/ft^3)

NOTE: If the paved drying bed does not allow for decantation, use W_0 rather than W_D in Eqs. (3.89*a*) and (3.89*b*).

The area-specific evaporation rate for sludge in a given location can be estimated using Eqs. (3.90*a*) and (3.90*b*). It should be noted that the area-specific evaporation rate for sludge assumes that the evaporation rate is a fraction of the pan evaporation rate.

$$R_e = 1000 k_e E_v \qquad \text{(SI units)} \qquad (3.90a)$$

$$R_e = 62.4 k_e E_v \qquad \text{(U.S. customary units)} \qquad (3.90b)$$

where R_e = evaporation potential for sludge on a mixed and aerated paved bed, kg/m^2 · yr (lb/ft^2 · yr)
k_e = reduction factor for sludge evaporation versus free water surface, percent as a decimal (0.6 is typical)
E_v = free water pan evaporation rate, m/yr (ft/yr)

Dividing the required moisture-removal rate by the area-specific evaporation rate allows estimation of the minimum paved drying bed area to achieve a final sludge moisture content [Eqs. (3.91*a*) and (3.91*b*)].

$$\text{Area (m}^2) = \frac{1.04\text{S}\left(\dfrac{1-s_d}{s_d} - \dfrac{1-s_e}{s_e}\right) + PA1000}{R_e} \qquad \text{(SI units)} \qquad (3.91a)$$

$$\text{Area (m}^2) = \frac{1.04S\left(\dfrac{1-s_d}{s_d} - \dfrac{1-s_e}{s_e}\right) + PA62.4}{R_e} \qquad \text{(U.S. customary units)} \qquad (3.91b)$$

where A = bed area, ft^2 (m^2)
R_e = evaporation potential for sludge on a mixed and aerated paved bed, kg/m^2 · yr (lb/ft^2 · yr)
1.04 = assumed specific gravity of the sludge
S = annual sludge production, dry solids, lb (kg)
s_d = dry solids in sludge after decantation, percent as a decimal
s_e = dry solids required in sludge after evaporation, percent as a decimal
P = annual precipitation, ft (m)

It should be noted that since sludge drying lagoons employ the same moisture-removal mechanisms as paved drying beds, the present design equations may be employed for estimating the minimum required area for these systems as well.

Example 3.29 An anaerobically digested sludge having 2.5 percent solids content is to be dewatered to 30 percent solids using a paved drying bed. Estimate the required bed area if the solids content of the sludge after decanting has been estimated at 10 percent and the annual sludge production rate at the facilities is estimated at 3.4 million gal/year. Assume that the annual precipitation evaporation rates are 2.2 and 3.6 ft/year, respectively, and that the reduction factor for sludge evaporation k_e has been measured to be 0.8.

solution

Step 1. Estimate the pounds of sludge solids generated each year:

Sludge production (lb/year)

$$= 25{,}000 \text{ mg/liter} \cdot 3.4 \text{ MG/year} \cdot 8.34 \text{ lb/ (mg/liter)} \cdot \text{MG}$$

$$= 708{,}900 \text{ lb/year}$$

Step 2. Estimate the sludge moisture evaporation rate R_e using Eq. (3.90*b*):

$$R_e = 62.4 k_e E_v$$

$$= 62.4 \text{ lb/ft}^3 \cdot 0.8 \cdot 3.6 \text{ ft/year} = 179.7 \text{ lb/ft}^2 \cdot \text{year}$$

Step 3. Using Eq. (3.91*b*), estimate the minimum required bed area:

$$\text{Area (ft}^2) = \frac{1.04S\left(\dfrac{1-s_d}{s_d} - \dfrac{1-s_e}{s_e}\right) + PA62.4}{R_e}$$

$$= \frac{1.04\,\dfrac{708{,}900 \text{ lb}}{\text{y}}\left(\dfrac{1-0.1}{0.1} - \dfrac{1-0.3}{0.3}\right) + \dfrac{2.2 \text{ ft}}{\text{yr}}A62.4}{\dfrac{179.7 \text{ lb}}{\text{ft}^2 \cdot \text{yr}}}$$

$$= 27{,}351.4 \text{ ft}^2 + 0.764A \text{ (ft}^2)$$

$$= 115{,}866.4 \text{ ft}^2 \text{ (2.65 acres)}$$

Once the minimum bed area has been determined, sludge storage capacity can be incorporated into the design by multiplying the necessary dewatering area by a safety factor (typically 1.5) to estimate the total design area. The total design bed area should be divided into at least three beds for all but the smallest operations to provide operational flexibility [48]. Month-to-month analyses of weather records will determine the optimum number of beds required.

With respect to drying bed physical layout, a long rectangular configuration improves operational efficiency by reducing the time required for turning the auger/aerator vehicle. A number of inlet and decantation structures are also possible. The minimum total depth of the bed should not be less than 2.6 ft (0.8 m) to provide sufficient freeboard above the 12-in (30-cm) sludge layer. In some systems, up to 3 ft (1 m) of liquid sludge is applied in the initial layer, with the freeboard increased proportionately [48].

The major operational steps for paved sludge drying beds include (1) sludge application, (2) decantation, (3) mixing/aeration, and (4) sludge removal. Depending on the size of the operation and the time of the year, the sludge should be augered several times a week to maintain optimal evaporating conditions. Maintenance requirements include routine care of the auger/aeration equipment, the sludge pumps and piping network, the decantation piping, and the bed and dikes. If the site experiences freezing weather in the winter months, the valves and pumps in the system need to be protected and checked periodically [30,48].

3.4.11 Vacuum-assisted drying beds

Vacuum-assisted drying beds are characterized by a vacuum applied to the underside of rigid, porous medium on which chemically conditioned sludge has been placed. The vacuum enhances the dewatering process by drawing significant quantities of free water, while sludge solids are retained on the bed surface, forming a cake of uniform thickness. A typical size for a vacuum-assisted drying bed is 20 ft by 40 ft (6 m by 12 m). A system of this size will typically be equipped with a 1-hp (0.7-kW) vacuum pump. Figure 3.123 is a schematic diagram of the basic vacuum-assisted drying bed design.

The porous medium filter plate is a fabricated material consisting of a thin carborundum plate overlying a layer of sized aggregate that is held together by epoxy (Fig. 3.124). The medium filter plates are supplied in sheets that are caulked together after they are placed on the aggregate filled plenum. Caulking is also used around the periphery of the bed to maintain the vacuum seal.

3.4.11.1 Design considerations. The design parameters for vacuum-assisted drying bed design are the annual sludge production rate and the number of cycles that can be conducted during a typical work week. To ensure reliability, the design engineer should consider adding extra capacity to the system by increasing the size and/or

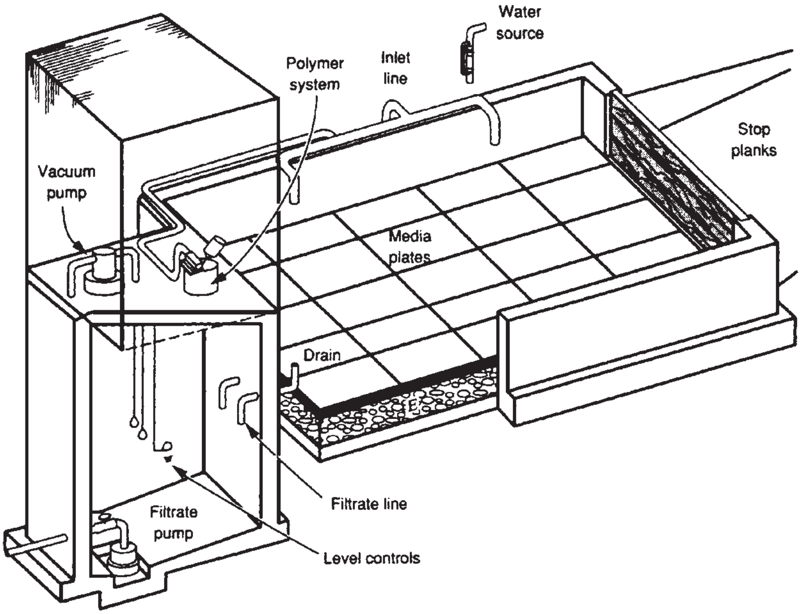


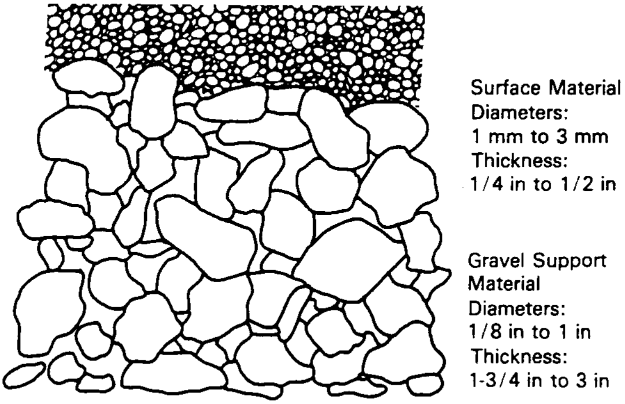
Figure 3.123 Schematic diagram of a vacuum-assisted drying bed.

number of beds [47,48]. A two-bed system is the minimum standard. However, if the sludge production rate exceeds 1 dry ton of solids per day, a three-bed system is recommended.

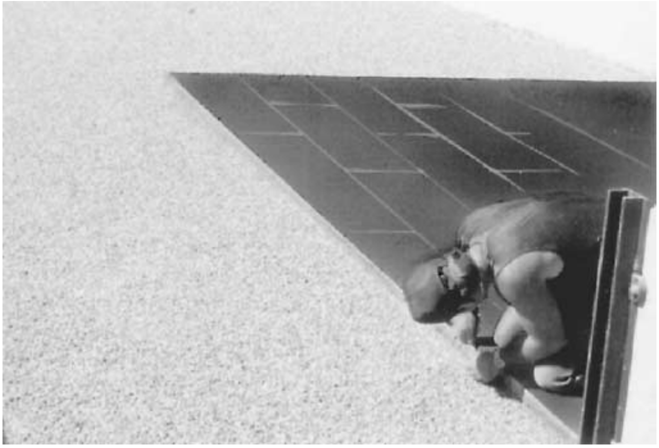
A properly sized three-bed system, using a 24-hour total cycle time, would use two of the three beds for dewatering each operating day, with the third bed remaining idle. Each bed in such a system should be sized to dewater, at a minimum, 70 percent of the average daily sludge solids discharged from the wastewater treatment plant (Fig. 3.125). This design allows the dewatering system to be operated 5 days per week and still provide for the dewatering of 7 days' accumulation of thickened sludge.

A typical solids loading rate to the vacuum-assisted sludge drying bed is approximately 2 $\mathrm{lb/ft^2 \cdot cycle}$ (10 $\mathrm{kg/m^2 \cdot cycle}$). Adjustments based on the expected efficiency of the operation should be considered by the design engineer and the solids loading rate adjusted accordingly. Table 3.34 describes the process steps that comprise a typical operational cycle for a vacuum-assisted drying bed.

Selection of the optimal polymer and dosage, the aging time, and the effectiveness of mixing and polymer dosing control are variables subject to various degrees of operator control. All these factors strongly affect dewatering performance.



(a)



(b)

Figure 3.124 (*a*) Schematic diagram of filter plate material. (*b*) Photograph of worker installing new vacuum filter surface. (*Courtesy of U.S. Environmental Products, Inc.*)

Plate cleaning is critical in the operation of vacuum-assisted drying beds. If not performed regularly and properly, the medium plates will clog. The drying bed design should incorporate (1) proper sizing and location of drains, (2) sufficient water pressure, and (3) selection of a satisfactory hose nozzle. The medium plates eventually will show some signs of decreased permeability, even with good maintenance, due to the accumulation of oils and greases or other substrates. Special cleaning measures are then required to restore system operation. Some of these cleaning measures include the application of (1) high-pressure hot-water cleaning, (2) commercial-grade hydrochloric

Figure 3.125 A vacuum-assisted drying bed.

TABLE 3.34 Process Steps Comprising Typical Operational Cycle for a Vacuum-Assisted Sludge Drying Bed*

Step 1.	At the start of the cycle, vacuum pumps are off, the bed closure is in place, the filtrate pumps are on automatic, sufficient polymer is mixed or otherwise available, and drains in the bed are covered and sealed.
Step 2.	Valves on the sludge feed line are opened, with the polymer feed pump also operational.
Step 3.	When the medium plates become completely covered with a well-flocculated sludge, the filtrate valves are opened to allow gravity drainage to begin. If a good separation of sludge solids and supernatant occurs, it is possible to decant prior to the opening of the filtrate valves.
Step 4.	When the desired volume of sludge has been applied to the bed, the polymer and sludge feed pumps are stopped.
Step 5.	Gravity drainage is allowed to continue until the operator decides that the rate of filtrate collection is too slow. The time for gravity drainage may range from 30 minutes to several hours after sludge application is completed.
Step 6.	The operator starts the vacuum cycle at the end of the gravity drainage period. The vacuum sequence proceeds in discrete steps beginning at 2 to 3 in (5–8 cm) of mercury vacuum for approximately 1 hour. The vacuum is then increased to between 5 and 6 in (13 and 15 cm) of mercury followed by a third step to between 10 and 12 in (25 and 30 cm) of mercury vacuum. The final vacuum level is maintained until the sludge has dried sufficiently to crack. Figure 3.126 illustrates "cracked" (i.e., dewatered) sludge being removed by a small mechanical loader.
Step 7.	The bed closure system, if used, is removed to allow access for sludge removal. Typically, a small tractor with a front-loading bucket is used at all but the smallest installations.
Step 8.	Remaining sludge is removed manually followed by manual rinsing of the bed with water. The plates must be cleaned carefully between each cycle if progressive loss of plate permeability is to be avoided.

*Adapted from ref. [47].

Figure 3.126 Photograph of “cracked” (i.e., dewatered) sludge being removed from a vacuum-assisted drying bed. (*Courtesy of U.S. Environmental Products, Inc.*)

acid (1% concentration), (3) trisodium phosphate (0.25–0.5%), (4) calcium or sodium hypochlorite (1%), or (5) enzyme-based cleaners.

3.4.11.2 Vacuum-assisted drying bed performance. Vacuum-assisted drying beds are a proven sludge dewatering technology. Table 3.35 provides performance data on 13 operational systems treating various types of sludge. The cycle time for each system is 24 hours, including cake removal and plate washing. The median polymer (liquid emulsion type) dosage was maintained at 20 lb/ton (9 kg/mt) dry solids.

3.4.12 Wedgewater sludge drying beds

Wedgewater sludge drying beds use a wedge-wire panel medium placed in an open concrete basin. The principal types of panel media available are stainless steel and polyurethane containing wedge-shaped slots approximately 0.01 in (0.25 mm) in width [48]. Both types of media can support a small front-end loader when properly installed. The interlocking modules create a shallow drainage plenum beneath the medium surface. To avoid solids accumulation in the plenum, the floor of the concrete beds should be sloped to ensure positive drainage. Figure 3.127 provides a schematic diagram of a wedgewater drying bed.

TABLE 3.35 Performance Data for Vacuum-Assisted Sludge Drying Beds*

Sludge type	Solids loading ($kg/m^2 \cdot$ cycle)	Total solids: Sludge feed (%)	Total solids: Sludge cake (%)
Unthickened sludge from oxidation ditch	3.18	0.8	9–12
Aerobically digested and thickened WAS	9.18	1.5–3.0	14–18
Lime-conditioned mixture of primary and thickened WAS	37.84	8–10	30–35

*Adapted from ref. [48].

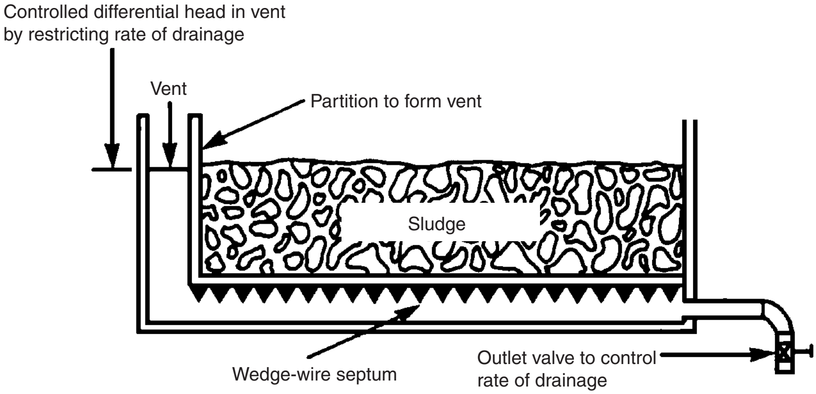


Figure 3.127 Schematic diagram of a wedge-wire drying bed.

Existing sand drying beds can be retrofitted for the wedgewater drying process, or a new concrete basin can be constructed. Small systems sometimes use large tilting metal trays. In this case, when the sludge is ready for removal, the whole bed is tilted to a steep angle, and the sludge cake slides out.

Prior to introducing sludge, the valve controlling removal of drainage water from the wedgewater drying bed is closed, and the bed is filled with water to a depth of about 0.4 in (1 cm) above the medium surface. Polymer-conditioned sludge is then applied to the bed, and the initial drainage rate is controlled by the rate of water removal from the volume beneath the medium. During the initial phase, the drain valve is closed so that the water and sludge stand on the bed. The valve is then opened to control the drainage rate for up to 2 hours. After the controlled drainage phase, the sludge is allowed to further dewater by natural drainage for up to an additional 24 to 48 hours. When the resulting sludge cake begins to crack, the sludge may be removed by a front-end loader.

3.4.12.1 Design considerations. Since wedgewater systems are proprietary devices, loading criteria are developed in conjunction with the manufacturer. However, typical solids loading rates are 0.4 to 1.0 $\text{lb/ft}^2 \cdot \text{cycle}$ ($2\text{–}5\ \text{kg/m}^2 \cdot \text{cycle}$). The number of operational cycles per year will vary depending on the type of system and other local conditions. For a routine 24-h operational cycle, the annual solids loading typically will exceed 328 lb/ft^2 (1600 kg/m^2), including allowances for maintenance and downtime. This annual loading exceeds the solids loading rate for a conventional sand drying bed by an order of magnitude. An enclosed and possibly heated facility would be needed to maintain such production rates in cold climates.

In many systems, the sludge cake is removed soon after completion of the drainage phase to maintain high production rates. Typically, the sludge cake will be from 8 to 12 percent solids after 24 hours of drying. Production of a drier sludge would require more time on the bed or removal to a stockpile area for evaporative drying.

3.4.12.2 Performance expectations. According to industrial representatives, polymer-treated aerobically digested sludge can be dewatered to 8 to 12 percent solids within 24 hours, and anaerobically digested sludge can be dewatered to between 16 and 20 percent solids in the same time period [48].

It is critical for the system operator to carefully manage the initial controlled drainage rate to ensure maximum water flow during this phase. If the rate is too slow, the total cycle time will have to be increased. Conversely, if the rate is too fast, complete drainage may not occur. The manufacturer's drainage recommendation can be used initially and then modified as necessary with operational experience.

3.4.12.3 Operation and maintenance. Polymer conditioning is critical for successful performance of wedgewater drying beds. The polymer dosages are similar to those required for vacuum-assisted drying beds. Although the typical depth of a single sludge application ranges from 4 to 10 in (10–25 cm), in some cases it may be possible to apply multiple sequential of sludge layers with decantation of the supernatant prior to starting the drainage phase [38,48].

There have been reports of damage to the plastic surfaces when front-end loaders have been used improperly to remove the sludge cake from wedgewater systems. The proper procedure for dewatered sludge removal requires driving straight in and backing straight out. Sharp turns can cause structural damage to the molded-polyurethane surfaces.

3.4.13 Sludge drying lagoons

When sufficient land is available, drying lagoons are a cost-effective method for sludge dewatering. Two types of drying lagoons may be used for sludge dewatering: (1) storage lagoons and (2) drying lagoons [30,33,48,71]. The objective of storage lagoons is to store the sludge in a relatively deep earthen or concrete basin for multiple years. Sludge storage lagoons are normally 5 to 15 ft (1.5–4.6 m) deep, with the storage time dictated by the sludge generation rate and the final disposal method.

Sludge drying lagoons are relatively shallow concrete basins that are designed for in-place drying of the sludge. Sludge is applied in these systems to a depth of 6 to 15 in (15–38 cm). Water removal is by evaporation, with decanting also practiced if significant amounts of standing water are generated. After the sludge has reached an air-dried state, it is removed either by a front-end loader or other mechanical equipment. Design approaches for both sludge storage and sludge drying lagoons are described in Chap. 5.

3.4.14 Solids capture during sludge dewatering

Since no dewatering process has a 100 percent solids capture efficiency, some solids are inevitably lost during the process and recycled. Solids loss during mechanical dewatering of sludge occurs by two mechanisms: (1) solids passage through the filtering mechanism or, with centrifuges, solids lost in the centrate and (2) incomplete separation of solids from the medium and the need to spray wash the medium prior to the next application of sludge.

The percentage of solids captured in a mechanical dewatering system is variable and depends on (1) the type of sludge, (2) the extent of stabilization, (3) the type and amount of conditioning chemicals used, (4) the type of dewatering equipment used, and (5) the desired solids content of the dewatered sludge [5,33,48]. For percolation/evaporation systems, solids are lost in percolation water from drying beds and in decant water for sludge lagoons. Table 3.36 lists the typical ranges of solids capture from the various dewatering processes. It should be noted that concentrations of solids in the liquid sidestreams from the dewatering processes (e.g., filtrate, centrate, and percolated liquid) are inversely proportional to the percentage solids capture.

TABLE 3.36 Typical Solids Capture of Dewatering Devices*

Dewatering process	Typical solids capture (%)
Centrifugation	80–98
Belt filter press	85–95
Vacuum filter	88–95
Filter press	>98
Drying bed	>99
Sludge lagoon	>99

*Adapted from ref. [48].

3.4.15 Evaluating sludge dewatering potential

A number of laboratory and bench-scale tests are available to determine the potential dewaterability of sludge. These tests serve a number of purposes, including

- Development of sizing criteria for full-scale dewatering equipment
- Evaluation and optimization of conditioning processes
- Dewatering operational control

Some of the general laboratory analyses used to evaluate the effectiveness of conditioning chemicals were described previously, e.g., filter leaf test, specific resistance (Buchner funnel) test, and capillary suction time. The following sections describe tests that are designed specifically for the particular dewatering device that is being considered.

3.4.15.1 Filter belt press simulator. To estimate the dewaterability of sludge using a belt filter press, the filter belt press simulator may be used. In this test, sludge is first placed in a filtration cell and pressed by a stainless steel piston into a section of the filter medium. The press is equipped with a pressure and filtrate recording device [48]. Placing a section of belt medium at the end of the piston and then rotating the piston simulates the shearing action that occurs on a full-scale belt as it passes around the rollers.

Data from the filter belt press simulator can be analyzed either to determine the required pressing time for a desired final cake solids content or to estimate the optimal conditioning chemical dosage for given final sludge cake moisture content. To determine the required pressing time, the final cake solids content is correlated with the pressing time for various pressures (Fig. 3.128*a*). It should be noted that Fig. 3.128*a* is based on the assumption that there is no gravity drainage prior to pressing. To determine the optimal chemical dosage, the final cake solids content is correlated with the various dosages of conditioning chemicals (Fig. 3.128*b*).

When compared with specific resistance tests, the filter belt press simulator appears to generate more realistic estimations of the dewaterability performance of belt filter presses [48].

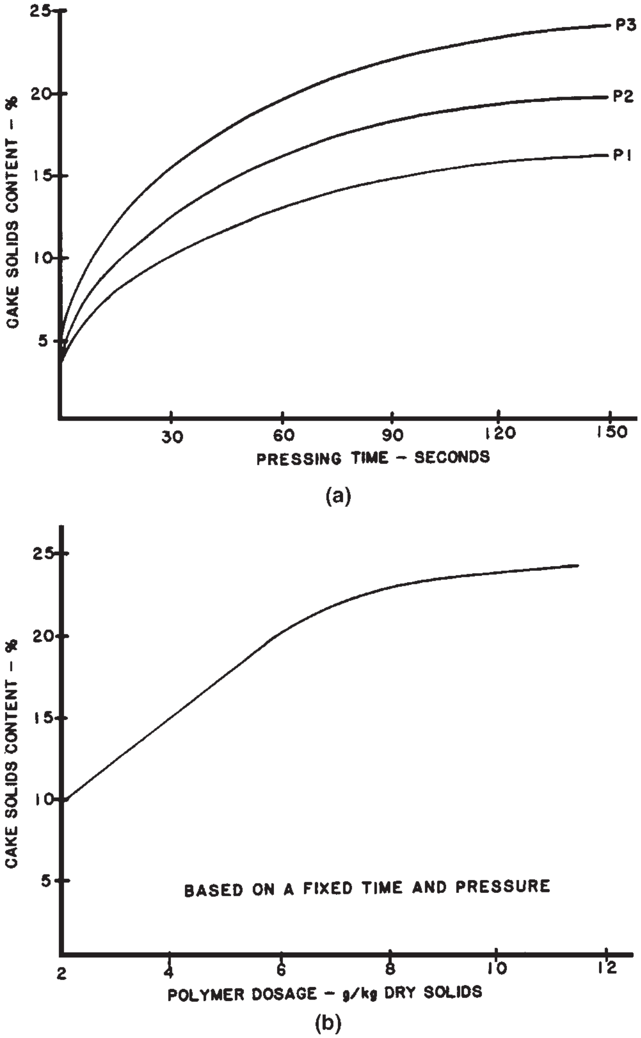


Figure 3.128 (*a*) Correlation of final cake solids versus pressing time from the filter belt press simulator. (*b*) Correlation of final cake solids content versus chemical dosage from the filter belt press simulator.

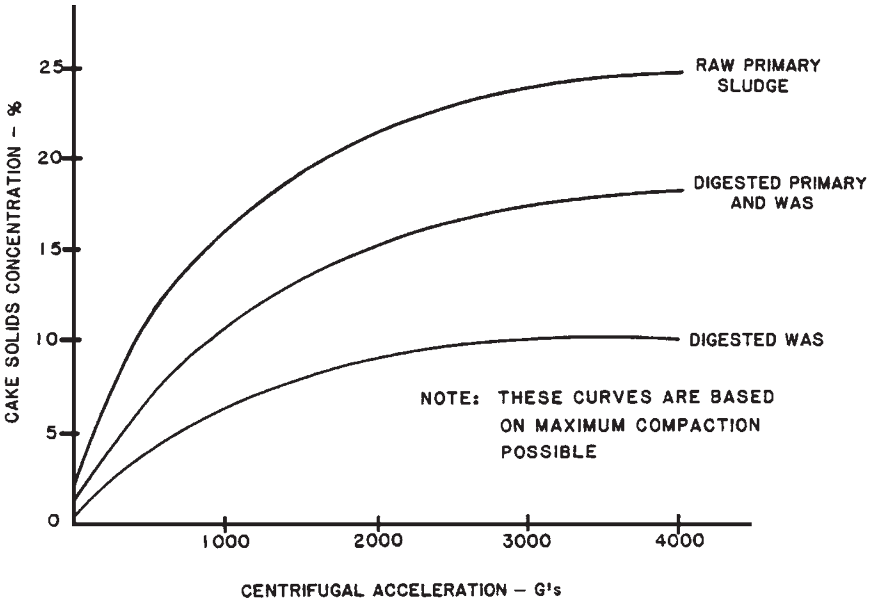


Figure 3.129 Typical data generated in a bottle centrifuge test.

3.4.15.2 Bottle centrifuge method. Like the vacuum filter and belt filter press, there are bench-scale tests available for evaluating the performance of a centrifugal dewatering process [48]. The most prevalent laboratory-scale centrifuge test is the bottle centrifuge method. This method involves spinning a graduated centrifuge tube filled with conditioned sludge at various *G*-forces or a constant *G*-force for various lengths of time. At the termination of testing, the centrate is decanted, and the percentage solids of the resulting sludge cake is measured. Figure 3.129 illustrates a plot of typical data from a bottle centrifuge test.

It should be noted that retention time in the centrifuge together with the type and dosage of conditioning chemical used will affect the shape of the curves and the final cake solids concentration. Readers interested in more details of any of the laboratory/bench scale tests are referred to the following references [38,48].

3.5 Heat Drying

Heat drying is the process of evaporating water from sludge by thermal means. Under normal conditions, heat drying is used to remove additional moisture from sludge following mechanical dewatering. Reasons for employing heat drying include the desire to (1) market the final sludge product in bags, (2) increase the fuel value of sludge, and (3) reduce sludge transportation costs. The minimum sludge moisture content achievable with heat drying depends on the design and operation of the dryer, moisture content of the feed, and its chemical composition. The typical range of solids content achieved by this process is in the range of 90 to 99 percent [33,38,48]. In heat drying, the temperature difference between the heating medium and the sludge-air interface provides the driving force for heat transfer. Dryers are classified on the basis of the predominant method of transferring heat to the wet solids.

3.5.1 General design of heat dryers

Mechanical dewatering is an important pretreatment step in conventional heat drying because it reduces the volume of water that must be removed in the dryer. In the dryer, water that has not been removed mechanically is evaporated without decomposing the organic matter in the sludge. This means that the sludge temperature must be kept between 140 and 200°F (60 and 93°C). A large portion of the dried sludge is blended with the wet sludge fed to the dryer, making the drying operation more efficient by reducing solids agglomeration (formation of larger chunks of sludge) and thus exposing a larger solids surface to the drying air.

Dried sludge and exhaust gases are separated in the dryer itself and/or in a cyclone. The gas stream can go to a pollution-control device (e.g., scrubber), whereas the dried sludge is sent to a finishing step such as a pelletizing/bagging process or is stored in bulk for marketing.

The number and size of heat dryers will depend on the type of drying operation. If the dryers are operated continuously, extra dryer capacity is required so that all the sludge produced can be dried while maintenance and repairs are being performed. In cases where noncontinuous operation is envisioned or where only one dryer is installed, the dryer(s) must have sufficient evaporative capacity to treat all the sludge generated.

The design engineer should determine the storage requirements for both the wet sludge feed and the dried product. Sufficient wet sludge storage should be provided to allow for an orderly shutdown of a continuously operating system. Storage for the dried product depends on the final disposal practice. Sale of bagged sludge is likely to be seasonal, and considerable storage may be necessary unless bulk buyers can be found that have off-site storage capacity. Dust also can become a problem if the dried product is stored in bulk and is not pelletized [38,48].

3.5.1.1 Heat source. Thermal evaporation of water from sludge requires considerable energy. The amount of fuel required to dry sludge depends on the amount of water evaporated. The energy required to evaporate water from thickened sludge is composed of the heat required to

- Raise the temperature of sludge to the temperature at which it leaves the dryer
- Raise the water temperature to the point at which it can evaporate (latent heat)
- Raise the temperature of the exhaust gas (including water vapor) to the exhaust temperature
- Offset heat losses

The large amounts of energy required for heat drying dictate that close attention be paid to the source of energy used and its cost. Natural gas and fuel oil are the most frequently used energy source. Energy recovery with the heat-drying system provides one approach to reducing energy cost. For example, heat exchangers can be employed to recover heat from the exhaust gases. It should be noted that the dried sludge has a heating value and may, in some circumstances, be used as the energy source [38,48].

3.5.2 Direct dryers

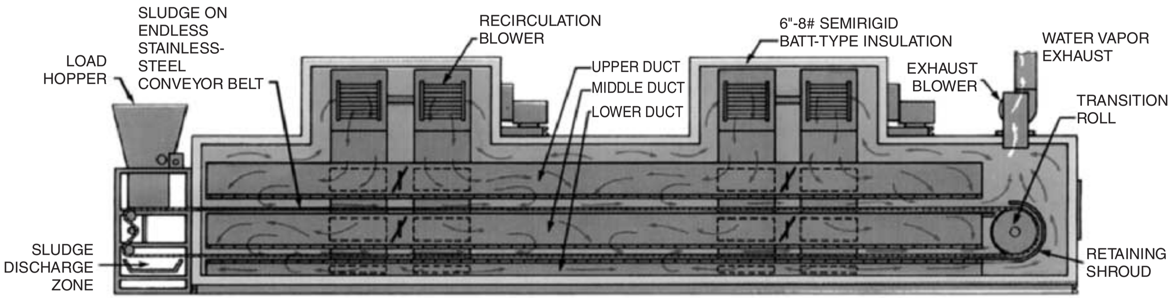
In direct dryers, the heat transfer for drying is accomplished by direct contact between the wet feed sludge and hot influent gases. Figure 3.130 provides a schematic diagram of a direct dryer system.

In direct drying systems, it is the sensible heat of the inlet gas that provides the latent heat for evaporating water. The convective heat-transfer characteristic of direct dryers is described by Eq. (3.92):

$$q_{\text{conv}} = h_c A\,(t_g - t_s) \tag{3.92}$$

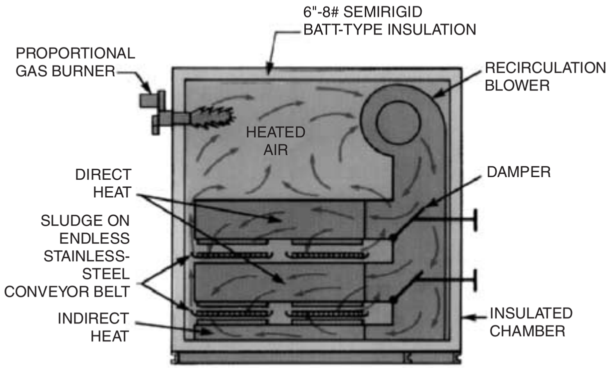
where q_{conv} = convective heat transfer, Btu/h
h_c = heat transfer coefficient, Btu/ft^2 · h · °F
t_g = gas temperature, °F
t_s = temperature of sludge-air interface, °F

Airflow is an important consideration in the design of direct-heating dryers. Airflow may be concurrent, countercurrent, or cross-flow. Concurrent flow describes a dryer design in which both the sludge and the gas phase flow in the same direction. Countercurrent flow describes a dryer design in which the directions of gas flow and sludge movement are opposite, whereas cross-flow describes a dryer design in which the gas flow and sludge movement are at right angles to one another [38].



Double-Pass Sludge Dryer: Side View

(a)



Double-Pass Sludge Dryer: Section View
Air Recirculation Path

(b)

Figure 3.130 (*a*) Schematic diagram of a direct biosolids/sludge dryer system (side view). (*Courtesy of US Filter/Envirex.*) (*b*) Schematic diagram of a direct biosolids/sludge dryer system (section view). (*Courtesy of US Filter/Envirex.*)

Concurrent gas flow normally offers the advantage of higher thermal efficiency due to rapid cooling of the heating medium near the feed end of the drying with reduced heat losses through the dryer. The rates of airflow are a function of dryer design. However, turbulent conditions must be maintained to ensure intimate contact between the warm air and wet sludge.

3.5.3 Indirect dryers

In indirect dryer systems, the heat for sludge drying is transferred through a retaining wall (normally metallic). The vaporized moisture is removed independently of the hot effluent gases. The rate of drying depends on the extent of contact between the wet sludge and the hot surfaces. Indirect dryers are also termed *conduction* or *contact dryers.* Examples of indirect dryers include (1) some rotary dryers, (2) drum dryers, (3) screw-conveyor dryers, (4) steam tube rotary dryers, and (5) freeze dryers. Figure 3.131 provides schematic diagrams of several indirect dryer systems that use hollow augers to both heat and transport the sludge. It should be noted that the heating fluid (which can be oil, steam, or water) is recirculated continuously within the systems.

The conductive heat transfer that is characteristic of indirect drying systems is described by Eq. (3.93):

$$q_{\text{cond}} = h_{\text{cond}} A\,(t_m - t_{\text{bio}}) \tag{3.93}$$

where q_{cond} = conductive heat transfer, Btu/h
h_{cond} = conductive heat transfer coefficient, Btu/ft^2 · h · °F
t_m = temperature of drying medium (e.g., steam), °F
t_{bio} = temperature of sludge or biosolids at drying surface, °F

The conductive heat-transfer coefficient h_{cond} is a composite term that includes the effects of the heat-transfer surface sludge and medium-side heat-resistance films [38].

3.5.4 Infrared or radiant-heat dryers

In infrared and radiant-heat dryers, the heat required to dry the sludge is provided by radiant energy supplied by electrical resistance elements, by gas-heated incandescent refractories, or by infrared lamps. Figure 3.132 illustrates an infrared heat drying system for biosolids/sludge.

As in indirect drying systems, no direct flame touches the biosolids/sludge within infrared drying systems. Infrared or radiant heat transfer is described by Eq. (3.94):

$$q_{\text{rad}} = \varepsilon_s A\sigma\,(t_r^{\,4} - t_B^{\,4}) \tag{3.94}$$

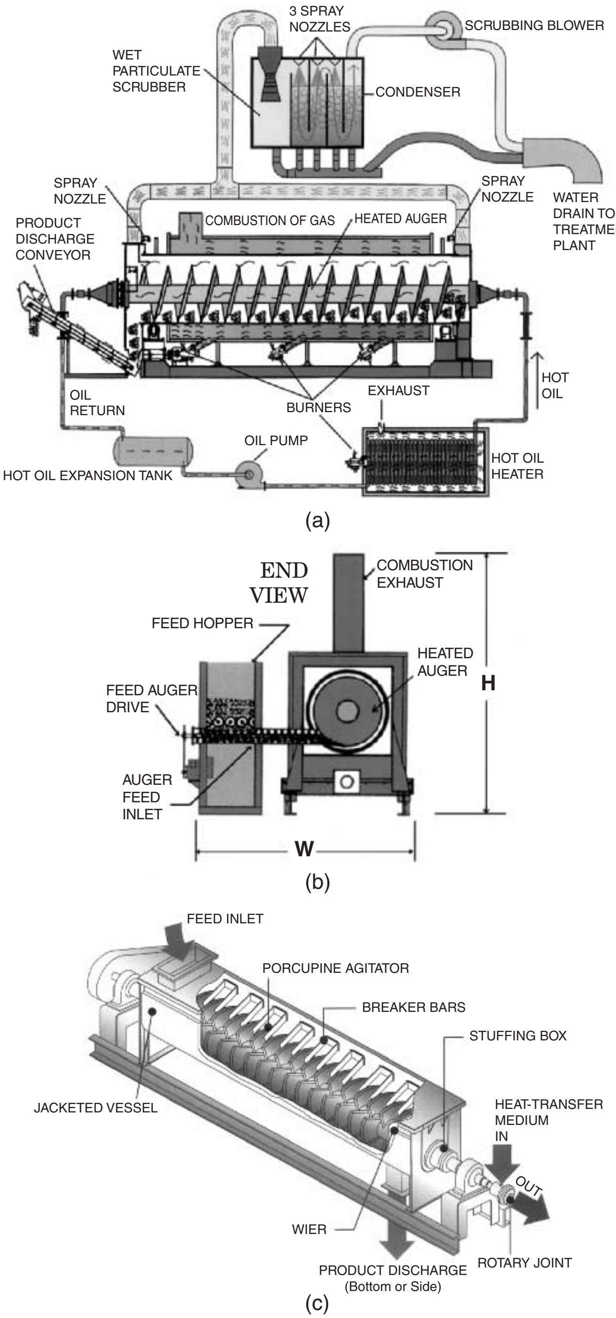


Figure 3.131 (*a*) Schematic diagram of SludgeMASTER indirect rotating sludge dryer (side view). (*Courtesy of US Filter/Davis Process.*) (*b*) Schematic diagram of SludgeMASTER indirect rotating sludge dryer (end view). (*Courtesy of US Filter/Davis Process.*) (*c*) Porcupine processor indirect sludge dryer. (*Courtesy of The Bethlehem Corporation.*)

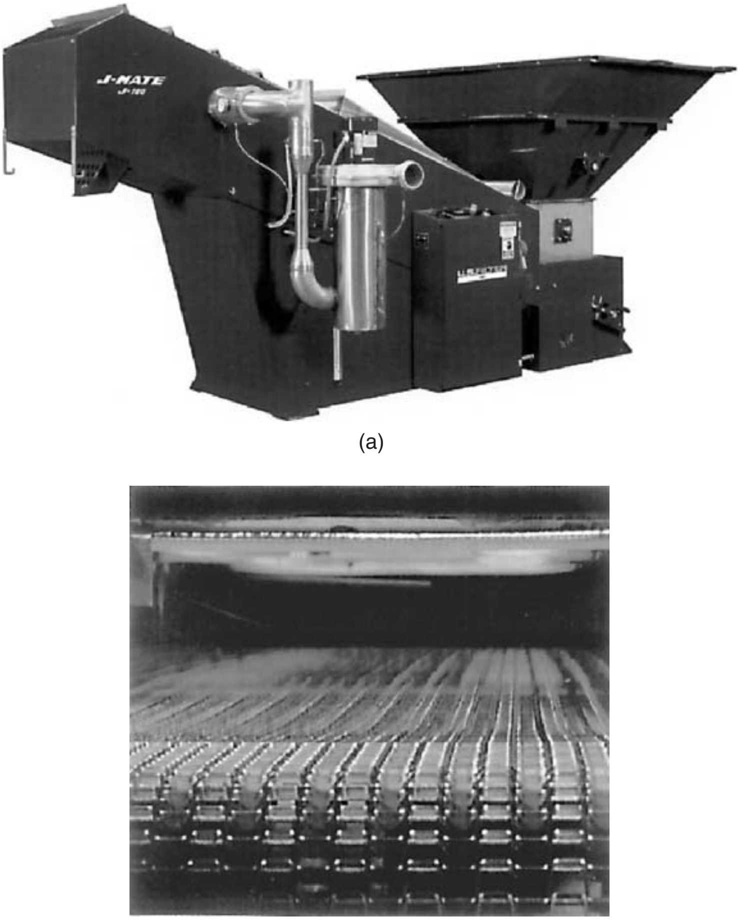


Figure 3.132 (*a*) Infrared biosolids/sludge dryer (J-Mate). (*Courtesy of US Filter/JWI.*) (*b*) Infrared heating elements within dryer. (*Courtesy of US Filter/JWI.*)

where q_{rad} = radiation heat transfer, Btu/h
ε_s = emissivity of drying surface, dimensionless
A = sludge surface area exposed to radiant source, ft^2
σ = Stefan-Boltzman constant, $1.73 \cdot 10^{-9}$ Btu/h · ft^2 · °R
t_r = absolute temperature of radiant source, °R
t_B = absolute temperature of sludge drying surface, °R
°R = degrees Rankine

Depending on the physical dryer configuration, the resulting biosolids/sludge may be in the forms of large chunks or pellets.

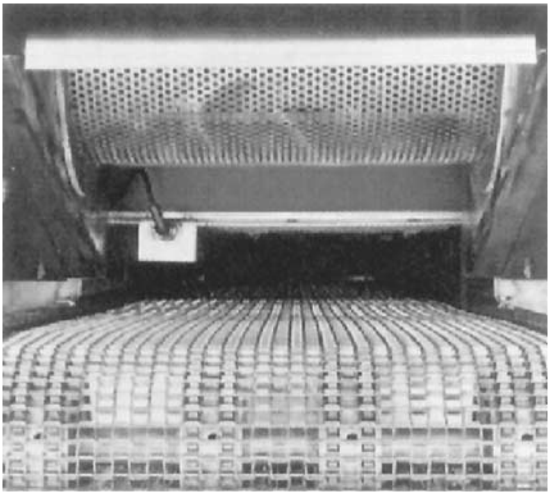


Figure 3.133 Drum extruder used to produce pelletized biosolids/sludge in infrared dryer system. (*Courtesy of US Filter / JWI.*)

Pelletized biosolids/sludge is more desirable from the standpoint of moisture removal. Moreover, pelletized biosolids/sludge is easier to handle and transport compared with larger material. Figure 3.133 depicts a drum extruder used on some dryer designs to produce pelletized material for enhanced drying.

In many circumstances, the results predicted by the heat-transfer equations can be significantly different from what is observed in the field. In practice, the most valuable heat-drying design information is obtained by evaluating the dryers using actual process feeds under conditions closely simulating prototype operations.

In designing a heat-drying process, the actual conditions of drying (e.g., temperature, humidity, detention time, and direction of the gas flow) must be established by the process engineer. The three defined stages of heat drying of sludge are described in Table 3.37.

One of the most important operational and design parameters that affects the energy requirement of the sludge heat-drying process is humidity and moisture transfer. The following section evaluates these critical operational parameters in the sludge heat-drying process.

3.5.5 Humidity and moisture transfer

Humidity is a measure of the moisture content of the gas phase at a given temperature. In heat drying of sludge, water is transferred to the air phase. The driving force for moisture transfer is the difference between the absolute humidity (pounds of water per pound of dry air) at the wetted sludge-air interface and the absolute humidity in the gas

TABLE 3.37 Three Stages of Heat Drying of Sludge*

First stage: Initial drying	During this stage, the sludge temperature and the drying rates are increased to the steady-state conditions of the second stage.
Second stage: Steady-state drying	During this period, the surfaces of the sludge are completely saturated with water. Surface water lost is replaced with water from the interior of the sludge as fast at it evaporates. The temperature of the sludge-air interface is kept at the wet bulb temperature of the gas.
Third stage: Final drying	The final stage occurs when sufficient water has evaporated from the sludge resulting in the solid surface being only partially saturated. During this stage, the temperature of the sludge-air interface increases because the latent heat cannot be transferred from the sludge to the gas phase as rapidly as sensible heat is received from the heating mechanism.

*Adapted from ref. [38].

phase. A related term, *molal humidity,* is the number of moles of water per mole of moisture free gas [27].

To properly evaluate heat drying systems, it is necessary to define two additional terms: *partial pressure* and *vapor pressure*. By definition, the partial pressure of water in a gas is the pressure that water would exert if it were the only component in the same volume and at the same temperature as the gas. Mathematically, partial pressure of water in a gas is equal to the total pressure of the gas (P_{total}) times the gas phase mole fraction of water ($Y_{\text{H}_2\text{O}}$). It should be noted that for gases (and vapors) exhibiting ideal behavior, the mole fraction of water is equivalent to the volume fraction of water in the gas (or vapor) phase.

When water evaporates into a limited space, two opposing processes are in operation: *vaporization* and *condensation*. If sufficient liquid is present, the pressure of the vapor will ultimately reach a value at which the rate of vaporization is just equal to the rate of condensation. At this dynamic equilibrium condition, the vapor is said to be saturated. The pressure exerted by the vapor at such equilibrium conditions is termed the *vapor pressure* of the liquid. The vapor pressure of any substance is a function only of temperature.

If air contains moisture at such a concentration that the partial pressure of the moisture in the gas phase is less than the vapor pressure of water at the same temperature, the air is unsaturated. In other words, the air is capable of absorbing more moisture than it presently contains. The *percent relative humidity* of such a gas mixture is defined as the ratio of the partial pressure of the water vapor in the mixture to the vapor pressure of the liquid at the existing temperature [Eq. (3.95)]:

Percent relative humidity (%)

$$= \frac{\text{partial pressure of water in gas phase}}{\text{vapor pressure of liquid water at given temperature}} \cdot 100 \tag{3.95}$$

On the other hand, the *percent saturation* is defined as the ratio of the existing weight of air–water vapor mixture to weight of the air–water vapor mixture if the mixture were at saturated conditions at the given temperature and pressure. Example 3.30 illustrates the approach for estimating both the percent relative humidity and percent saturation of heated air.

Example 3.30 Air leaving a dryer at 170°F (77°C) contains 15 percent moisture by volume. Given the dryer's operating conditions, what is the relative humidity and percent saturation of the exiting air? Assume that the total pressure remains constant at 1 atm (760 mmHg) and the vapor pressure of water at 170°F is 310 mmHg.

solution

Step 1. Estimate the partial pressure of water in the exiting gas and the percent relative humidity.

$$\text{Partial pressure of moisture in gas} = 0.15 \cdot 760 \text{ mmHg} = 114 \text{ mmHg}$$

$$\text{Percent relative humidity} = \frac{\text{partial pressure}}{\text{vapor pressure}} \cdot 100$$

$$= \frac{114 \text{ mmHg}}{310 \text{ mmHg}} \cdot 100 = 36.8 \text{ percent}$$

Step 2. Estimate the moles of moisture in air at saturated and unsaturated conditions.

Basis: 1 lb · mol of saturated air at 170°F:

$$\text{\% volume of } H_2O \text{ in air} = \frac{310 \text{ mmHg}}{760 \text{ mmHg}} = 0.408$$

$$\text{Moles of } H_2O \text{ in saturated air} = 0.408 \text{ mol}$$

$$\text{Moles of } H_2O\text{-free air} = 1 - 0.408 = 0.592 \text{ mol}$$

$$\frac{\text{Moles of } H_2O}{\text{Moles of } H_2O\text{-free air}} = \frac{0.408}{0.592} = 0.689$$

Basis: 1 lb · mol of unsaturated air at 170°F:

$$\text{\% volume of } H_2O \text{ in air} = \frac{114 \text{ mmHg}}{760 \text{ mmHg}} = 0.15$$

$$\text{Moles of } H_2O \text{ in saturated air} = 0.15 \text{ mol}$$

$$\text{Moles of } H_2O\text{-free air} = 1 - 0.15 = 0.85 \text{ mol}$$

$$\frac{\text{Moles of } H_2O}{\text{Moles of } H_2O\text{-free air}} = \frac{0.15}{0.85} = 0.176$$

Step 3. Estimate the percent saturation based on moisture content of the air under saturated and unsaturated conditions.

$$\text{Percent saturation } (\%) = \frac{\left(\dfrac{\text{moles of } H_2O}{\text{moles of } H_2O\text{-free air}}\right)_{\text{unsaturated conditions}}}{\left(\dfrac{\text{moles of } H_2O}{\text{moles of } H_2O\text{-free air}}\right)_{\text{saturated conditions}}} \cdot 100$$

$$= \frac{0.176}{0.689} \cdot 100 = 25.6 \text{ percent}$$

When wet sludge is exposed to either hot gases or a hot surface, moisture will continue to evaporate from the sludge until the partial pressure of water in the gaseous phase equals the vapor pressure of liquid water. Under equilibrium conditions, the temperature at which the partial pressure of water in the gas phase equals the vapor pressure of the liquid is called the *dew point*. At the dew point, the air phase is saturated with moisture, and no further evaporation will occur. It should be noted that the dew point of an air stream depends only on the total pressure and the moisture content of the air. Example 3.31 illustrates the approach for estimating the dew point of an air stream.

Example 3.31 Hot air leaving a batch dryer contains 16 percent moisture (by volume). If the air is saturated with moisture, what is the dew point of this gas if the total pressure is maintained at 760 mmHg?

solution Determine the partial pressure of moisture in the gas phase.

$$\text{Partial pressure of water in air} = 0.16 \cdot 760 \text{ mmHg}$$

$$= 121.6 \text{ mmHg}$$

From vapor pressure tables, water has a vapor pressure of 121.6 mmHg at a temperature of 133°F (which is the dew point).

In all calculations involving moisture and heat transfer in a sludge dryer, it is assumed that the influent and effluent gases behave ideally. Under this assumption, the ratio of the number of moles of water in the gas phase to the number of moles of moisture-free air is equal to the ratio of the partial pressure of water in the gas phase to the partial pressure of the moisture-free gas. Example 3.32 illustrates how this ideal gas principal can be employed for estimating the moisture removal from a sludge drying operation.

Example 3.32 Air maintained at a temperature of 220°F (104°C) and containing 5 percent moisture (volume basis) enters a batch biosolids dryer. After a 20-second residence time, the air is released from the dryer at a temperature of 150°F (65°C). If the effluent air is assumed to be saturated with moisture, estimate the pounds of water removed per 1000 ft^3 of hot air entering the dryer. Assume that the following data apply:

1. Total pressure: 760 mmHg
2. Vapor pressure of H_2O at 220°F (104°C): 888.5 mmHg
3. Vapor pressure of H_2O at 150°F (65°C): 192.3 mmHg
4. Molecular weight of H_2O: 18 lb/lb · mol
5. Molar volume of ideal gas at 32°F and 760 mmHg: 359 ft^3

solution

Step 1. Perform a mass (mole) balance on water entering and leaving dryer.

Air entering dryer:

$$\text{Partial pressure of moisture} = 760 \text{ mmHg} \cdot (0.05) = 38 \text{ mmHg}$$

$$\text{Partial pressure of moisture-free gas} = 760 - 38 = 722 \text{ mmHg}$$

Moles of moisture per mole of moisture-free gas

$$= \frac{38 \text{ mmHg}}{722 \text{ mmHg}} = \frac{0.0526 \text{ lb} \cdot \text{mol moisture}}{\text{lb} \cdot \text{mol moisture-free gas}}$$

Air leaving dryer:

$$\text{Partial pressure of moisture} = 192.3$$

$$\text{Partial pressure of moisture-free gas} = 760 - 192.3 = 567 \text{ mmHg}$$

Moles of moisture per mole of moisture-free gas

$$= \frac{192.3 \text{ mmHg}}{567.7 \text{ mmHg}} = \frac{0.339 \text{ lb} \cdot \text{mol moisture}}{\text{lb} \cdot \text{mol moisture-free gas}}$$

Step 2. Estimate the moles of water removed per mole of moisture-free gas into the dryer.

Basis: 1 lb · mol of moisture-free gas entering dryer:

$$\begin{aligned} \text{Moisture leaving dryer} &= 0.339\ \text{lb} \cdot \text{mol} \\ -\ \text{Moisture entering dryer} &= \underline{0.0566\ \text{lb} \cdot \text{mol}} \\ \text{Moisture evaporated from biosolids} &= 0.286\ \text{lb} \cdot \text{mol} \end{aligned}$$

Step 3. Estimate the molar volume of an ideal gas at 220°F (104°C) using the ideal gas law and the Rankine scale.

$$\begin{aligned} V_2 &= V_1 \frac{T_2}{T_1} \\ &= 359\ \text{ft}^3\ \frac{460°\text{R} + 220°\text{F}}{460°\text{R} + 32°\text{F}} = 359\ \text{ft}^3\ \frac{680}{492} \\ &= 496.2\ \text{ft}^3 \end{aligned}$$

Step 4. Estimate the pounds of water removed per 1000 ft^3 of entering gas.

$$\frac{\text{lb of H}_2\text{O removed}}{\text{ft}^3\ \text{of influent gas}} = \frac{0.286\ \text{lb} \cdot \text{mol H}_2\text{O}}{\text{lb} \cdot \text{mol of H}_2\text{O-free gas}}$$

$$\cdot \frac{0.95\ \text{lb} \cdot \text{mol of H}_2\text{O-free gas}}{\text{lb} \cdot \text{mol of influent gas}} \cdot \frac{\text{lb} \cdot \text{mol of influent gas}}{496.2\ \text{ft}^3} \cdot \frac{18\ \text{lb H}_2\text{O}}{\text{lb} \cdot \text{mol H}_2\text{O}}$$

$$= \frac{0.00985\ \text{lb H}_2\text{O removed}}{\text{ft}^3\ \text{of influent gas}}$$

$$\left(\text{or}\ \frac{9.85\ \text{lb H}_2\text{O removed}}{1000\ \text{ft}^3\ \text{of influent gas}}\right)$$

3.5.5.1 Psychrometry. When water evaporates into a large volume of an unsaturated gas, the liquid will cool to supply part of the energy needed for moisture evaporation. As the temperature of the liquid decreases below the gas temperature, sensible heat from the gas is transferred to the liquid moisture. Ultimately, a steady-state liquid moisture temperature is reached at which the heat needed to evaporate the liquid moisture and raise it to the temperature of the gas is exactly balanced by the sensible heat flowing from the gas to the liquid moisture. This steady-state liquid temperature is defined as the wet-bulb temperature, while the gas temperature is defined as the dry-bulb temperature [21,27,38]. Approaches to measuring the wet- and dry-bulb temperatures of gas streams are described in the following references [21,27]. In all cases of unsaturated air streams, the dry-bulb temperature is greater than the wet-bulb temperature.

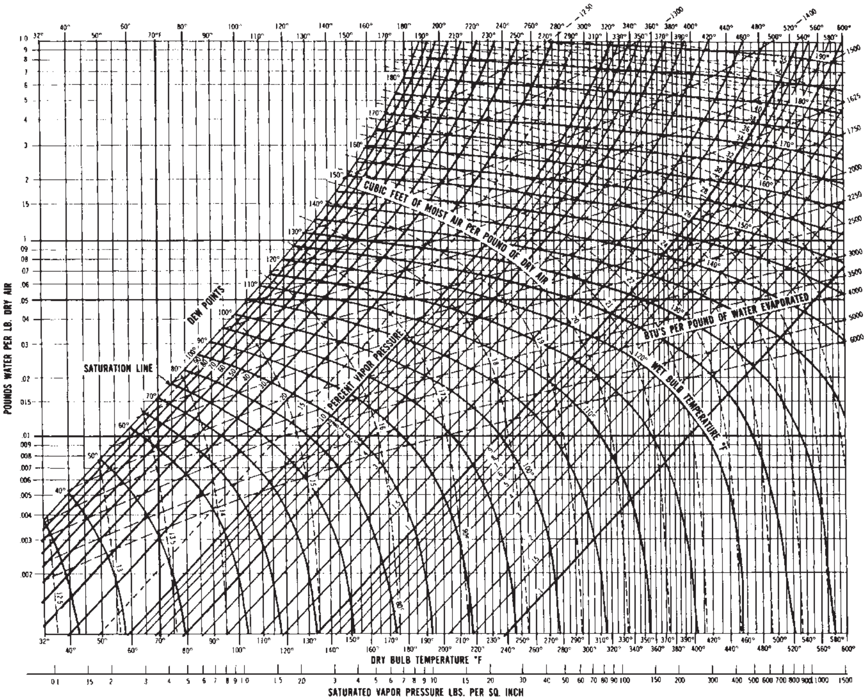


Figure 3.134 Psychrometric chart for estimating the water content of air.

If the gas is saturated with moisture, neither vaporization of the liquid no depression of the wet-bulb temperature occurs. Therefore, depression of the wet-bulb temperature can be used as a measure of the degree of unsaturation of the gas. The process of using wet- and dry-bulb measurements to estimate moisture removal in drying operations is called *psychrometry*.

Use of a psychrometry or humidity chart can simplify moisture transfer calculations in sludge drying operations (Fig. 3.134). In the psychrometry chart, humidity (pounds of moisture per pound of moisture free air) is plotted on the y axis, with the dry-bulb gas temperatures plotted on the x axis. Lines of constant wet-bulb temperatures are plotted on the curves together with the percent relative humidity. Any point on a given wet-bulb temperature line establishes the corresponding dry-bulb temperature, absolute humidity, and percent humidity. Fixing any two of these properties establishes the values for the other two. It should be noted that the wet-bulb and dry-bulb temperatures become identical at the dew point. Example 3.33 illustrates the use of the psychometric chart in solving sludge drying problems.

Example 3.33 Influent air to a biosolids dryer has a temperature of 200°F (93°C) and at atmospheric pressure has a wet-bulb temperature of 140°F

(60°C). After it leaves the dryer, it is at a temperature of 150°F (65°C) and has a wet-bulb temperature of 145°F (63°C). Given the operating conditions of the dryer, estimate the weight of water evaporated per 1000 ft^3 of moisture-free air entering the dryer. Assume that 1 lb · mol of dry air weighs approximately 29 lb.

solution

Step 1. Determine the absolute humidity of the influent and effluent air streams.

- *Influent air:* The dry-bulb temperature of 200°F is found on the x axis of Fig. 3.134. Move vertically up this temperature until the intersection with the wet-bulb temperature of 140°F is reached. At the point of intersection, move horizontally until the intersection with the y axis is reached, and read the absolute humidity that is approximately 0.133 lb H_2O per lb dry air. The percent relative humidity from the psychometric chart is approximately 6 percent.
- *Effluent air:* The dry-bulb temperature of 150°F is found on the x axis of Fig. 3.134. Move vertically up this temperature until the intersection with the wet-bulb temperature of 145°F is reached. At the point of intersection, move horizontally until the intersection with the y axis is reached, and read the absolute humidity that is approximately 0.177 lb H_2O per lb dry air.

Step 2. Using the absolute humidity of the influent and effluent gas, estimate the pounds of water removed per pound of dry air entering the dryer.

$$\frac{\text{lb H}_2\text{O removed}}{\text{lb dry air entering dryer}}$$

$$= \left(\frac{0.177\ \text{lb H}_2\text{O}}{\text{lb dry air}}\right)_{\text{leaving dryer}} - \left(\frac{0.133\ \text{lb H}_2\text{O}}{\text{lb dry air}}\right)_{\text{entering dryer}}$$

$$= \left(\frac{0.044\ \text{lb H}_2\text{O}}{\text{lb dry air}}\right)$$

Step 3. Estimate the molar volume of an ideal gas at 200°F using the ideal gas law and the Rankine scale.

$$V_2 = V_1 \frac{T_2}{T_1}$$

$$= 359\ \text{ft}^3 \frac{460^\circ\text{R} + 200^\circ\text{F}}{460^\circ\text{R} + 32^\circ\text{F}} = 359\ \text{ft}^3 \frac{660}{492}$$

$$= 481.6\ \text{ft}^3$$

Step 4. Using the molecular weight of dry air, estimate the pounds of water removed per 1000 ft^3 of dry air entering the dryer.

$$\frac{\text{lb H}_2\text{O removed}}{\text{ft}^3\ \text{of dry air entering dryer}}$$

$$= \frac{0.044 \text{ lb } H_2O \text{ removed}}{\text{lb of dry air entering dryer}} \cdot \frac{29 \text{ lb}}{\text{lb} \cdot \text{mol of dry air}} \cdot \frac{\text{lb} \cdot \text{mol of dry air}}{481.6 \text{ ft}^3}$$

$$= \frac{0.00265 \text{ lb } H_2O \text{ removed}}{\text{ft}^3 \text{ of dry air entering dryer}}$$

$$\left(\text{or } \frac{2.65 \text{ lb } H_2O \text{ removed}}{1000 \text{ ft}^3 \text{ of dry air entering dryer}}\right)$$

Knowledge of the absolute humidity at the sludge-air interface and within the air phase can be used directly to estimate the pounds of water removed per hour (sludge drying rate). The expression that can be used to estimate the sludge drying rate is described by Eq. (3.96):

$$W = K_y A \, (Y_s - Y_a) \tag{3.96}$$

where W = rate of drying, lb of water per hour
K_y = mass transfer coefficient for the gas phase, lb of water/h · ft², humidity difference
A = area of wetted surface, ft²
Y_s = humidity at the sludge-air interface temperature, lb of water/lb of dry air
Y_a = humidity of the air phase, lb of water/lb of dry air

Example 3.34 illustrates the approach for estimating the overall energy requirement to operate a heat drying system.

Example 3.34 The Omondi County Sewer District has purchased a dryer to process biosolids prior to rail transport. If 15,000 pounds per hour of dewatered biosolids containing 80 percent moisture and having an average temperature of 60°F are to be dried by direct contact with hot air, estimate (a) the required airflow rate in pounds per hour and (b) the required air inlet temperature to the dryer. Assume that the temperature of the air prior to heating is 70°F (21°C), and its absolute humidity is 0.008 lb of H_2O per pound of dry air. The temperature of the dried sludge is 140°F (60°C), and it contains 9 percent water. The dryer exhaust gas is 180°F (82.2°C) and contains 0.12 lb of water per pound of dry air. Radiant heat losses from the dryer structure are $0.5 \cdot 10^6$ Btu/hour.

Assume the following heat-capacity information is valid for the given problem:

Media	Heat capacity (Btu/lb · °F)
Dry air	0.24
Dry biosolids	0.30
Water	1.00
Water vapor	0.45

solution

Step 1. Estimate the airflow by conducting a moisture balance around the dryer.

Moisture into dryer:

$$\text{Moisture in biosolids} = \left(\frac{15{,}000\ \text{lb biosolids}}{\text{h}}\right)\left(\frac{0.8\ \text{lb H}_2\text{O}}{\text{lb biosolids}}\right)$$

$$= \frac{12{,}000\ \text{lb H}_2\text{O}}{\text{h}}$$

$$\text{Moisture in inlet air} = \left(\frac{Q\ \text{lb dry air}}{\text{h}}\right)\left(\frac{0.08\ \text{lb H}_2\text{O}}{\text{lb dry air}}\right)$$

$$= 0.008Q/\text{h}$$

Moisture out of dryer:

Moisture in biosolids

$$= \left(\frac{15{,}000\ \text{lb biosolids}}{\text{h}}\right)\left(\frac{0.2\ \text{lb dry solids}}{\text{lb biosolids}}\right)\cdot\left(\frac{9\ \text{lb H}_2\text{O}}{91\ \text{lb dry solids}}\right)$$

$$= \frac{296.7\ \text{lb H}_2\text{O}}{\text{h}}$$

$$\text{Moisture in air} = \left(\frac{Q\ \text{lb dry air}}{\text{h}}\right)\cdot\left(\frac{0.12\ \text{lb H}_2\text{O}}{\text{lb dry air}}\right) = 0.12\ Q$$

Equate moisture in and out and solve for required airflow rate Q (lb/h):

$$\frac{12{,}000\ \text{lb H}_2\text{O}}{\text{h}} + 0.008Q = \frac{296.7\ \text{lb H}_2\text{O}}{\text{h}} + 0.12Q$$

$$Q = 104{,}493.7\ \text{lb air per hour}$$

Step 2. Determine the required air inlet temperature by conducting a heat balance on the dryer. To perform a heat balance, a reference temperature must be chosen. For this example, a reference temperature of 32°F is arbitrarily chosen.

Heat content of biosolids into dryer:

Heat content of dry biosolids

$$= \left(\frac{15{,}000 \text{ lb biosolids}}{\text{h}}\right)\left(\frac{0.2 \text{ lb solids}}{\text{lb solids}}\right)\left(\frac{0.30 \text{ Btu}}{\text{lb} \cdot {}^\circ\text{F}}\right)(60^\circ - 32^\circ\text{F})$$

$$= 25{,}200 \text{ Btu/h}$$

Heat content of water associated with biosolids

$$= \left(\frac{15{,}000 \text{ lb biosolids}}{\text{h}}\right)\left(\frac{0.8 \text{ lb H}_2\text{O}}{\text{lb solids}}\right)\left(\frac{1.0 \text{ Btu}}{\text{lb} \cdot {}^\circ\text{F}}\right)(60^\circ - 32^\circ\text{F})$$

$$= 336{,}000 \text{ Btu/h}$$

$$\text{Heat content of biosolids entering dryer} = 25{,}200 + 336{,}000$$

$$= 361{,}200 \text{ Btu/h}$$

Heat content of air entering dryer:

$$\text{Heat content of dry air} = \left(\frac{104{,}493.7 \text{ lb air}}{\text{h}}\right)\left(\frac{0.24 \text{ Btu}}{\text{lb} \cdot {}^\circ\text{F}}\right)(t_2 - 32^\circ\text{F})$$

$$= 25{,}078.5\,(t_2 - 32^\circ\text{F}) \text{ Btu/h}$$

Heat content of moisture associated with air: This includes heat required to raise the moisture temperature from 32°F to the dew point, vaporizing the moisture, and finally increasing the vapor temperature to t_2. From the psychometric chart (Fig. 3.134) , the dew point temperature of air containing 0.008 lb H_2O per pound of dry air is 50°F. From steam tables (ref. [27]), the latent heat of vaporization at 50°F is 1065 Btu/lb.

$$\text{Heat content of moisture in air} = \left(\frac{104{,}493.7 \text{ lb dry air}}{\text{h}}\right)\left(\frac{0.008 \text{ lb H}_2\text{O}}{\text{lb dry air}}\right)$$

$$\cdot \left(\frac{1.0 \text{ Btu}}{\text{lb} \cdot {}^\circ\text{F}}\right) \cdot (50^\circ\text{F} - 32^\circ\text{F}) + 1065 + \left(\frac{0.45 \text{ Btu}}{\text{lb} \cdot {}^\circ\text{F}}\right)(t_2 - 50^\circ\text{F})\ ]$$

$$= 905{,}333.6 \text{ Btu/h} + 376.2\,(t_2 - 50^\circ\text{F})$$

Heat content of air entering the dryer

$$= 25{,}078.5\,(t_2 - 32^\circ\text{F}) + 905{,}333.6 \text{ Btu/h} + 376.2\,(t_2 - 50^\circ\text{F})$$

$$= 25{,}454.7t_2 + 84{,}011.6 \text{ Btu/h}$$

Heat out of dryer:

Heat content of dried biosolids

$$= \left(\frac{15{,}000 \text{ lb biosolids}}{\text{h}}\right)\left(\frac{0.20 \text{ lb solids}}{\text{lb biosolids}}\right)\left(\frac{0.30 \text{ Btu}}{\text{lb} \cdot {}^\circ\text{F}}\right)(140^\circ\text{F} - 32^\circ\text{F})$$

$$= 97{,}200 \text{ Btu/h}$$

$$\text{Heat content of residual water} = \left(\frac{15{,}000\ \text{lb biosolids}}{\text{h}}\right)\left(\frac{0.20\ \text{lb solids}}{\text{lb biosolids}}\right) \cdot \left(\frac{9\ \text{lb H}_2\text{O}}{91\ \text{lb solids}}\right)\left(\frac{1.0\ \text{Btu}}{\text{lb}\cdot{}^\circ\text{F}}\right)(140^\circ\text{F} - 32^\circ\text{F})$$

$$= 32{,}044\ \text{Btu/h}$$

Total heat out of dryer associated with biosolids

$$= 97{,}200\ \text{Btu/h} + 32{,}044\ \text{Btu/h}$$

$$= 129{,}244\ \text{Btu/h}$$

Heat content of exhausted air:

$$\text{Heat content of dry air} = \left(\frac{104{,}493.7\ \text{lb dry air}}{\text{h}}\right)\left(\frac{0.24\ \text{Btu}}{\text{lb}\cdot{}^\circ\text{F}}\right)(180^\circ\text{F} - 32^\circ\text{F})$$

$$= 3{,}711{,}616.9\ \text{Btu/h}$$

Heat content of moisture (from psychometric charts, the dew point of air containing 0.12 lb of water per pound of dry air is 135°F; the latent heat of vaporization at 135°F is 1017 Btu/h)

$$= \left(\frac{104{,}493.7\ \text{lb dry air}}{\text{h}}\right)\left(\frac{0.12\ \text{lb H}_2\text{O}}{\text{lb dry air}}\right) \cdot \left[\left(\frac{1.0\ \text{Btu}}{\text{lb}\cdot{}^\circ\text{F}}\right)(135^\circ\text{F} - 32^\circ\text{F}) + 1017 + \left(\frac{0.45\ \text{Btu}}{\text{lb}\cdot{}^\circ\text{F}}\right)(180^\circ\text{F} - 135^\circ\text{F})\right]$$

$$= 14{,}297{,}873.0\ \text{Btu/h}$$

Step 3. Using a heat balance, estimate the air inlet temperature to the dryer t_2: At steady state, the overall heat balance around the dryer is given as follows:

$$361{,}200\ \text{Btu/h} + 25{,}454.7t_2/\text{h} + 84{,}011.6\ \text{Btu/h}$$

$$= 129{,}244\ \text{Btu/h} + 3{,}711{,}616.9\ \text{Btu/h} + 14{,}297{,}873\ \text{Btu/h} + 500{,}000\ \text{Btu/h}$$

$$t_2 = 714.7^\circ\text{F (or } 379.3^\circ\text{C)}$$

3.6 Problems

3.1 The Siaya City Water Reclamation Plant is currently treating 6.0 million gal/day of domestic wastewater using a slow-rate trickling-filter system. The six secondary sedimentation tanks employed at the facility each operates with an average overflow rate of 800 gal/ft^2·day. Given the following laboratory batch flux data, estimate (A) underflow velocity (U_b), (B) limiting solids flux (SF_L), (C) maximum underflow solids concentration (X_u), and (D) minimum solids settling area. Assume that the mixed-liquor suspended solids (MLSS) concentration in the trickling-filter effluent has an average concen-

MLSS (mg/liter)	Flow (ft/h)
600	21.2
1,200	21.0
1,800	19.0
2,500	18.0
3,200	17.0
4,600	12.0
5,800	9.0
6,900	6.0
8,100	4.0
10,000	3.0
13,000	1.7
21,000	0.8
32,000	0.4
45,000	0.2
59,000	0.1
68,000	0.05

tration of 3100 mg/liter and that the secondary clarification tank recycle ratio (Q_r/Q) is 0.8.

3.2 The Bambari Sewer Improvement District has employed a gravity thickener to increase the solids content of its secondary sludge from 1.4 to 5.0 percent solids before pumping the sludge to an aerobic digestion tank. Given the following batch solids flux data, estimate the limiting solids flux, the bulk discharge velocity of the sludge U_B, and the cross-sectional area of the thickener.

Solids concentration (mg/liter)	Initial settling velocity (ft/h)
1,800	12.0
2,400	11.2
3,900	9.1
5,600	7.2
7,100	4.9
9,000	3.4
11,200	2.0
14,500	1.3
22,000	0.7
31,000	0.4
44,000	0.2
58,000	0.1
69,000	0.05

3.3 The Pokomo City Wastewater Treatment Plant is currently employing dissolved air floatation (DAF) to thicken its combined primary and secondary sludge from 0.8 to 5.0 percent solids. If the daily combined sludge flow rate is 340,000 gal/day, estimate the value for the following DAF design parameters for systems operated with and without recycle: (A) pressure (atm), (B) surface area (ft^2) of DAF system, and (C) solids loading rate (lb/ft^2·day). Assume that through bench-scale tests the following data were generated:

Optimal A/S ratio: 0.02 ml/mg
Water temperature: 62°F
Air solubility: 19.0 ml/liter
Recycle system pressure: 60 lb/in^2
Fraction of water saturation: 0.60
Surface loading rate: 0.30 gal/ft^2·min

3.4 The Pangani County Sewer Improvement District is considering purchasing a solid-bowl decanter centrifuge to increase the solids content of its thickened sludge from 1.2 to 6.0 percent. If the centrifuge manufacturer claims that the solids capture efficiency of the device is 98 percent, what concentration of solids should be expected in the return centrate?

3.5 The Cushite City Water Reclamation Plant recently has purchased a disk-nozzle centrifuge to thicken its secondary sludge from a 1.6 to 7.0 percent solids content. If the flow rate of sludge to the new centrifuge system is 360,000 gal/day, estimate the return centrate flow rate (gallons per day) if the solids capture efficiency of the device is 96 percent. If the wastewater treatment facility has centrate sump pumps with rated capacities of 80 gal/min, how many pumps must be operating continuously at the plant?

3.6 The Malindi City public works director has contracted with a private environmental laboratory to develop a physical and chemical profile of the biosolids that are being applied on local agricultural fields. If a 1-lb sample of wet biosolids that is oven dried at 104°C for 6 hours has a resulting dry weight of 0.12 lb, determine the (A) specific gravity of the dry sludge and (B) specific gravity of the wet sludge. Results from heating the oven-dried biosolids sample for 6 hours in a muffle furnace set at 550°C indicated that the dried biosolids are 80 percent volatile (organic) and 20 percent fixed (mineral) solids. Assume that the specific gravity of the volatile and fixed solids fractions of the dried biosolids is 1.0 and 2.8, respectively.

3.7 The Meru County Water Reclamation Plant is currently generating 4800 lb/day of dry sludge from its primary and secondary wastewater treatment systems. If the combined sludge is to be thickened from a 1.8 to 6.5 percent solids content in a centrifugal thickener, what is the average volume flow rate of the thickened sludge? Assume that the specific gravity of the combined thickened sludge is 1.04.

3.8 The Turkana City Wastewater Treatment Plant employs anaerobic digestion to stabilize its combined primary and secondary sludge. If the combined thickened sludge is pumped to an anaerobic digestion system at the rate of 16,000 gal/day, estimate the mass of anaerobic sludge generated daily from the anaerobic treatment system. From sampling the anaerobic digestion operation, it has been found that the average ultimate biochemical oxygen demand (BOD_L) of the influent sludge is 55,000 mg/liter, whereas the average effluent BOD_L concentration was estimated to be 7500 mg/liter. Assume that the total volume of the anaerobic digester is 250,000 gal and that the system is operated without sludge recycle. The applicable kinetic coefficients are (A) yield coefficient $Y = 0.05$ lb/lb and (B) decay constant $k_d = 0.02\ \text{day}^{-1}$.

3.9 The Laisamis City Wastewater Treatment Plant is currently treating 18,000 gal/day of thickened sludge using anaerobic digestion. If the ultimate biochemical oxygen demand (BOD_L) of the influent sludge is 62,000 mg/liter and the biochemical oxygen demand (BOD_L) of the effluent sludge is 8000 mg/liter, estimate the volume of the digester and the volume of biogas produced. Assume that the anaerobic digester is operating as a continuously mixed reactor without recycle and that a mean cell residence time of 20 days is maintained. In addition, assume that the biogas is 70 percent methane (by volume) and that the following kinetic coefficients apply: (A) $Y = 0.05$ lb/lb and (B) $k_d = 0.02\ \text{day}^{-1}$.

3.10 The Tabora City Water Reclamation Plant is operating a thermophilic anaerobic digester system (140°F) to stabilize its thickened sludge. If the influent sludge has an average temperature of 68°F and the ambient air temperature at the facility is 73°F, what are the average annual heating costs for the digestion system? Assume that the sludge flow rate is 28,000 gal/day and that the average energy purchase price is \$18 per decatherm ($10 \cdot 10^6$ Btus). Assume that the digester walls and floor are in contact with the ground, which has an average temperature of 53°F. Other design specifications for the digester are given as follows:

Digester dimensions:

Diameter: 100 ft
Side depth: 25 ft
Middle depth: 30 ft
Digester material: Concrete
Digester floor: Conical shape
Digester cover: Flat circular fixed roof

Heat transfer coefficients:

Dry earth embankment for
the entire depth and floor: 0.12 Btu/ft^2·h·°F
Roof exposed to air: 0.17 Btu/ft^2·h·°F
Specific heat of biosolids: 1.0 Btu/lb·°F (4200 J/kg·°C)

3.11 The Kigali City Wastewater Treatment Plant is operating a 250,000-gal thermophilic anaerobic digester to recover energy from its sludge stabilization process. To minimize possible digester upset due to variations in organic matter loading, the plant desires to increase the digester pH and buffering capacity. If the CO_2 concentration in the biogas is to be maintained at a concentration of no greater than 40 percent (by volume), estimate the amount of sodium bicarbonate that should be added to increase the pH of the digester from 6.6 to 7.4.

3.12 The Odongo County Sewer Improvement District is currently employing biogas recirculation to mix the contents of its anaerobic digester. If the digester volume is 175,000 gal, estimate the rate of power transferred to the liquid if the gas recirculation rate is 150 ft^3/min (acfm) at a pressure of 15 lb/ft^2 (gauge). Assume that the liquid surface of the digester is under 6 in of water pressure and that atmospheric pressure is 14.7 lb/in^2.

3.13 The Odongo County Sewer Improvement District (see Prob. 3.12) has found that the gas recirculation system employed to mix the anaerobic digester system is inadequate. Assuming that the surface and inlet gas pressures are 6 in of water pressure and 15 lb/in^2 (gauge), respectively, estimate the minimum gas flow rate in cubic feet per minute per cubic foot of digester ($ft^3/min \cdot ft^3$) needed to achieve a velocity gradient of 70 s^{-1}. Assume that the viscosity of the digester contents μ is $1.7 \cdot 10^{-5}$ lb·s/ft^2.

3.14 The Garsen City Wastewater Treatment Plant is currently generating 26,000 gal/day of thickened combined sludge. If the plant chooses to employ anaerobic digestion to reduce the pathogen content of the sludge, what is the minimum digester volume needed if the system is operated without recycle? Assume that the digester will be operated at mesophilic temperatures (35°C) and maintain a mean cell residence time of 20 days.

3.15 The Gitega City Water Reclamation Plant desires to construct a thermophilic anaerobic digester system to recover methane gas for use in cogeneration of hot water and electricity. If the digester is to receive 25,000 gal/day of sludge having an average volatile solids content of 4.2 percent, estimate the minimum digester volume. Assume that the digester volume will be designed to receive a maximum solids loading of 0.25 lb of volatile suspended solids per cubic foot of digester per day (i.e., 0.25 lb VSS/ft^3·day).

3.16 The Nyala County Sewer Improvement District is currently treating 22,000 gal/day of thickened sludge in its thermophilic anaerobic digester system. Assuming that there is a 40 percent reduction in sludge volume during sludge digestion, estimate the minimum volume of the digester if the mean cell residence time is maintained at 12 days.

3.17 The Eldoret City Wastewater Treatment Plant is currently generating 3700 lb/day of primary sludge (dry-mass basis) and 1800 lb/day of secondary sludge (dry-mass basis). The sludges from the two unit operations are combined in a gravity thickener, where they are concentrated to a solids content of 4 percent having a specific gravity of 1.03. If the thickened sludge is to be stabilized in a semicontinuously operated aerobic digester, estimate the following:

A. Volume flow rate of thickened sludge to the digester
B. Solids retention time
C. Mass of volatile solids removed per day
D. Oxygen and air requirement for stabilization

E. Volume of digester

F. Air requirement per 1000 ft^3 of digester volume

Assume that the following data apply:

Digester design will be based on a minimum winter temperature of 15°C.

Digester must achieve at least 40 percent volatile solid removal at 15°C.

Suspended solids in digester are 70 percent of thickened sludge concentration.

Volatile fraction of the influent sludge solids is 0.8.

Stabilized and thickened sludge will have a solids content of 5 percent.

Supernatant will have a solids concentration of 400 mg/liter.

Density of air is 0.075 lb/ft^3.

Air contains 23.2 percent oxygen by weight.

Diffused air oxygen transfer efficiency is 12 percent.

3.18 The Kitale County Wastewater Treatment Plant is considering the use of thermophilic aerobic digestion to stabilize its thickened sludge flow rate of 30,000 gal/day. Given that nitrification is sufficiently inhibited at thermophilic temperatures (55°C), estimate the minimum digester volume assuming that the following data apply:

Q_i (influent sludge flow rate to digester): 30,000 gal/day

X_i (influent suspended solids concentration): 35,000 mg/liter (4 percent)

y (fraction of influent BOD_5 consisting of raw primary sludge): 0.50 (50 percent)

S_i (influent BOD_5): 67,000 mg/liter

X (digester suspended solids concentration): 22,000 mg/liter

K_d (reaction rate constant): 0.37 day^{-1}

P_v (volatile fraction of digester suspended solids concentration): 0.85 (85 percent)

θ_c (mean cell residence time): 6 days

3.19 The Kitale County Wastewater Treatment Plant (see Prob. 3.18) desires to package and sell its biosolids to the public as a low-cost fertilizer/soil conditioner. To ensure minimal levels of pathogens in the treated biosolids, the facility is considering the use of biosolids pasteurization. Assuming that the temperature of the preheated sludge exiting the heat exchanger is 45°C and the final pasteurization temperature is 75°C, estimate the energy required by the boiler (Btu/h) to pasteurize the sludge if the required heating time is 40 minutes and the boiler energy conversion efficiency is 80 percent. Other relevant information is given as follows:

Sludge flow rate: 30,000 gal/day

Specific gravity of sludge: 1.03

Heat capacity of sludge: 1.2 Btu/lb·°F

3.20 The Lamu City Solid Waste Advisory Board has suggested that aerated static pile composting be employed to stabilize dewatered sludge from several of the local wastewater treatment plants. To ensure proper compost operation, the design engineer has recommended that the initial compost mixture should have a carbon to nitrogen ratio (C/N) of 30 (weight basis). If rice hulls (C/N ratio of 160) are to be employed as a compost amendment, estimate the pounds of rice hulls required to be mixed with dewatered sludge (C/N ratio of 12) if the wastewater treatment plants collectively generate 8500 lb of dewatered sludge per day (dry-weight basis). Assume that the following data apply:

Moisture content of sludge: 80 percent

Moisture content of rice hulls: 50 percent

Nitrogen content of sludge: 5.0 percent (dry-weight basis)

Nitrogen content of rice hulls: 0.8 percent (dry-weight basis)

3.21 The Kigoma County Materials Recovery Facility is currently receiving 120 tons of dewatered wastewater treatment sludge per day from the county's municipal wastewater treatment plants. If the wastewater sludge has a solids content of 30 percent and a bulk density of 1700 lb/yd^3, estimate the minimum area required for the composting operation. Assume that the composting curing area is equivalent to the size of the composting area and that the area needed for access roads, turn-arounds, runoff collection, etc. is approximately 60 percent of the total area. Preliminary tests have shown that the volumetric ratio of bulking agent (e.g., woodchips) to sludge necessary for adequate composting using an 8-ft aerated static pile system is 3:1.

3.22 The Tunduru City Water Reclamation Plant is currently treating 4 million gal/day of municipal wastewater using both primary and secondary treatments. If the total solids content from sludge generated during primary and secondary treatment is approximately 3.5 and 1.2 percent, respectively, estimate the pounds of conditioning polymer needed and the minimum volume of the polymer day tank required for proper sludge conditioning. Assume that the combined primary and secondary sludge is thickened to 4 percent total solids prior to polymer conditioning and that polymer is added at a rate of 25 lb/dry ton of solids. It may also be assumed that polymer solution in the day tank is maintained at 1.0 percent polymer concentration (weight basis) and that sludge will only be processed 5 days per week.

3.23 The Makurdi County Sewer Improvement District is currently producing 32,000 gal/day of thickened sludge having a solids content of 3.0 percent. If a conventional sand drying bed is to be employed for dewatering, estimate the minimum area needed if climatic conditions will permit use of the drying beds for 12 months per year. Assume that the following conditions apply:

Pan evaporation rate: 8.0 in/month

Moisture removed by drainage: 22 percent

Final sludge solids content: 25 percent
Reduction factor (k_e): 0.6
Design safety factor: 1.8

3.24 The Makurdi County Sewer Improvement District (see Prob. 3.23) has decided to replace its sand drying beds with a paved drying bed to dewater its thickened wastewater treatment sludge from a 3 to 25 percent solids content. Estimate the required paved bed drying area if the solids content of the sludge after the initial decanting of moisture has been estimated at 10 percent and the average thickened sludge production rate is 32,000 gal/day. Assume that the monthly precipitation and evaporation rates are 0.25 and 0.45 ft, respectively, and that the reduction factor for sludge evaporation k_e has been measured to be 0.8.

3.25 The Gusau City Wastewater Treatment Plant is employing a counterflow sludge dryer to reduce the weight of its sludge prior to rail transport. If the temperature of the dryer exit gas is 200°F and it contains 31 percent moisture (by volume), estimate the relative humidity and percent saturation of the exiting gas. Assume that the total pressure is maintained at 760 mmHg.

3.26 The Gaborone County Water Reclamation Plant has purchased a cocurrent flow dryer to reduce the moisture content of its biosolids prior to barge transport. If the exit gas from the dryer contains 22 percent moisture (by volume), what is the dew point of this gas if the gas can be assumed to be saturated with moisture? Assume that the total pressure is maintained at 760 mmHg.

3.27 The Bissau County Wastewater Treatment Plant has purchased a crossflow biosolids dryer to reduce the material's weight prior to truck transport. If the air entering the dryer has a temperature of 290°F and contains 10 percent moisture (volume basis), estimate the pounds of water removed per one thousand cubic feet (1000 ft^3) of hot air entering the dryer. Assume that after a 10-s residence time, the air is released from the dryer at a temperature of 190°F and that it is saturated with moisture.

3.7 References

1. Bastian, R. K. (1997), "The Biosolids (Sludge) Treatment, Beneficial Use and Disposal Situation in the USA," *European Water Pollution Control,* pp. 62–79.
2. Butler, J. N. (1982), *Carbon Dioxide Equilibria and Their Applications,* Addison-Wesley Publishing Company, Reading, Mass.
3. Chaney, R. L. (1984), "Potential Effects of Sludge-Borne Heavy Metals and Toxic Organics on Soils, Plants, and Animals, and Related Regulatory Guidelines," in *Workshop on the International Transportation, and Utilization or Disposal of Sewage Sludge, Including Recommendations* (December 12–15), Final Report PNSP/85-01, Pan American Health Organization, Washington, DC, p. 56.
4. Corey, R. B. (1992), "Phosphorus Regulations: Impact of Sludge Regulations" *Crops Soils Newsletter* 20:5–10.
5. Davis, M. L., and D. A. Cornwell (1991), *Introduction to Environmental Engineering,* McGraw-Hill, New York.

6. Dick, R. I. (1972), "Sludge Treatment," in *Physicochemical Processes for Water Quality Control,* Wiley, New York.
7. Eckenfelder, W. W. (1989), *Industrial Water Pollution Control,* 2d ed., McGraw-Hill, New York.
8. Garvey, D., C. Guarino, and R. Davis (1993), "Sludge Disposal Trends Around the Globe," *Water/Engineering and Management,* pp. 17–20.
9. Grady, C. P. L., and H. C. Lim (1980), *Biological Wastewater Treatment,* Marcel Dekker, New York.
10. Hartenstein, R. (1981), "Sludge Decomposition and Stabilization," *Science* 212:743–749.
11. Haug, R. T. (1980), *Compost Engineering, Principles and Practices,* Ann Arbor Science Publications, Ann Arbor, Mich.
12. Haug, R. T. (1986), "Composting Process Design Criteria," *Biocycle,* September 1986, pp. 36–39.
13. Jewell, W. J. (1980), "Use and Treatment of Municipal Wastewater and Sludge in Land Reclamation and Biomass Production Projects: An Engineering Assessment," in *Utilization of Municipal Wastewater and Sludge: An Engineering Assessment for Land Reclamation and Biomass Production,* EPA 430/9-81-012, Washington, pp. 448–480.
14. Jewell, W. J., and R. M. Kabrick (1980), "Autoheated Aerobic Thermophilic Digestion with Air Aeration," *Journal of Water Pollution Control Federation* 52:512–523.
15. Jewell, W. J., R. M. Kabrick, and J. A. Spada (1980), "Autoheated Aerobic Thermophilic Digestion with Air Aeration," U.S. Environmental Protection Agency Final Report, EPA-600/S2-82-023.
16. Jewell, W. J. (1988), "Anaerobic Sewage Treatment," *Environmental Science and Technology* 22:14–21.
17. Linsley, R. K., J. B. Fanzini, D. L. Freyberg, and G. Tchobanoglous (1992), *Water Resource Engineering,* McGraw-Hill, New York.
18. Loehr, R., W. Jewell, J. Novak, W. Clarkson, and G. Friedman (1979), *Land Application of Wastes,* Vol. 2., Van Nostrand Reinhold, New York.
19. Loehr, R. C. (1977), *Land as a Waste Management Alternative,* Ann Arbor Science, Inc., Ann Arbor, Mich.
20. Logan, T. J., and R. L. Chaney (1983), "Utilization of Municipal Wastewater and Sludge on Land: Metals," in A. L. Page et al. (eds.), *Utilization of Municipal Wastewater and Sludge on Land, Denver, CO,* Univ. of California, Riverside, pp. 235–326.
21. McCabe, W. L., J. C. Smith, and P. Harriot (1993), *Unit Operations of Chemical Engineering,* McGraw-Hill, New York.
22. McCarty, P. L. (1968), "Anaerobic Treatment of Soluble Wastes," in E. F. Gloyna and W. W. Eckenfelder (eds.), *Advances in Water Quality Improvement,* University of Texas Press, Austin.
23. McFarland, M. J., and W. J. Jewell (1989), "In Situ Control of Sulfide Emissions during the Thermophilic Methane Fermentation Process," *Water Research* 23(12):1579–1591.
24. McFarland, M. J., and W. J. Jewell (1990), "The Effect of Sulfate Reduction on the Thermophilic (55°C) Methane Fermentation Process," *Journal of Industrial Microbiology,* 5:247–258.
25. Parkin, W. P. (1993), *The Complete Guide to Environmental Law,* Specialty Technical Publishers, New York.
26. Peavy, H. S., D. R. Rowe, and G. Tchobanoglous (1985), *Environmental Engineering,* McGraw-Hill, New York.
27. Perry, R. H., and C. H. Chilton (1973), *Chemical Engineers Handbook,* McGraw-Hill, New York.
28. Porter, M. C. (1990), *Handbook of Industrial Membrane Technology,* Noyes Publications, New York.
29. Reed, S. C. (1987), "Sludge Freezing for Dewatering," *Biocycle* 28(1).
30. Reed, S., R. Crites, and E. Middlebrooks (1994), *Natural Systems for Waste Management and Treatment,* McGraw Hill, New York.

31. Ryan, J., and R. Chaney (1993), "Regulation of Municipal Sewage Sludge under the Clean Water Act Section 503: A Model for Exposure and Risk Assessment for MSW-Compost," in *Science and Engineering of Composting,* Renaissance Publications, Worthington, OH.
32. Tchobanoglous, G. (1981), *Wastewater Engineering: Collection and Pumping of Wastewater,* McGraw-Hill, New York.
33. Tchobanoglous, G. (1991), *Wastewater Engineering: Treatment Disposal and Reuse,* 3d ed., McGraw-Hill, New York.
34. U.S. Army Corps of Engineers (1987), "Wetlands Delineation Manual," Technical Report Y-87-1, Waterways Experiment Station, Vicksburg, MI.
35. U.S. Environmental Protection Agency (1975), "Process Design Manual for Suspended Solids Removal," EPA/625/1-75/003a, Washington.
36. U.S. Environmental Protection Agency (1976), "Anaerobic Sludge Digestion: Operations Manual," EPA/430/9-76/001, Washington.
37. U.S. Environmental Protection Agency (1979), "Fate of Priority Pollutants in Publicly Owned Treatment Works," EPA/440/1-79/300, Washington.
38. U.S. Environmental Protection Agency (1979), "Process Design Manual for Sludge Treatment and Disposal," EPA/625/1-79/011, Washington.
39. U.S. Environmental Protection Agency (1982), "Design Manual for Dewatering Municipal Wastewater Sludges," EPA/625/1-82/014, Washington.
40. U.S. Environmental Protection Agency (1983), "Process Design Manual for Land Application of Municipal Sludge," EPA 625/1-83-016, Washington.
41. U.S. Environmental Protection Agency (1984), "Environmental Regulations and Technology, Use and Disposal of Municipal Wastewater Sludge," EPA 625/10-84-003, Washington.
42. U.S. Environmental Protection Agency (1984), "Handbook: Septage Treatment and Disposal," EPA/625/6-84/009, Washington.
43. U.S. Environmental Protection Agency (1985), "Design Manual: Odor and Corrosion Control in Sanitary Sewerage Systems and Treatment Plants," EPA/625/1-85/018, Washington.
44. U.S. Environmental Protection Agency (1985), "Handbook Estimating Sludge Management Costs," EPA-625/6-85010, Washington.
45. U.S. Environmental Protection Agency (1986), "Regrowth of *Salmonellae* in Composted Sewage Sludge," USEPA 600/2-86/106, Washington.
46. U.S. Environmental Protection Agency (1987), "Anaerobic Digester Mixing Systems: Design Information Report," *Journal of the Water Pollution Control Federation,* 59(3):162–170.
47. U.S. Environmental Protection Agency (1987), "EPA Design Information Report—Design, Operational, and Cost Considerations for Vacuum Assisted Sludge Dewatering Bed Systems," *Journal of Water Pollution Control Federation,* 59:228.
48. U.S. Environmental Protection Agency (1987), "Design Manual for Dewatering Municipal Wastewater Sludges," EPA/625/1-87/014, Washington.
49. U.S. Environmental Protection Agency (1989), "Summary Report: In-Vessel Composting of Municipal Waste Water Sludge," EPA/625/8-89/016, Washington.
50. U.S. Environmental Protection Agency (1990), "Autothermal Thermophilic Aerobic Digestion of Municipal Wastewater Sludge," EPA/625/w-90/007, Washington.
51. U.S. Environmental Protection Agency (1990), "Environmental Regulations and Technology: Autothermal Thermophilic Aerobic Digestion of Municipal Wastewater Sludge," EPA/625/10-90/007, Washington.
52. U.S. Environmental Protection Agency (1990), "40 CFR Part 503, National Sewage Sludge Survey: Availability of Information and Data and Anticipated Impacts on Proposed Regulations," *Federal Register* 55(218):47210–47283.
53. U.S. Environmental Protection Agency (1991), "Residential and Commercial Toxic Pollutant Loadings and POTW Removal Efficiency," PB93-209872, Washington.
54. U.S. Environmental Protection Agency (1992), "Determining Wastewater User Service Charge Rates—A Step-by-Step Manual," EPA/832/B-92/003, Washington.
55. U.S. Environmental Protection Agency (1992), "Environmental Regulations and Technology—Control of Pathogens and Vector Attraction in Sewage Sludge," EPA/625/R-92/013, Washington.

56. U.S. Environmental Protection Agency (1993a), "Technical Support Document for Land Application of Sewage Sludge," Vol. I, PB93-110575; Vol. II, PB93110583, Washington.
57. U.S. Environmental Protection Agency (1993), "Guides to Pollution Prevention: Municipal Pretreatment Programs," EPA/625/R-93/006, Washington.
58. U.S. Environmental Protection Agency (1993), "Domestic Septage Regulatory Guidance—A Guide to The EPA 503 Rule," EPA/832/B-92/005, Washington.
59. U.S. Environmental Protection Agency (1993), "Regulatory Impact Analysis of the Part 503 Sewage Sludge Regulation," EPA/821/R-93/006, Washington.
60. U.S. Environmental Protection Agency (1994), "A Plain English Guide to the EPA Part 503 Biosolids Rule," EPA/832/R-93/003, Washington.
61. U.S. Environmental Protection Agency (1994), "Composting Yard Trimmings and Municipal Solids Waste," EPA/530/R-94/003, Washington.
62. U.S. Environmental Protection Agency (1994), "Biosolids Recycling: Beneficial Technology for a Better Environment," EPA/832/R-94/009, Washington.
63. U.S. Environmental Protection Agency (1994), "Guide to Septage Treatment and Disposal," EPA/625/R-94/002, Washington.
64. U.S. Environmental Protection Agency (1995), "Ground-Water and Leachate Treatment Systems," EPA/625/R-94/005, Washington.
65. U.S. Environmental Protection Agency (1995), "Process Design Manual: Land Application of Sewage Sludge and Domestic Septage," EPA/625/R-95/001, Washington.
66. U.S. Environmental Protection Agency (1995), "Process Design Manual: Surface Disposal of Sewage Sludge and Domestic Septage," EPA/625/R-95/002, Washington.
67. U.S. Environmental Protection Agency (1995), "Part 503 Implementation Guidance," EPA/833/R-95/001, Washington.
68. U.S. Environmental Protection Agency (1996), "Technical Support Document for the Round Two Sewage Sludge Pollutants," EPA/822/R-96/003, Washington.
69. U.S. Environmental Protection Agency (1999), "Environmental Regulations and Technology—Control of Pathogens and Vector Attraction in Sewage Sludge," EPA/625/R-92/013, Washington.
70. Vesilind, P. A., G. C. Hattman, and E. T. Shene (1986), *Sludge Management and Disposal for the Practicing Engineer,* Lewis Publishers, New York.
71. Viessman, W., and M. J. Hammer (1985), *Water Supply and Pollution Control,* 4th ed., Harper Collins, New York.
72. Viessman, W., and C. Welty (1985), *Water Management,* Harper & Row, New York.
73. Water Pollution Control Federation (1983), "Sludge Dewatering, Manual of Practice," no. 20.
74. Water Pollution Control Federation (1985), "Sludge Stabilization, Manual of Practice," FD-9.
75. Water Pollution Control Federation (1985), "Clarifier Design, Manual of Practice," FD-8.
76. Water Pollution Control Federation (1987), "Activated Sludge, Manual of Practice," OM-9.
77. Water Pollution Control Federation (1987), "Anaerobic Sludge Digestion, Manual of Practice," no. 16, 2d ed.
78. Water Pollution Control Federation (1988), "O&M of Trickling Filters, RBCs and Other Related Processes, Manual of Practice," OM-10.
79. Water Pollution Control Federation (1988), "Aeration, Manual of Practice," FD-13.
80. Water Pollution Control Federation (1988), "Sludge Conditioning, Manual of Practice," FD-14.
81. Weber, W. (1972), *Physicochemical Processes for Water Quality Control,* Wiley, New York.

Chapter 4

Control of Biosolids Quality

4.0 Introduction

The most cost-effective approach for controlling biosolids quality is to limit the types and amounts of pollutants discharged into the local sewer system or treatment plant headworks. Publicly owned treatment works (POTWs) are responsible for limiting, where necessary, the character and volume of pollutants being discharged into their sewage-collection systems. In addition to improving the opportunities for biosolids and wastewater recycling, other reasons for limiting the discharge of certain pollutants include protection of the treatment facility, receiving water quality, and worker health and safety. POTWs should control the discharge of pollutants to their sewage-collection systems through the development and implementation of technically based local wastewater discharge limits [26].

4.1 The Clean Water Act

In 1948, the U.S. Congress enacted the original Federal Water Pollution Control Act, which authorized the U.S. Surgeon General of the Public Health Service to work with federal, state, and local agencies in preparing comprehensive pollution control plans for interstate rivers [16,20]. The 1948 legislation also authorized a financial assistance program specifically designed to aid local governments desiring to improve their sewage works programs [16]. Although approved by Congress, no federal funds were appropriated for this purpose.

Since its passage, the 1948 Federal Water Pollution Control Act has been amended many times. Two of the most important sets of amendments occurred in 1972 and in 1977 [16,20]. The 1972 amendments (PL 92-500) not only increased the level of federal funding appropriated for the improvement of local wastewater treatment facilities and sewage collection systems but also established a regulatory mechanism requiring technology-based POTW effluent standards as well as a nationwide permit system for all point sources of water pollution.

The nationwide permit system, known as the National Pollutant Discharge Elimination System (NPDES), requires all persons responsible for wastewater discharges to obtain a permit prior to releasing any pollutant into the waters of the United States from any point source. Since POTWs normally discharge treated wastewater to water bodies, POTWs with a nonzero plant effluent flow rate must obtain a NPDES permit prior to discharging their treated wastewater. NPDES permits are issued only after the opportunity of a public hearing and are effective for terms not exceeding 5 years [23].

In 1977, the Federal Water Pollution Control Act was again amended to specifically address deficiencies in the legislation. The 1977 amendments (PL 95-217) mandated that individual states acquire primacy over all water quality and water use issues and that technology-based effluent standards be established for toxic pollutants [16]. The original Federal Water Pollution Control Act complete with its subsequent amendments is now referred to as the *Clean Water Act* (CWA).

In order to protect the operation of POTWs and to prevent the discharge from POTWs of pollutants that have not received adequate treatment, the CWA requires the U.S. Environmental Protection Agency (EPA) to adopt and amend, as necessary, national pretreatment standards for industrial and other nondomestic discharges into POTW sewage-collection systems [26,34]. It should be noted that nondomestic sewer users discharging to a POTW sewage-collection system are not required under the CWA to obtain a NPDES permit. Instead, POTWs may impose wastewater discharge restrictions or “pretreatment standards” on industrial and other nondomestic sewer users to ensure compliance with their own NPDES permit and its effluent discharge limitations. These standards may be concentration- or mass-based pollutant limitations placed on specific industrial and commercial wastewater discharges. The POTW has the authority to issue pretreatment permits (through its sewer ordinance authority) to its nondomestic sewer users in order to establish a regulatory mechanism to enforce local wastewater discharge limits. A typical wastewater treatment system employed by the metal finishing industry to meet its pretreatment discharge standards is shown in Fig. 4.1. The wastewater effluent from such a system would be sampled routinely by POTW pretreatment personnel to verify compliance with pretreatment discharge standards.

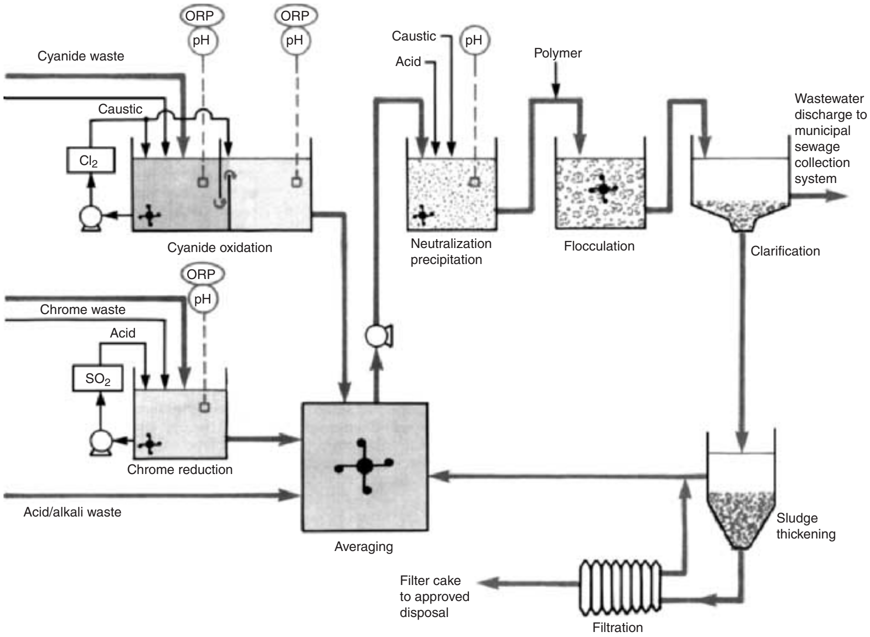


Figure 4.1 Conventional pretreatment system employed in the metal finishing industry.

In addition to ensuring compliance with national pretreatment discharge standards for their nondomestic sewer users, POTWs may choose to impose more stringent standards on their industrial users. Some of the reasons for POTWs to be even more stringent in establishing pretreatment discharge limits than the national standards include the following:

1. The need to prevent introduction of pollutants into POTWs that would interfere with equipment or endanger personnel
2. The need to prevent introduction of pollutants into POTWs that would pass through (i.e., would not be adequately treated prior to discharge) or be incompatible with POTW operations
3. The desire to improve the opportunities to recycle wastewater and/or biosolids

4.1.1 Industrial pretreatment discharge standards

The national pretreatment discharge standards take two forms; prohibitions on discharges to POTWs and categorical discharge standards [26]. The EPA prohibits introduction into POTWs of any pollutant that may pass though the system without adequate treatment or cause interference with POTW operations. Although the EPA leaves implementation of this general rule to the individual POTWs, it has applied more specific discharge prohibitions to the following type of pollutants:

1. Pollutants that would create a fire or explosive hazard in the collection system or at the POTW
2. Corrosives that would cause structural damage to the POTW
3. Solids or viscous pollutants in amounts that would obstruct flow at the POTW
4. Heat in amounts that would inhibit biological activity at the POTW
5. Any pollutant at a flow rate that would cause interference with POTW operation

In addition to the general discharge prohibitions, certain sewer users also must conform to national categorical standards for existing and new sources of industrial discharges [26]. These standards, which are contained in the U.S. Code of Federal Regulations (40 CFR Parts 405 through 471), specify the quantities and/or concentrations of pollutants that may be discharged into a municipal sewer by industrial and other nondomestic sewer users [23]. Each existing or new nondomestic sewer user must be in compliance with the local standard prior to discharging pollutants to the POTW sewage-collection system. In addition to meeting the national pretreatment discharge standards, the POTW can require that any nondomestic sewer user that uses, manufactures, stores, handles, or discharges any pollutant listed as toxic under Section 307 of the CWA or any pollutant listed as hazardous under Section 311 of the CWA establish best management practices (BMPs) to limit municipal sewer discharges. BMPs are in-house procedures aimed at preventing or minimizing the potential release of these substances [39]. In many cases, BMPs are incorporated into the industrial pretreatment permit (or sewer ordinance) issued to the sewer user by the POTW.

4.1.2 Development of wastewater discharge limits

The National Pretreatment Program as implemented under the CWA and General Pretreatment Regulations (40 CFR Part 403) is designed to control the introduction of nondomestic wastes to POTWs. The specific objectives of the program are to protect POTWs from pass-through and interference, to protect receiving waters, and to improve opportunities to recycle biosolids. The general pretreatment regulations require that each POTW with an average daily flow rate of at least 5 million gal/day (5 MGD) develop a pretreatment program that establishes and enforces local wastewater discharge limits generally called *local limits* [26]. POTWs are required to develop and enforce local limits as mandated by 40 CFR Part 403.5 and 40 CFR Part 403.8 [16,26]. Local limits may be concentration- or mass-based industrial/commercial discharge limits that result in a pollutant headworks loading (lb/day) that the POTW can treat safely while maintaining regulatory compliance with its effluent wastewater discharge limits and biosolids quality requirements. In addition to meeting local discharge limits, specific industries and nondomestic sewer users must comply with categorical pretreatment standards that are established under 40 CFR Part 403.6 of the CWA. Examples of specific categorical standards are presented in Table 4.1. The enforcement of local wastewater discharge limits is normally the responsibility of the POTW issuing the pretreatment discharge permit and typically includes routine sampling inspections conducted to verify compliance [16,23,28].

To establish local wastewater discharge limits, POTWs should use the best available technical information to identify pollutants of concern (POCs) and the maximum allowable headworks loading that can be treated adequately by their facility accounting for the final wastewater effluent discharge and biosolids quality. Identification of the POCs typically involves the following series of steps:

- Preliminary data collection
- Development of a pretreatment sampling plan
- Determining the POCs
- Establishing a sampling program for POCs

TABLE 4.1 Examples of Categorical Pretreatment Standards*

Industrial process	Code of Federal Regulations†
Electroplating/metal finishing	40 CFR Parts 413/433
Electric and electronic components	40 CFR Part 469
Gum and wood chemicals	40 CFR Part 454
Industrial laundries	40 CFR Part 444
Inorganic chemicals manufacturing	40 CFR Part 415
Metal molding and casting	40 CFR Part 464
Rubber manufacturing	40 CFR Part 428
Soap and detergent manufacturing	40 CFR Part 417
Textile mills	40 CFR Part 410
Timber products processing	40 CFR Part 429

*Adapted from ref. [26].

†Categorical Standards for all Industrial Subcategories can be found in 40 CFR Parts 405 through 471.

Each of these steps is described in the following sections.

4.1.2.1 Preliminary data collection. The first step in identifying the POCs is to collect background information from the state water quality office and the POTW's National Pollutant Discharged Elimination System (NPDES) permit. At a minimum, the following background information should be obtained:

1. Identification of POTW effluent wastewater receiving water body together with its present quality. Particular emphasis should be placed on the following items:
    - State water quality standards
    - Hardness of receiving water
    - Preexisting background pollutant levels
    - Aquatic life uses
    - Minimum flow rates
2. Specific chemical limitations identified in the POTW's NPDES permit for water quality and biosolids.
3. Historical chemical analyses on the POTW's influent wastewater, effluent wastewater, and biosolids collected over the past 3 to 5 years.
4. Data on POTW plant design and monthly average flow rate.

Following the collection of background data, a pretreatment sampling plan must be developed to characterize the influent wastewater quality (including any septage and landfill leachate discharges to the POTW).

4.1.2.2 Development of a pretreatment sampling plan. The pretreatment sampling plan, which must be submitted to the U.S. Environmental Protection Agency (EPA) as part of the permitted pretreatment program, outlines the procedure(s) that the POTW will use to collect and analyze wastewater quality data [23,26]. The sampling plan should include, at a minimum, the following items:

- Identification of sampling locations
- Parameters to be sampled at each sampling location
- Sample type for each parameter
- Identification of containers, preservatives, holding times, and shipping/storage procedures for all parameters
- Identification of methods required for the analysis of each parameter
- Identification of data to be recorded for each sample

(a)

(b)

Figure 4.2 (*a*) Wastewater sampling. (*Courtesy of Water Environment Federation.*) (*b*) Dewatered sludge sampling. (*Courtesy of Water Environment Federation.*)

Since identification of the POCs is an iterative process, the pretreatment sampling plan should include guidelines for periodic updating and review. These guidelines should describe the types and frequency of sampling of POTW wastewater influent, wastewater effluent, and biosolids, as well as the monitoring of industrial and other nondomestic sewer users [21,23,26,28].

4.1.2.3 Determining the POCs. Once the pretreatment sampling plan has been approved by the regulatory authority, the POTW should conduct a sufficient number of sampling events to establish a database of typical wastewater and biosolids quality to be used in identifying the POCs. The significance of being designated a POC is that each POC is required by law to be carried through the complete local wastewater discharge limits sampling and data evaluation [21,26]. It should be noted that local wastewater discharge limits are not necessarily established for all POCs, since the POTW will be capable of reducing the concentration of certain POCs to levels specified in the POTW's NPDES wastewater discharge and biosolids disposal permit. It is through negotiations between the POTW and the regulatory authority that the final list of pollutants to be included in the local limit development process is established [26].

During the pretreatment sampling process, at least one influent wastewater pollutant scan should be performed to determine all toxic and hazardous pollutants being discharged into the municipal sewer system. The initial influent pollutant scan should be conducted no earlier than 12 months prior to the submission of the local wastewater discharge limits to the regulatory authority for approval [26]. At a minimum, the pollutants that should be quantified in the initial POC scan include the following:

1. Chemicals listed in 40 CFR Part 122, Appendix D, Table II (toxic organic pollutants) and Table III (metals, cyanide, phenol)
2. Any chemicals listed in 40 CFR Part 122, Appendix D, Table V that are known to be a constituent of any industrial discharge or suspected of being present
3. Any other additional toxic pollutants designated in the state water quality standards and/or NPDES permit

In 40 CFR Part 122, toxic organic compounds are categorized as volatiles, acids, base/neutral compounds, or pesticides (Table 4.2). Scanning for additional toxic pollutants may be required if the POTW has any reason to believe that these materials are contained in any industrial waste discharge.

Table 4.3 lists additional toxic pollutants that must be quantified if POTW pretreatment personnel suspect their presence in any industrial discharge. In addition, if POTW pretreatment personnel suspect that certain conventional and/or nonconventional pollutants are present in an industrial discharge, they must also be quantified in the initial pollutant scan. A list of these pollutants is provided in Table 4.4. Finally, particular substances categorized as toxic or hazardous in 40

TABLE 4.2 Organic Toxic Pollutants (40 CFR Part 122, Table II)*

Volatiles

- Acrolein
- Acrylonitrile
- Benzene
- Bromoform
- Carbon tetrachloride
- Chlorobenzene
- Chlorodibromomethane
- Chloroethane
- 2-Chloroethylvinyl ether
- Chloroform
- Dichlorobromomethane
- 1,1-Dichloroethane
- 1,2-Dichloroethane
- 1,1-Dichloroethylene
- 1,2-*trans*-Dichloroethylene
- 1,2-Dichloropropane
- 1,3-Dichloropropylene
- Ethylbenzene
- Methyl bromide
- Methyl chloride
- Methylene chloride
- 1,1,2,2-Tetrachloroethane
- Tetrachloroethylene
- Toluene
- 1,1,1-Trichloroethane
- 1,1,2-Trichloroethane
- Trichloroethylene
- Vinyl chloride

Acid compounds

- *p*-Chloro-*m*-cresol
- 2-Chlorophenol
- 2,4-Dichlorophenol
- 2,4-Dimethylphenol
- 4,6-Dinitro-*o*-cresol
- 2,4-Dinitrophenol
- 2-Nitrophenol
- 4-Nitrophenol
- Pentachlorophenol
- Phenol
- 2,4,6-Trichlorophenol

Base/neutral compounds

- Acenaphthene
- Acenaphthylene
- Anthracene
- Benzidine
- Benzo(*a*)anthracene
- Benzo(*a*)pyrene
- 3,4-Benzofluoranthene
- Benzo(ghi)perylene
- Benzo(*k*)fluoranthene
- Bis(2-chloroethoxy)methane
- Bis(2-chloroethoxy)methane
- Bis(2-chloroisopropyl)ether
- Bis(2-ethylhexyl)phthalate
- 4-Bromophenyl phenyl ether
- Butylbenzyl phthalate
- 2-Chloronaphthalene
- 4-Chlorophenyl phenyl ether
- Chrysene
- Dibenzo(*a,h*)anthracene
- 1,2-Dichlorobenzene
- 1,3-Dichlorobenzene
- 1,4-Dichlorobenzene
- Diethyl phthalate
- 3,3′-Dichlorobenzidine
- Dimethyl phthalate
- Di-*n*-butyl phthalate
- 2,4-Dinitrotoluene
- 2,6-Dinitrotoluene
- Di-*n*-octyl phthalate
- 1,2-Diphenylhydrazine
- Fluorene
- Fluroranthene
- Hexachlorobutadiene
- Hexachlorocyclopentadiene
- Hexachlorobenzene
- Hexachloroethane
- Indeno(1,2,3-*cd*)pyrene
- Isophorone
- Napthalene
- Nitrobenzene
- *N*-Nitrosodimethylamine
- *N*-Nitrosodi-*n*-propylamine
- *N*-Nitrosodiphenylamine
- Phenanthrene
- Pyrene
- 1,2,4-Trichlorobenzene

*Adapted from refs. [16,26].

TABLE 4.2 Organic Toxic Pollutants (40 CFR Part 122, Table II) (*Continued*)

Pesticides			
Aldrin	4,4′-DDE	Endosulfan sulfate	PCB-1254
Alpha-BHC	4,4′-DDD	Endrin	PCB-1221
beta-BHC	Dieldrin	Endrin aldehyde	PCB-1232
gamma-BHC	alpha-Endosulfan	Heptachlor	PCB-1248
delta-BHC	beta-Endosulfan	Heptachlor epoxide	PCB-1260
Chlordane	Toxaphene	PCB-1242	PCB-1016
4,4′-DDT			

TABLE 4.3 Other Toxic Pollutants (Metals and Cyanide) and Total Phenols to Be Quantified if Suspected to Be Present in Influent Wastewater (40 CFR Part 122, Table III)*

Antimony (total)	Mercury (total)
Arsenic (total)	Nickel (total)
Beryllium (total)	Selenium (total)
Cadmium (total)	Silver (total)
Chromium (total)	Thallium (total)
Copper (total)	Zinc (total)
Lead (total)	Cyanide (total)
Phenols (total)	

*Adapted from refs. [16,26].

TABLE 4.4 Conventional and Nonconventional Pollutants Required to Be Quantified if Suspected to Be Present in POTW Influent Wastewater (40 CFR Part 122, Table IV)*

Bromide	Surfactants
Chlorine (total) residual	Aluminum (total)
Color	Barium (total)
Fecal coliform	Boron (total)
Fluoride	Cobalt (total)
Nitrate-nitrite	Iron (total)
Nitrogen organic (total)	Magnesium (total)
Oil and grease	Molybdenum (total)
Phosphorus (total)	Manganese (total)
Radioactivity	Tin (total)
Sulfate	Titanium (total)
Sulfide	Sulfite

*Adapted from refs. [16,26].

CFR Part 122 and that must be included as part of the initial pollutant scan are listed in Table 4.5.

All wastewater samples used in developing the list of POCs must be collected and analyzed in accordance with the methods specified by 40 CFR Part 136. Analysis of biosolids must be completed in accordance with 40 CFR Part 503. If a particular pollutant concentration is found to be below the detection limit of the analytical instrumentation, the POTW has the option of using 50 percent of the pollutant's detection limit in the averaging calculation [16]. Alternatively, the POTW may want to consider a more conservative approach by using the instrument's detection limit in the averaging estimate. No matter which approach is chosen by the POTW, a written justification must accompany the local wastewater discharge limit documentation submitted to the regulatory authority.

Once the initial influent scan is complete, the data on the chemical compounds must be evaluated to determine the POCs. POCs are identified as any chemical species falling into one or more of the categories specified in Table 4.6.

TABLE 4.5 Toxic Pollutants and Hazardous Substances Required to Be Identified if Presence in Industrial Discharge Is Suspected (40 CFR Part 122, Table V)*

Toxic pollutants
Asbestos

Hazardous substances
Acetaldehyde
Allyl alcohol
Allyl chloride
Amyl acetate
Aniline
Benzonitrile
Benzyl chloride
Butyl acetate
Butylamine
Captan
Carbaryl
Carbofuran
Carbon disulfide
Chlorpyrifos
Coumaphos
Cresol
Crotonaldehyde
Cyclohexane
2,4-Dichlorophenoxy acetic acid
Diazinon
Dicamba
Dichlobenil
Dichlone
2,2-Dichloro-propionic acid
Dichlorvos
Diethyl amine
Dimethyl amine
Dinitrobenzene
Diquat
Disulfoton
Diuron
Epichlorohydrin
Ethion
Ethylene diamine
Ethylene dibromide
Formaldehyde
Furfural
Guthion
Isoprene
Isopropanolamine DDBS†
Kelthane
Kepone
Malathion
Mercaptodimethur
Methoxychlor
Methyl mercaptan
Methyl methacrylate
Methyl parathion
Mevinphos
Mexacarbate
Monoethyl amine
Monomethyl amine
Naled
Napthenic acid
Nitrotoluene
Parathion
Phenolsulfanate
Phosgene
Propargite
Propylene oxide
Pyrethrins
Quinoline
Resorcinol
Strontium
Strychnine
Styrene
2,4,5-Trichlorophenoxy acetic acid
Tetrachlorodiphenylethane
2,4,5-Trichlorophenoxy propanoic acid
Trichlorofan
Triethanolamine DDBS†
Triethylamine
Trimethylamine
Uranium
Vanadium
Vinyl acetate
Xylene
Xylenol
Zirconium

*Adapted from refs. [16,26].
†DDBS, dodecylbenzenesulfonate.

4.1.2.4 Establishing a sampling program for POCs. After the regulatory authority has accepted a list of POCs, the POTW must then identify the major nondomestic sources of those POCs. The identified sources (e.g., industrial/commercial) may then be regulated under the POTW's pretreatment program. In addition to the major nondomestic sources of POCs, the POTW must estimate the pollutant contributions from both residential and unregulated commercial sources. POTWs may establish the pollutant loadings from residential and unregulated commercial sources through the use of a site-specific monitoring program. Such a program entails the periodic collection and analysis of samples from trunk lines receiving waste-

TABLE 4.6 Wastewater Pollutants Qualifying as Pollutants of Concern (POCs)*

Arsenic, cadmium, total chromium, chromium(VI), chromium(III), copper, lead, nickel, zinc, silver, molybdenum, selenium, and mercury, regardless of whether they were detected in the initial scan
Any pollutant listed in the state water quality standards and/or NPDES permit that is detected
Any pollutant with a concentration equal to or greater than 0.1 mg/liter
Any pollutant specified in 40 CFR Part 122, Appendix D, Tables II or V, that was analyzed and its concentration is equal to or greater than 0.05 mg/liter
Any pollutant that is present at 0.01 mg/liter or greater and has a bioconcentration factor of 300 or greater†
Any pollutant designated by the POTW pretreatment program and/or regulatory authority

*Adapted from refs. [16,26].
†Bioconcentration factor = (mg/kg) of pollutant in organism/(mg/liter) in wastewater solution.

water from residential/commercial sources. Site-specific total residential and unregulated commercial pollutant loadings are calculated from pollutant concentrations and wastewater flow monitoring data [23,26,30].

Commercial source monitoring data are useful to POTWs in identifying sources of toxic pollutants and in determining which commercial sources should be regulated for the purpose of estimating local limits. Tables 4.7, 4.8, and 4.9 provide representative data of pollutants found in the effluent of the following commercial sources: (1) hospitals, (2) automobile radiator shops, (3) car washes, (4) truck cleaners, (5) dry cleaners, and (6) commercial laundries. Included in these tables are residential monitoring data from 24 POTWs, septage monitoring data from 9 POTWs, and landfill leachate monitoring data from 8 POTWs. In the absence of site-specific monitoring data, these values may be used as typical wastewater concentrations for residential/commercial sources, septage, and landfill leachate when establishing local discharge limits.

Occasionally, a POTW may find that the pollutant loadings entering the wastewater collection system exceed the estimated maximum allowable headworks loading due to contributions of the following nonindustrial sources:

1. Nonpoint sources (e.g., runoff) discharging to combined sewers
2. Elevated pollutant levels in water supplies
3. Household disposal of chemicals into sanitary sewers
4. Toxic pollutant discharges from commercial sources

TABLE 4.7 Inorganic Pollutant Concentrations Discharged from Residential, Septage, Landfill Leachate, and Commercial Sources*

Pollutant†	Residential	Septage	Landfill leachate	Car washes	Dry cleaners	Hospitals	Industrial cleaners	Radiator shops	Truck cleaners
Aluminum			0.34					1.13	7.7
Antimony			0.142			0.018			0.09
Arsenic	0.007	0.141	0.042			0.026	0.34	0.12	0.68
Barium	0.115	5.758	0.201			1.779	0.506		
Beryllium									0.013
Boron	0.3								
Cadmium	0.008	0.097	0.03	0.017	0.008	0.018	0.034	0.165	0.027
Chromium	0.034	0.490	0.633	0.074	0.022	0.117	0.216	0.128	0.120
Chromium(III)	0.006								
Cobalt		0.406			0.004				
Copper	0.109	4.835	0.395	0.139	0.086	0.452	0.552	22.218	0.233
Cyanide	0.082	0.469	0.029				0.101	0.030	55.587
Fluoride	0.255					0.637			
Iron	0.989	39.287	33.8		0.51	2.249	3.796	64.430	
Lead	0.116	1.210	0.156	0.162	0.032	0.881	1.514	69.210	0.353
Lithium	0.031								
Manganese	0.087	6.088	13.224				0.553	1.23	
Mercury	0.002	0.005	0.001			0.002	0.004	0.0004	
Nickel	0.047	0.526	0.550	0.080	0.009	0.060	0.140	0.300	0.177
Selenium	0.004		0.010			0.011	0.016		0.012
Silver	0.019	0.099	0.019	0.018		0.098	0.123	0.024	0.114
Thallium									0.042
Tin		0.076							
Zinc	0.212	9.971	12.006	0.543	0.174	0.563	1.873	145.295	4.416

*Adapted from refs. [16,26].
†Concentrations are in milligrams per liter.

TABLE 4.8 Organic Pollutant Concentrations Discharged from Residential, Septage, Landfill Leachate, and Commercial Sources*

Pollutant†	Residential	Septage	Landfill leachate	Car washes	Dry cleaners	Hospitals	Industrial cleaners	Radiator shops	Truck cleaners
1-Ethyl-4-methyl benzene							150		
Fluoranthene	0.001								
Formaldehyde						0.58			
2-Hexanone			0.094						36,478
Isopropyl alcohol	14.06								
Methyl alcohol		15.84							
Methyl ethyl ketone		3.65							
Methylene chloride	0.027	0.101	0.310				0.006		
4-Methylephenol			0.065						
4-Methyl-2-pentanone			0.43						
m-Xylene							6.744		
Naphthalene			0.113				0.310		
n-Butyl benzene-sulfonamide					1.2				
n-Nitrosodiphenylamide			0.011						
Pentachlorophenol			0.016						
Phenols	0.010		0.71		0.117	0.201	0.244		
2-Propanol							25.5		
1-Propanol							74		
Pyrene	0.0002								
p-Chloro-*m*-Cresol			0.018						
p-Xylene							3.543		1.881
1,1,2,2-Tetrachloroethane							0.099		
Tetrachloroethene	0.015						0.163		
Tetrachloroethylene	0.00001								

*Adapted from refs. [16,26].

†Concentrations are in milligrams per liter.

TABLE 4.8 Organic Pollutant Concentrations Discharged from Residential, Septage, Landfill Leachate, and Commercial Sources (*Continued*)

Pollutant†	Residential	Septage	Landfill leachate	Car washes	Dry cleaners	Hospitals	Industrial cleaners	Radiator shops	Truck cleaners
Toluene		0.17	0.735		0.016		4.032		
Total BHC	0.001								
Total endosulfan	0.002								
trans-1,2-Dichloroethene	0.013						0.026		
1,2,4-Trichlorobenzene	0.013								
1,1,1-Trichloroethane			0.019				0.025		
Trichloroethene			0.28						
Vinyl acetate			0.25						
Vinyl chloride			0.067						
Xylene		0.051	0.317						

TABLE 4.9 Conventional Pollutant Concentrations Discharged from Residential, Septage, Landfill Leachate, and Commercial Sources*

Pollutant†	Residential	Septage	Landfill leachate	Car washes	Dry cleaners	Hospitals	Industrial cleaners	Radiator shops	Truck cleaners
Ammonia	43.11								
COD		21,248	34.545	126.333	315.57	346.72	1431/4	7.667	
Phosphate	28.8				25.72	4.465	13.2		7.85
Sulfide							4.8		
Surfactants					0.02	1.791			
TDS					625	426.6			3364
Total phosphorus	0.7								

*Adapted from refs. [16,26].

†Concentrations are in milligrams per liter.

Regulating the pollutant contributions of nonpoint sources may be addressed through combined sewer overflow (CSO) abatement programs as well as urban/agricultural chemical management practice programs [20,23,26]. The POTW can effectively reduce elevated pollutant levels in municipal water supplies by interacting with the city water department. Proper disposal of household chemicals can be encouraged by the POTW by instituting public education programs and establishing chemical and used-oil recovery stations, while elevated levels of pollutants discharged from specific commercial sources are most effectively reduced through implementation of local limits. Finally, pollutant loadings discharged from other wastewater sources to the POTW headworks (e.g., septage and landfill leachate discharges) should be carefully emulated when establishing local limits.

Once the various sources and mass flows of POCs to the POTW are determined, the maximum allowable headworks loading for each specific pollutant is estimated. The maximum allowable headworks loading is the mass flow of a specific POC that the POTW can treat will maintaining regulatory compliance with both its effluent wastewater discharge and biosolids disposal permits. The maximum allowable headworks loading is estimated by considering both the regulatory limits placed on the POTW effluent wastewater and biosolids quality and the POTW's pollutant removal efficiency (R_{POTW}).

4.1.2.5 POTW pollutant removal efficiency. Estimation of the pollutant removal efficiency (R_{POTW}) for local limits development is necessary to estimate the fraction of a POTW pollutant loading that is ultimately discharged to the receiving water and the fraction that is transferred to biosolids. To ensure that the pollutant removal efficiency is based on representative data, wastewater influent, wastewater effluent, biosolids, and suspicious domestic/commercial wastewater discharges must be sampled for a minimum of 6 months prior to estimating the POTW's pollutant removal efficiency [34]. Moreover, proper correlation of the changes in wastewater characteristics with final biosolids quality requires that the lag time for wastewater and biosolids processing be considered. In other words, the sampling schedule should take into account both the hydraulic and solids retention times in the various unit operations in order to obtain an accurate estimate of the POTW pollutant removal efficiency.

It should be noted that validation of a POTW pollutant removal efficiency for conservative pollutants (e.g., metals) is readily achieved through a mass-balance approach. However, because of volatility and biodegradability, an alternative approach must be employed for establishing the maximum allowable headworks loading for nonconservative pollutants (e.g., organic compounds) [26,34].

Both the average daily removal efficiency and the mean removal efficiency are used to describe the average POTW pollutant removal efficiency. A daily removal efficiency is defined as the percentage change of a pollutant's mass flow for samples taken before and after treatment, as described by Eq. (4.1):

$$\text{Removal efficiency} \quad \frac{\text{influent mass (lb/day)} - \text{effluent mass (lb/day)}}{\text{influent mass (lb/day)}} \quad 100 \qquad (4.1)$$

For example, if the mass flow for chromium in an influent wastewater sample on a specific day is estimated to be 100 lb/day and the mass flow of chromium in the effluent wastewater sample on the same day is 7 lb/day, the daily removal efficiency for chromium would be 93 percent. It is important to recognize that a 93 percent removal efficiency means that for a conservative pollutant, 93 percent of the influent pollutant mass ultimately was transferred to the biosolids, whereas 7 percent of the pollutant mass remained in the wastewater effluent.

An average daily or mean pollutant removal efficiency may be estimated from daily removal efficiency data using one of two approaches. An average daily pollutant removal efficiency is calculated by taking the arithmetic average of individual daily removal values, whereas the mean pollutant removal efficiency computes the average of the influent and effluent samples and then estimates the average pollutant removal efficiency based on the percentage change. Example 4.1 illustrates both approaches for establishing the average daily or mean pollutant removal efficiency.

Example 4.1 To establish the average removal efficiency for copper, a POTW has taken three influent and effluent samples and computed the mass flows. The results of these samples are as follows:

Sample day	Influent mass (lb/day)	Effluent mass (lb/day)
1	20	5
2	10	3
3	40	8

Given the mass flow rates, estimate the average daily removal efficiency using the two averaging approaches.

solution

Step 1. Estimate each daily removal efficiency using Eq. (4.1):

Day 1:

$$\text{Daily removal efficiency} = \frac{20 - 5}{20} \cdot 100$$

$$= 75 \text{ percent}$$

Each of the remaining daily removal efficiencies may be solved similarly. The arithmetic average of the three daily removal efficiencies can be used to estimate an average daily removal efficiency.

Sample day	Influent mass (lb/day)	Effluent mass (lb/day)	Daily removal efficiency (%)
1	20	5	75
2	10	3	70
3	40	8	80
Average			75

Step 2. The mean removal efficiency can be estimated by calculating the average percentage changes in influent and effluent mass flows:

Sample day	Influent mass (lb/day)	Effluent mass (lb/day)
1	20	5
2	10	3
3	40	8
Average	23.3	5.3

$$\text{Mean removal efficiency} = \frac{23.3 - 5.3}{23.3} \cdot 100$$

$$= 77 \text{ percent}$$

As illustrated in Example 4.1, application of the two arithmetic procedures sometimes results in different estimates of the average POTW pollutant removal efficiency. To determine whether the average daily or mean removal efficiency should be used in establishing the maximum allowable pollutant headworks loading, the POTW would want to determine the frequency at which the daily removal efficiency would be above or below a specified value. This approach is necessary because, in actual POTW operation, the daily POTW pollutant removal efficiency can vary significantly [34]. If, during a period of time, the average removal efficiency is high, biosolids quality may deteriorate. Conversely, during periods when the average pollutant removal efficiency is low, receiving water quality may be adversely affected. To ensure that the estimated maximum allowable pollutant headworks loading will be protective of both the effluent wastewater discharge and biosolids quality under most circumstances, the EPA has recommended employing the decile approach to account for the inherent variability of the POTW pollutant removal efficiency [26,34].

The first step in the decile approach involves ordering a set of daily pollutant removal efficiency data from the lowest daily removal efficiency to the highest. The ordered set of data is then divided into nine deciles that reflect the frequency of occurrence of the pollutant removal efficiencies. For example, the first decile corresponds to an individual daily pollutant removal efficiency value below which 10 percent of the pollutant removal efficiencies fall. Similarly, the second decile corresponds to a daily removal value below which 20 percent of the daily removal fall, etc. To identify the position of the individual decile values, Eq. (4.2) should be used. Example 4.2 illustrates the use of Eq. (4.2) in identifying the nine deciles.

$$\begin{aligned}\text{First decile} &= \frac{N+1}{10}\\ \text{Second decile} &= 2\left(\frac{N+1}{10}\right)\\ \text{Third decile} &= 3\left(\frac{N+1}{10}\right)\end{aligned}\tag{4.2}$$

where N = number of data sets (i.e., influent and effluent mass flows).

NOTE: In identifying the decile position from the ordered list of pollutant removal efficiencies, the whole number is used. For example, if the third decile is estimated to be 7.7 using Eq. (4.2), the removal efficiency at the seventh position corresponds to the third decile. In other words, 30 percent of the removal efficiencies are less than the removal efficiency located at the seventh position.

Example 4.2 The Langston County Water Reclamation Facility has generated the zinc daily removal efficiencies from 25 influent and effluent wastewater samples shown at the top of the next page. Estimate the values of the removal efficiencies corresponding to the nine decile positions.

Day	POTW removal efficiency	Day	POTW removal efficiency
1	70.1	14	81.3
2	39.9	15	71.4
3	82.1	16	74.2
4	48.7	17	75.3
5	77.5	18	78.1
6	55.3	19	64.6
7	61.4	20	70.5
8	76.8	21	65.7
9	60.6	22	83.9
10	76.1	23	67.8
11	69.3	24	85.6
12	71.0	25	69.9
13	71.3		

solution

Step 1. Order the pollutant removal efficiencies from the lowest removal efficiency value to the highest:

Data sequence	Efficiency	Data sequence	Efficiency
1	39.9	14	71.3
2	48.7	15	71.4
3	55.3	16	74.2
4	60.6	17	75.3
5	61.4	18	76.1
6	64.6	19	76.8
7	65.7	20	77.5
8	67.8	21	78.1
9	69.3	22	81.3
10	69.9	23	82.1
11	70.1	24	83.9
12	70.5	25	85.6
13	71.0		

Step 2. Estimate the position of each of the nine deciles using Eq. (4.2):

First decile:
$$\frac{N+1}{10} = \frac{25+1}{10} = 2.6$$

A decile estimate of 2.6 corresponds to a data position of 2. From the ordered table, this corresponds to a removal efficiency of 48.7. Using the same approach, each of the nine deciles is summarized in the following table.

Decile	Decile calculation	Pollutant removal efficiency position	Pollutant removal efficiency
1	2.6	2	48.7
2	5.2	5	61.4
3	7.8	7	65.7
4	10.4	10	69.9
5	13.0	13	71.0
6	15.6	15	71.4
7	18.2	18	76.1
8	20.8	20	77.5
9	23.4	23	82.1

Since the first decile occurs at a removal efficiency of 48.7 percent, this means that the POTW should anticipate that the pollutant removal efficiency should be at this level or lower approximately 10 percent of the time. Similarly, the ninth decile occurs at a removal efficiency of 82.1 percent. This means that the POTW should expect that the pollutant removal efficiency will be at this level or lower 90 percent of the time.

4.1.2.6 Maximum allowable headworks loading. Once an acceptable POTW pollutant removal efficiency (R_{POTW}) is established by the POTW, the maximum allowable headworks loading for specific POCs may be estimated. In general, the maximum allowable headworks loading based on POTW effluent quality is estimated using Eq. (4.3), whereas the maximum allowable headworks loading based on biosolids quality is estimated using Eq. (4.4).

Headworks loading based on POTW effluent quality:

$$\text{Headworks loading (lb/day)} = \frac{Q_{\mathrm{POTW}} C_{\mathrm{crit}} 8.34}{1 - R_{\mathrm{POTW}}} \tag{4.3}$$

NOTE: $Q_{\mathrm{POTW}} C_{\mathrm{crit}} 8.34$ = maximum mass flow (lb/day) of pollutant in effluent wastewater discharge.

Headworks loading based on biosolids quality:

$$\text{Headworks loading lb/day} = \frac{Q_{\mathrm{bio}} C_{\mathrm{biocrit}} PS\ 8.34}{R_{\mathrm{POTW}}} \tag{4.4}$$

where Q_{POTW} = POTW influent wastewater flow, million gal/day (MG/day)
C_{crit} = POTW wastewater discharge limit, mg/liter (e.g., NPDES permit level)
R_{POTW} = pollutant removal efficiency, decimal
Q_{bio} = POTW biosolids flow, MG/day

C_{biocrit} = biosolids pollutant regulatory level, mg/kg
PS = percent solids of disposed biosolids, decimal
8.34 = conversion factor, lb/ [MG · (mg/liter)]

NOTE: $Q_{\text{bio}}C_{\text{biocrit}}PS$ 8.34 = maximum mass flow (lb/day) of pollutant in biosolids.

Equations (4.3) and (4.4) illustrate that establishing a maximum allowable pollutant headworks loading that is protective of effluent wastewater and biosolids quality requires the proper choice of the POTW's pollutant removal efficiency (R_{POTW}). In many cases, the POTW simply will substitute the average POTW pollutant removal efficiency in Eq. (4.3) or (4.4) and divide the resulting maximum allowable headworks loading by a safety factor (normally 2). The purpose of the safety factor is to build a pollutant discharge reserve capacity for protection of both biosolids and wastewater quality and to allow for the future expansion of the quantity of sewer discharges. The basic approach for establishing the POTW headworks loading for a conservative pollutant is illustrated in Example 4.3.

Example 4.3 The Louisville County Sewage Treatment Works has completed its initial wastewater pollutant scan, and it indicates that copper must be categorized as a POC. If the average sewage flow rate to the POTW is 8 million gal/day (Q_{POTW}) and the average POTW removal efficiency for copper is 98 percent (R_{POTW}), estimate the maximum allowable headworks loading of copper in pounds per day. Assume that the only applicable regulatory requirement for copper is the NPDES effluent wastewater discharge limit of 0.005 mg/liter and that the pretreatment personnel want to incorporate a safety factor of 2 into all the headworks loading estimates.

solution

Step 1. Estimate the maximum allowable headworks loading based on maintaining compliance with the NPDES permit [Eq. (4.3)]:

$$\text{Headworks loading (lb/day)} = \frac{Q_{\text{POTW}}C_{\text{crit}}8.34}{1 - R_{\text{POTW}}}$$

$$= \frac{8 \cdot 0.005 \cdot 8.34}{1 - 0.98} = \frac{16.68\text{ lb}}{\text{day}}$$

Step 2. Estimate the maximum allowable headworks loading of copper assuming a safety factor of 2:

$$\text{lb/day} = \frac{\text{theoretical maximum headworks loading (lb/day)}}{\text{safety factor}}$$

$$= \frac{16.68\text{ lb/day}}{2} = 8.34\text{ lb copper/day}$$

Although use of an average pollutant removal efficiency and safety factor(s) in establishing the maximum allowable headworks loading is common, this method does not account for the inherent variability in the POTW pollutant removal efficiency. Therefore, in some circumstances, this approach may result in establishing overly conservative estimates of the allowable headworks loadings, thereby preventing the POTW from taking full advantage of its inherent pollutant treatment capacity. Alternatively, this approach could cause an underestimation of the POTW pollutant removal capacity, resulting in a regulatory violation of wastewater effluent and/or biosolids quality. To account for the inherent variability in the POTW pollutant removal efficiency, the decile approach should be employed when establishing the maximum allowable pollutant headworks loading. The decile approach permits the POTW to base its choice of a specific pollutant removal efficiency on the frequency of its observed occurrence. Example 4.4 illustrates the use of the decile approach in establishing a maximum allowable headworks loadings protective of both POTW effluent and biosolids quality.

Example 4.4 The Langston County Water Reclamation Facility wants to establish a maximum pollutant headworks loading for zinc that is protective of both the effluent wastewater discharge and biosolids quality (see Example 4.2). Assume that based on the POTW effluent flow rate (Q_{POTW}) and the NPDES limit for zinc (C_{crit}), the maximum discharge limit for zinc is 9 lb/day, whereas, based on the POTW biosolids flow rate to disposal (Q_{bio}), the pollutant concentration limit for zinc (C_{biocrit}), and the percent solids (PS), the maximum discharge limit for zinc in biosolids is 22 lb/day. The following nine decile levels were computed based on the daily zinc removal efficiencies given in Example 4.2.

Decile	Zinc removal efficiency
1	48.7
2	61.4
3	65.7
4	69.9
5	71.0
6	71.4
7	76.1
8	77.5
9	82.1

solution

Step 1. Calculate the allowable headworks loading based on POTW effluent quality at each of the nine decile levels using Eq. (4.3):

First decile:

$$\text{Headworks loading (lb/day)} = \frac{Q_{\text{POTW}} C_{\text{crit}} 8.34}{1 - R_{\text{POTW}}}$$

$$= \frac{9 \text{ lb/day}}{1 - 0.487} = 17.5 \text{ lb/day}$$

Headworks loadings based on the removal efficiencies of the other eight deciles are given as follows:

Decile	Pollutant removal efficiency	Allowable headworks loading based on POTW effluent wastewater quality (lb/day)
1	48.7	17.5
2	61.4	23.3
3	65.7	26.2
4	69.9	29.9
5	71.0	31.0
6	71.4	31.5
7	76.1	37.7
8	77.5	40.0
9	82.1	50.3

NOTE: As the removal efficiency increases, the allowable headwork loading increases.

Step 2. Calculate the allowable headworks loading based on biosolids quality at each of the nine decile levels using Eq. (4.4):

First decile:

$$\text{Headworks loading (lb/day)} = \frac{Q_{\text{bio}} C_{\text{biocrit}} PS\ 8.34}{R_{\text{POTW}}}$$

$$= \frac{22 \text{ lb/day}}{0.487} = 45.2 \text{ lb/day}$$

Headworks loadings based on the removal efficiencies of the other eight deciles are given as follows:

Decile	Pollutant removal efficiency	Allowable headworks loading based on POTW biosolids quality (lb/day)
1	48.7	45.2
2	61.4	35.8
3	65.7	33.5
4	69.9	31.5
5	71.0	31.0
6	71.4	30.8
7	76.1	28.9
8	77.5	28.4
9	82.1	26.8

NOTE: As the POTW pollutant removal efficiency increases, the allowable headworks loading based on biosolids quality decreases.

Step 3. Estimate the average daily removal rate from the reported daily removal efficiency data given in Example 4.2:

$$\text{Average daily removal efficiency} = \frac{70.1 + 39.9 + \cdots + 69.9}{25}$$

$$= 69.9 \text{ percent}$$

From the allowable headworks loading based on effluent wastewater quality, use of an average pollutant removal rate of 69.9 percent results in a maximum headworks loading of 29.9 lb/day. However, this removal efficiency corresponds to the fourth decile, which means that for 40 percent of the time, on average, the POTW will be *unable* to comply with its effluent quality limitations even when all the POTW's industrial users are in compliance with their local discharge limits. If this frequency of noncompliance is unacceptable to the POTW, a lower decile should be chosen. For example, if the first decile is chosen, the maximum headworks loading is only 17.5 lb/day, which is considerably more stringent than 29.9 lb/day. However, under these conditions, the POTW is certain that even with the variability of the POTW pollutant removal efficiency, it will remain in compliance with its wastewater discharge permit at least 90 percent of the time when all industrial users are in compliance.

Similarly, for the biosolids mass pollutant loading, if the headworks loading is based on a pollutant removal efficiency of 69.9 percent, the maximum headworks loading is 31.5 lb/day. Under these conditions, 60 percent of the time the biosolids concentration will exceed the biosolids concentration limits even when all the industrial users are in compliance. If this frequency of noncompliance is unacceptable to the POTW, a higher decile should be chosen. For example, if the ninth decile is chosen, the maximum headworks loading is 26.8 lb/day. Under these conditions, the POTW is certain that its biosolids quality will remain in compliance at least 90 percent of the time when all industrial users are in compliance.

Step 4. The final step in the decile approach is to choose a maximum allowable headworks loading that will be protective of wastewater discharge/biosolids quality and be achievable most of the time. This is accomplished by identifying the appropriate deciles that result in an acceptable frequency of protection of both effluent wastewater discharge and biosolids quality. The headworks loading that is the most stringent is then chosen as the maximum allowable for the POTW. For example, in the present problem, if the first and ninth deciles provide an adequate frequency of protection for effluent and biosolids quality, respectively, then the maximum allowable headworks loadings based on these two criteria can be compared:

Loading basis	Decile	Maximum allowable headworks loading (lb/day)
Effluent wastewater quality	1	17.5
Biosolids quality	9	26.8

From the preceding table, it is clear that the POTW should base its zinc local limits on the effluent wastewater discharge quality for the first decile that is the more stringent criterion. This corresponds to an maximum allowable zinc headworks loading of 17.5 lb/day (and a POTW removal efficiency of 48.7 percent). This headworks loading will be protective of both biosolids and effluent wastewater quality at least 90 percent of the time when all industrial discharges are in compliance.

4.1.2.7 Nonconservative pollutants. In developing local limits, a distinction must be drawn between conservative pollutants (i.e., those which are not degraded or volatilized within a POTW) and nonconservative pollutants (i.e., those which are, to some degree, biologically/chemically transformed and/or volatilized by the POTW's unit operations). Conservative pollutants are discharged from the POTW solely through the POTW's wastewater effluent and biosolids streams, whereas nonconservative pollutants are destroyed by chemical/biochemical reactions and/or partition into the air or are adsorbed onto a solid surface. Establishing the maximum allowable pollutant headworks loadings for nonconservative pollutants requires a different approach than that employed for conservative pollutants.

The EPA recommends using Eq. (4.5) for estimating the maximum allowable headworks loading for nonconservative pollutants based on biosolids regulatory standards [26,34]:

$$L_{\mathrm{in}} = \frac{C_{\mathrm{biocrit}}}{C_{\mathrm{bio}}/L_{\mathrm{inf}}} \tag{4.5}$$

where L_{in} = maximum allowable headworks loading, lb/day
L_{inf} = POTW influent loading, lb/day
C_{biocrit} = biosolids regulatory limit, mg/kg (e.g., local limit or from 40 CFR Part 503)
C_{bio} = biosolids pollutant level, mg/kg

The factor $C_{\mathrm{bio}}/L_{\mathrm{inf}}$ is a partitioning factor that relates the pollutant concentration found in biosolids to the headworks loading of the pollutant. To determine the partitioning factor for a particular pollutant, the POTW's influent and biosolids quality must be monitored routinely for that pollutant [34]. The partitioning factor enables calculation of an allowable headworks loading from a biosolids regulatory standard

(C_{biocrit}). Example 4.5 illustrates the approach for establishing the maximum allowable headworks loading for a nonconservative pollutant using Eq. (4.5).

Example 4.5 Baldwin County desires to establish the maximum allowable headworks loading for the organic compound naphthalene based on biosolids quality. From sampling both the influent wastewater and biosolids quality for naphthalene, it has been determined that at an average daily naphthalene loading of 96 lb/day to the POTW headworks, the average concentration of naphthalene in the biosolids is 65 mg/kg. The Baldwin County Health Department has established a maximum concentration of naphthalene in land-applied biosolids of 5 mg/kg. Given the local regulatory threshold, estimate the maximum allowable headworks loading for naphthalene.

solution

Step 1. Estimate the partitioning factor $C_{\text{bio}}/L_{\text{inf}}$ from the sampling data:

$$\text{Partitioning factor}\left(\frac{C_{\text{bio}}}{L_{\text{inf}}}\right) = \frac{65\text{ mg/kg}}{96\text{ lb/day}} = \frac{0.68\text{ mg}\cdot\text{day}}{\text{kg}\cdot\text{lb}}$$

Step 2. Estimate the maximum allowable headworks loading for naphthalene using Eq. (4.5):

$$L_{\text{in}} = \frac{C_{\text{biocrit}}}{C_{\text{bio}}/L_{\text{inf}}} = \frac{5\text{ mg/kg}}{(0.68\text{ mg}\cdot\text{day}/(\text{kg}\cdot\text{lb})} = 7.4\text{ lb/day}$$

It is important to recognize that the partitioning factor $C_{\text{bio}}/L_{\text{inf}}$ is highly variable and depends on such factors as wastewater temperature, ambient air temperature, humidity, biodegradation rates, aeration rates, and POTW influent flow [34]. To account for its inherent variability, the decile approach should be employed in identifying an appropriate partitioning factor to use in estimating a maximum allowable headworks loading for a particular nonconservative pollutant.

4.1.2.8 POTW headworks loading based on biosolids beneficial use. The structure and effectiveness of a POTW's pretreatment program and internal unit operations will have an impact on both the quantity and quality of the resulting biosolids. Having knowledge of both the plant unit operations and the biosolids beneficial-use criteria, POTW pretreatment personnel can develop local wastewater discharge limits that result in generating biosolids that have the desired quality. To estimate the maximum allowable headworks loading based on biosolids beneficial-use criteria, Eq. (4.4) may be used directly for all beneficial-use scenarios except when the biosolids application rate is based on cumulative application criteria. When the cumulative application criteria scenario is used, the cumulative pollutant regulatory limits must be transformed to a biosolids pollutant concentration limit (e.g., mg/kg) through the use of Eq. (4.6). The resulting biosolids pollutant concentration limit (C_{biocrit}) may then be employed in Eq. (4.4). The application of Eq. (4.6) in estimating the limiting pollutant concentration in biosolids is illustrated in Example 4.6.

$$C_{\text{biocrit}} = \frac{CAR \cdot SA}{SL \cdot Q_{\text{bio}} \cdot PS \cdot 3046} \tag{4.6}$$

where C_{biocrit} = contaminant concentration limit in biosolids based on cumulative land-application rate, mg/kg dry biosolids
CAR = cumulative application rate, lb/acre over site life
SA = site area, acres
SL = site life, years
Q_{bio} = biosolids flow to disposal, MG/day
PS = percent solids, decimal
3046 = constant, 365 days/year times 8.34 lb/gal

It should be noted that because of state or local regulatory requirements, there could be more than one biosolids pollutant concentration that applies to a respective pollutant. If this is the case, the most stringent criterion must be employed in estimating the maximum allowable headworks loading.

Example 4.6 Based on wastewater and biosolids chemical analyses, the Sandersville County Wastewater Treatment Plant is considering land application of biosolids generated from its wastewater treatment system. From previous studies, it was determined that the treatment plant generates 18,000 gal/day of biosolids with an 88 percent moisture content. Moreover, based on the nitrogen content of the biosolids, a 150-acre site was required for annual beneficial use of the biosolids as a low-grade fertilizer. If cadmium is determined to be the limiting pollutant of concern, estimate (1) the maximum concentration of cadmium permissible in biosolids if the site is to be operated for 30 years and (2) the theoretical maximum allowable headworks loading. Assume that the POTW has an acceptable average cadmium removal efficiency of 92 percent.

solution

Step 1. Determine the cumulative application of cadmium in units of lb/acre. From Table 1.2, the cumulative application rate for cadmium is 39 kg/ha (or 34.8 lb/acre).

Step 2. Estimate the solids content of the biosolids. If the moisture content is 88 percent, the percent solids (PS) is 12 percent (0.12).

Step 3. Use Eq. (4.6) to estimate the maximum permissible concentration of cadmium in biosolids:

$$C_{\text{biocrit}} = \frac{CAR \cdot SA}{SL \cdot Q_{\text{bio}} \cdot PS \cdot 3046}$$

$$= \frac{34.8 \text{ lb/acre} \cdot 150 \text{ acres}}{30 \text{ years} \cdot 0.018 \text{ MG} \cdot 0.12 \cdot (3046 \text{ days} \cdot \text{lb}) / (\text{year} \cdot \text{gal})}$$

$$= 26.4 \text{ mg/kg}$$

Maximum allowable headworks loading (lb/day)

$$= \frac{Q_{\text{bio}} \cdot C_{\text{biocrit}} \cdot PS \cdot 8.34}{R_{\text{POTW}}}$$

$$= \frac{0.018 \text{ MG/day} \cdot 26.4 \text{ mg/kg} \cdot 0.12 \cdot 8.34}{0.92}$$

$$= 0.52 \text{ lb/day}$$

In the absence of more stringent biosolids criteria, the EPA encourages POTWs to use either of two sets of biosolids criteria found in 40 CFR Part 503 for the development of local wastewater discharge limits [40,52]. The first set of biosolids criteria reflects instantaneous maximum pollutant concentration limits (i.e, ceiling concentration limits), whereas the second set contains monthly average limits (i.e., pollutant concentration limits). These biosolids criteria are summarized in Tables 4.10 and 4.11. Because of lower pollutant concentration limits, use of monthly maximum pollutant concentrations to establish local limits would yield a more conservative headworks loading.

In the final step of the local limits development process, a percentage of the maximum allowable headworks loading of a particular pollutant is allocated to individual industrial and regulated commercial sources. For each regulated industrial/commercial discharger, the allocated pollutant percentage of the maximum allowable headworks load-

TABLE 4.10 Instantaneous Maximum Pollutant Concentrations in Land-Applied Biosolids (40 CFR Part 503, Table 1)*

Pollutant	Ceiling concentration (mg/kg)
Arsenic	75
Cadmium	85
Copper	4300
Lead	8400
Mercury	57
Molybdenum	75
Nickel	420
Selenium	100
Zinc	7500

*Adapted from ref. [40].

TABLE 4.11 Monthly Maximum Pollutant Concentrations in Land-Applied Biosolids (40 CFR Part 503, Table 3)*

Pollutant	Monthly average concentration (mg/kg)
Arsenic	41
Cadmium	39
Copper	1500
Lead	300
Mercury	17
Nickel	420
Selenium	100
Zinc	2800

*Adapted from ref. [40,51].

ing is translated into a mass- or concentration-based local limit that is specified in the pretreatment permit issued to the regulated nondomestic pollutant source by the POTW.

4.1.2.9 Allocation of pollutant loadings. The process used to allocate the pollutant loadings among the various regulated nondomestic sources is the sole responsibility of the local POTW. To allow for future expansion and development within the service area, the POTW normally allocates only a fraction of the maximum allowable pollutant headworks loading [26,34]. Regardless of the procedure employed by the POTW, the federally mandated categorical pretreatment standards must not be exceeded at any time (40 CFR Parts 405 to 471).

In the allocation of pollutants to nondomestic sewer users, the POTW has the option of establishing local wastewater discharge limits based on either the pollutant concentration or the pollutant mass flow rate (e.g., pounds per day). Although there are no legal requirements that prescribe how the POTW should allocate pollutant loadings, the regulatory authority must ensure that the limits are based on technically justifiable estimates and that they must protect against pollutant pass through and/or plant interference. If a concentration-based local limit is to be used, Eq. (4.7) may be employed to transform the mass discharge limit allocated to the facility by the POTW into a concentration-based limit. Example 4.7 illustrates the approach for translating the allocated portion of the maximum allowable pollutant headworks loading into a concentration-based local limit for several industrial sewer users.

$$\text{Local discharge limit (mg/liter)} = \frac{M_{\text{amf}}}{Q_{\text{facility}} \cdot 8.34} \tag{4.7}$$

where M_{amf} = daily mass flow rate of pollutant allocated to the discharging facility, lb/day
Q_{facility} = discharge flow rate of facility, MG/day
8.34 = conversion factor, lb/[MG · (mg/liter)]

Following the decision to use concentration- or mass-based limits, the POTW must submit the following information to the EPA or state regulatory agency:

- A description of the allocation method employed
- A listing of each industrial user and the mass of each POC that will be allocated to each user
- A description of the tracking/methodology to be used to demonstrate that the maximum allowable industrial loading is not exceeded

A public notice is used to disseminate the local wastewater discharge limit decisions to the community. Both the local wastewater discharge limits and the sewer ordinance documents (i.e., pretreatment permit) will become part of the permitted pretreatment program on approval by the EPA or the state as appropriate.

Example 4.7 The Poole County Wastewater Treatment Facility has established a maximum allowable POTW headworks loading of 0.9 lb/day for cadmium. In reviewing the operations of its nondomestic sewer users, three industrial facilities have been identified as being the predominant nondomestic sources of cadmium. They include (1) Meadow Lake Electroplating Company, (2) Cache Iron and Steel Works, Inc., and (3) U.S. Army Vehicle Maintenance Depot. The industrial discharge flow rate for each facility is given as follows:

Facility	Average daily flow rate, gal/day
Meadow Lake Electroplating Co.	97,000
Cache Iron and Steel Works, Inc.	400,000
U.S. Army Vehicle Maintenance Depot	210,000

To protect biosolids quality and to allow for future expansion of industrial sewer users, the pretreatment personnel have determined that they would like to allocate only 50 percent of the maximum allowable cadmium loading to the three industrial users (i.e., 0.45 lb/day). If each industrial user is allowed to discharge one-third of the daily allowable POTW cadmium headworks loading (i.e., 0.15 lb/day), estimate the permissible local wastewater discharge cadmium concentration for each of the three facilities.

solution Using the respective average daily flow rates, calculate the concentration-based local wastewater discharge limit using Eq. (4.7):

Meadow Lake Electroplating Facility:

$$\text{mg/liter} = \frac{0.15\ \text{lb/day}}{(8.34\ \text{MG} \cdot \text{liter/day} \cdot \text{mg})\ (97{,}000\ \text{gal}/10^6\ \text{gal} \cdot \text{day})} = 0.185\ \text{mg/liter}$$

Cache Iron and Steel Works, Inc.:

$$\text{mg/liter} = \frac{0.15\ \text{lb/day}}{(8.34\ \text{MG} \cdot \text{liter/day} \cdot \text{mg})\ (400{,}000\ \text{gal}/10^6\ \text{gal} \cdot \text{day})} = 0.045\ \text{mg/liter}$$

U.S. Army Vehicle Maintenance Depot:

$$\text{mg/liter} = \frac{0.15\ \text{lb/day}}{(8.34\ \text{MG} \cdot \text{liter/day} \cdot \text{mg})\ (210{,}000\ \text{gal}/10^6\ \text{gal} \cdot \text{day})} = 0.086\ \text{mg/liter}$$

NOTE: These local limits would ensure that the total contribution of cadmium from nondomestic users would not exceed 0.45 lb/day. Future expansion of these facilities and/or additions of new facilities to the sewer system would require recalculation of the local limits.

If an industry plans to expand capacity and therefore the quantity of pollutants generated, the POTW reserves the option of allowing increased industrial discharges if sufficient treatment capacity exists or may require the imposition of additional levels of pretreatment. If the industry is allowed to increase its pollutant discharge, the POTW should negotiate an increase in the surcharge rate for the affected industry to offset the increased costs of treatment [36]. An example of an approach that may be employed in the development of the new surcharge rate is illustrated in Example 4.8.

Example 4.8 The Walcott County Wastewater Treatment Facility is currently receiving 1.0 lb/day of cadmium into the plant headworks. Majestic Electroplaters, Inc., is currently permitted to discharge 10 percent of the total cadmium load (0.1 lb/day). Majestic Electroplaters, Inc., would like to expand its operation, which will result in increasing its current wastewater flow rate of 20,000 gal/day to 35,000 gal/day to the municipal sewer. If the cadmium concentration in the new discharge flow rate is unchanged, estimate a reasonable surcharge rate for Majestic Electroplaters, Inc., if the company's present sewer charge is $11,000.00 per year.

solution

Step 1. Estimate the treatment cost per pound of cadmium discharged into the sewer system:

$$\text{Current rate} = \frac{\$11{,}000}{\text{year}} \cdot \frac{1\text{ day}}{0.1\text{ lb cadmium}} \cdot \frac{1\text{ year}}{365\text{ days}}$$

$$= \$301.40/\text{lb of cadmium}$$

Step 2. Estimate the concentration of cadmium in the wastewater from Majestic Electroplaters, Inc.:

$$\text{Concentration (mg/liter)}$$

$$= \frac{\text{mass flow rate (lb/day)}}{[8.34\text{ lb/MG} \cdot \text{(mg/liter)}] \cdot \text{volumetric flow rate (MG/day)}}$$

$$\frac{0.1\text{ lb/day}}{[8.34\text{ lb/MG} \cdot \text{(mg/liter)}] \cdot (20{,}000\text{ gal}/10^6\text{ gal})} \quad 0.6\text{ mg/liter}$$

Step 3. Estimate the new mass flow rate of cadmium due to the expansion in operations:

$$\text{Mass flow rate (lb/day)}$$

$$= \text{concentration (mg/liter)} \cdot \text{flow rate (MG/day)} \cdot [8.34\text{ lb/MG} \cdot \text{(mg/liter)}]$$

$$= 0.6\text{ mg/liter} \cdot (35{,}000\text{ gal}/10^6\text{ gal}) \cdot [8.34\text{ lb/MG} \cdot \text{(mg/liter)}]$$

$$= 0.175\text{ lb cadmium/day}$$

Step 4. Estimate the new annual surcharge rate:

$$\text{New surcharge (\$/year)}$$

$$= \frac{0.175\text{ lb cadmium}}{\text{day}} \cdot \frac{\$301.40}{\text{lb cadmium}} \cdot \frac{365\text{ days}}{\text{year}}$$

$$= \$19{,}252.00/\text{year}$$

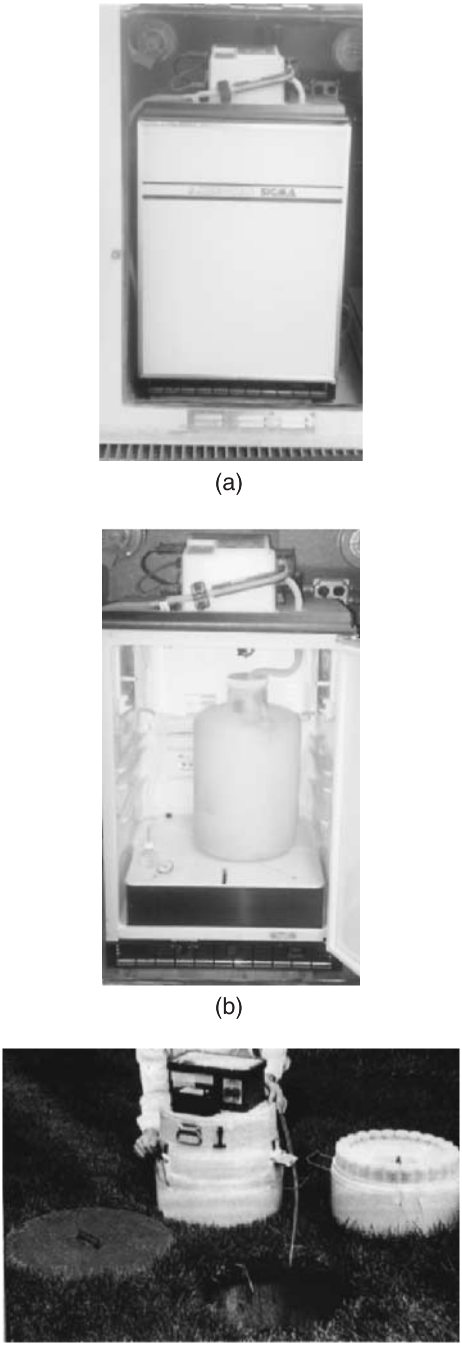
NOTE: This is the minimum surcharge rate that should be levied on the industrial sewer user. Because the expansion of operations by Majestic Electroplaters, Inc., will limit future cadmium discharges from other sources, it is usually desirable to multiply this surcharge rate by a safety factor (normally 2) to encourage greater pretreatment and/or pollution-prevention efforts.

Instead of increasing the industrial surcharge rate, the POTW may choose to assist the industry in identifying potential opportunities for reducing pollutant discharge loadings within the facility. This approach (called *pollution prevention*) is a proactive method that ensures effective wastewater treatment while providing the industrial user economic benefits (e.g., reduced wastewater treatment costs). Pollution prevention is discussed in further detail in Sec. 4.3.

4.1.2.10 Mass-based local limits. Although the pollutant allocation issued to regulated nondomestic sewer users is estimated from the mass-based maximum allowable pollutant headworks loading (e.g., lb/day, kg/day, etc.), most POTWs issue pretreatment permits specifying the allowable concentrations (e.g., mg/liter, lb/gal, etc.) of the regulated pollutants. Concentration-based local limits generally are expressed in milligrams per liter and are averaged over some specified time period to account for the normal fluctuations in industrial acticity. Concentration-based local limits are preferred by the POTWs because compliance is easily verified by taking a wastewater sample at a facility's sewer connection. Mass-based limits, an alternative approach, establishes a specific quantity of a given contaminant that nondomestic sewer users may legally discharge into the sewage-collection system (expressed as pounds per day).

The attractiveness of mass-based local limits stems from the increased technological options and potential financial savings that they offer to the affected industry. For example, if an environmentally conscious company decided to eliminate one part of a waste stream through pollution-prevention efforts and/or the company reduced its water consumption through conservation, the pollutant concentration in its wastewater discharge might increase even though the total mass of pollutant actually decreased. In this case, the company's waste-minimization program would result in a violation of the company's pretreatment permit if its discharge limit were based on a pollutant concentration. Industries that recognize the environmental and economic benefits of minimizing waste generation (including water recycling) should be encouraged to pursue these activities by the POTW through the establishment of mass-based local wastewater discharge limits.

The primary concern regarding the establishment of mass-based local limits is the cost of ensuring permit compliance. To effectively monitor facilities for permit compliance using mass-based limits, POTWs must have reliable data on both the concentrations of pollutants in the discharged wastewater and the wastewater flow rate. This may require modifying the compliance sampling protocol to include installation of a dedicated wastewater flowmeter to monitor industrial flow rates as well as an automatic wastewater pollutant-sampling device. The capital, installation, and maintenance costs for the sampling systems must be negotiated between the POTW and the affected industry. Typical flowmeters used to monitor industrial flow rates are illustrated in Figs. 4.3 and 4.4. Example 4.9 illustrates the use of a mass-based surcharge rate to encourage the increased use of pollution-prevention efforts within an industry.

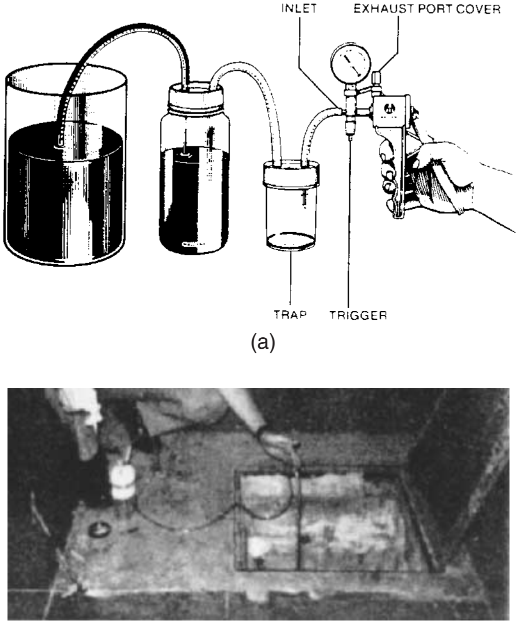


(a)

(b)

(c)

Figure 4.3 (*a*) Refrigerated automatic sampler. (*b*) Refrigerated automatic sampler depicting sampling container. (*c*) A technician setting up a portable automatic sampler.



(a)

(b)

Figure 4.4 (*a*) Manual vacuum pump sampling system. (*b*) Technician taking an industrial wastewater sample with a manual vacuum pump sampler.

Example 4.9 The Medina Aircraft Company is planning to establish a new facility in your sewer district. From industrial records and discussions with facility personnel, you estimate that the new facility will discharge approximately 80,000 gal/day of industrial wastewater into the sewage-collection system with an average nickel concentration of 2.0 mg/liter. As the manager of the POTW pretreatment program, you are concerned about the concentration of nickel in the discharged wastewater because your biosolids frequently have exceeded the beneficial-use criteria established for nickel (see Table 4.8). To reduce the nickel loading to the POTW, the sewer district has decided that of the 10 known industrial dischargers of nickel, no single company can discharge more than 5 percent of the total permissible nickel loading to the POTW headworks.

At a preinspection facility meeting, you indicate to Medina personnel that installing an ion-exchange system to recover and recycle nickel together with industrial process water not only would assist them in meeting their nickel discharge limit but also would result in significant cost savings in terms of water-use costs, raw materials costs, and sewer fees. However, the company claims that the $9000 per year purchase and maintenance costs for an ion-exchange system are too exorbitant.

Assuming that the POTW produces 30,000 gal/day of biosolids with a solids content of 8 percent and that an acceptable POTW nickel removal efficiency is 96 percent, estimate the maximum amount of nickel that any one facility can discharge to the sewer system. Also estimate the percentage of total nickel contributed to the POTW headworks by Medina Aircraft Company. To encourage all industrial sewer users to reduce their nickel discharge, what mass-based surcharge rate should be established for nickel? The local POTW has chosen to limit the nickel concentration in biosolids to the monthly maximum level specified in Table 4.11.

solution

Step 1. From Table 4.11, the monthly average nickel concentration in biosolids is 420 mg/kg. Using Eq. (4.4), the maximum allowable headworks loading of nickel can be estimated:

Headworks loading (lb/day)

$$= \frac{Q_{\text{bio}} \cdot C_{\text{biocrit}} \cdot PS \cdot 8.34}{R_{\text{POTW}}}$$

$$= \frac{(30{,}000 \text{ gal}/10^6 \text{ gal}) \cdot (420 \text{ mg/kg}) \cdot (0.08) \cdot (8.34)}{0.96}$$

$$= 8.76 \text{ lb nickel/day}$$

Therefore, if no one company can discharge more than 5 percent of the total permissible nickel loading, the Medina Aircraft Company is limited to a maximum daily discharge of no more than 0.44 lb of nickel (i.e., 0.05 · 8.76 lb/day).

Step 2. Estimate the mass flow rate of nickel contributed by Medina Aircraft Company if no control technology is employed:

$$\text{lb/day} = (2.0 \text{ mg/liter}) \cdot (80{,}000 \text{ gal/day})/10^6 \text{ gal}$$

$$\cdot (8.34 \text{ MG} \cdot \text{liter})/(\text{day} \cdot \text{mg})$$

$$= 1.33 \text{ lb/day } (487.1 \text{ lb/year})$$

Therefore, Medina Aircraft Company would be contributing approximately 15.2 percent (i.e., 1.33/8.76 · 100) of the total permissible nickel loading to the POTW headworks if no control technology were employed. This would be in clear violation of the sewer ordinance.

To assist Medina Aircraft Company in meeting its local wastewater discharge limits, the POTW pretreatment personnel can identify suitable nickel-control alternatives but cannot require that a specific technology be adopted. However, in the development of a technically justifiable mass-based surcharge rate, the POTW should establish its minimum fee based on a proven control technology.

Step 3. Calculate the mass-based surcharge rate based on the ion-exchange technology being an acceptable and proven nickel-control technology. This approach involves dividing the annual cost of the proven control technology by the annual mass-based discharge limit.

Annual mass-based discharge limit:

$$\text{lb/year} = 0.44\ \text{lb/day} \cdot 365\ \text{days/year} = 160.6\ \text{lb/year}$$

Technology-based nickel surcharge rate:

$$\text{\$/lb nickel} = \frac{\$9000.00/\text{year}}{160.6\ \text{lb nickel/year}} = \$56.04/\text{lb nickel}$$

NOTE: This is the minimum mass-based surcharge rate that should be imposed on each industrial facility. In most cases, this surcharge rate would be multiplied by a safety factor (normally 2) to encourage greater industrial pretreatment and/or use of pollution prevention within the facility. To ensure that the biosolids quality is not compromised, the impact of this surcharge rate should be evaluated with regard to its influence on the behavior of other nondomestic sewer users. In other words, it must be determined whether or not this surcharge rate is sufficient to significantly reduce the discharge of nickel.

4.2 Pollutant Generators of Concern

POTWs may target specific pollutants for pollution prevention because of problems associated with maintaining compliance with their current NPDES permit or because they anticipate problems in meeting future NPDES permit limits. It is essential that the principal generators of the problem pollutant be identified during the initial industrial survey. If the source of a particular pollutant is not identified during the initial nondomestic discharger survey, the POTW pretreatment personnel will need to resurvey sewer users to identify the source(s) of toxic and/or hazardous pollutants [21,34].

4.2.1 Industrial users

To identify an industrial source of a particular pollutant, the POTW should have information on its categorical and other significant industrial and nondomestic sewer users from recent inspections, existing and past pretreatment permits, and industrial waste surveys. Determining which sewer user is discharging the POC involves review of the data POTWs routinely collect on industrial users. Under the general pretreatment regulations of the CWA (i.e., 40 CFR Part 403), POTWs must be notified regarding the types and volumes of toxic and hazardous wastes generated and disposed by their industrial and other nondomestic sewer users [16]. To facilitate identification of new or unknown industrial discharges, POTW personnel may contact local and state agencies to cross-reference records on water users, new utility connections, and building permits.

4.2.2 Commercial users

Commercial and small industrial sewer users (e.g., laundries, dental offices, laboratories, publishing operations, etc.) are sometimes not required to obtain pretreatment permits. However, these facilities may represent a significant portion of the total POC loading entering a POTW. The EPA estimates that approximately 15 percent of all priority pollutants currently entering POTWs originate from commercial and unpermitted small industrial facilities [21].

To define which commercial users discharge POCs, the pretreatment personnel should survey commercial establishments in the POTW service area that are suspected to be discharging a pollutant. Cross-referencing records of businesses with other local or state agencies will assist in identifying unknown or new commercial users to include in the survey. The survey list should include all potential commercial contributors and estimates of sewage discharge concentrations and flows from each facility. The survey should be updated after site inspection to include information about the pollution-prevention measures presently employed at each facility.

4.3 Pollution Prevention

Pollution prevention, or P2, is a proactive approach to environmental protection that can lead to improvements in environmental quality and economic efficiency by reducing harmful pollutants at the source through cost-effective changes in production, operation, and raw materials use. With the enactment of the Pollution Prevention Act of 1990 (1990 PPA), the U.S. Congress formally established pollution prevention as a national objective, placing it ahead of waste recycling, treatment, and disposal in the hierarchy of environmental management methods [39]. Moreover, the 1990 PPA directs the EPA to integrate pollution-prevention concepts fully into all environmental regulatory programs.

POTW pretreatment personnel can broaden their approach to meeting the goals of the national pretreatment program, as well as to improving wastewater treatment efficiency, by encouraging pollution-prevention measures among nondomestic sewer users. Application of pollution-prevention techniques to pretreatment programs allows the POTW to mitigate current and anticipated compliance problems. A

TABLE 4.12 Advantages of Applying Pollution Prevention (P2) to POTW Pretreatment Operations*

- P2 can assist in identifying multimedia transfers of pollutants.
- P2 can increase POTW worker safety and reduce collection system hazards.
- P2 can reduce pollutant pass-through.
- P2 can reduce biosolids management costs.

*Taken from ref. [39].

TABLE 4.13 Advantages of Employing Pollution-Prevention Measures for Industrial, Commercial, and Other Nondomestic Sewer Users*

- Reduction in waste monitoring, treatment, and disposal costs
- Reduction in raw materials use, feed stock purchases, and manufacturing costs
- Reduction in operation and maintenance costs
- Increased productivity and reduce off-specification products
- Reduction in regulatory compliance costs
- Reduction in hazards to employees through exposure to chemicals
- Improved public image and employee morale
- Reduction in potential liability associated with toxic wastes

*Adapted from ref. [39].

summary of the advantages in applying pollution-prevention approaches to POTW pretreatment operations is provided in Table 4.12.

POTWs should focus their pollution-prevention efforts on problem pollutants and identify the industrial, commercial, or other nondomestic pollutant generators of concern. The first steps in initiating a pollution-prevention program include (1) identifying POCs, (2) identifying generators of problem pollutants, and (3) prioritizing the list of sewer users that could reduce the discharge of problem pollutants through implementation of a pollution-prevention plan.

Once a POTW has identified problem pollutants and prioritized the list of industrial, commercial, and other nondomestic users accordingly, it has the option of incorporating pollution prevention into its pretreatment permitting and enforcement activities. Table 4.13 lists some of the advantages of pollution prevention to nondomestic sewer users.

There are basically three approaches available to the POTW for promotion of pollution-prevention concepts within an industry. They include

- Requiring pollution-prevention plans and implementation of best management practices (BMPs).
- Controlling discharges from small industrial and commercial users.
- Employing mass-based local wastewater discharge limits.

Pollution-prevention plans are documents developed by industry and submitted to the regulatory agency that identify potential areas in the facility that could employ pollution prevention activities (see Sec.

4.3.1). Related to pollution prevention activities are best management practices (BMPs), which are defined as cost-effective approaches employed to control discharges of toxic and/or hazardous substances by industries and other nondomestic sewer users into the wastewater collection system [39]. BMPs are developed through negotiations between industry and POTW pretreatment personnel and normally would be included as part of the industry's pretreatment permit.

Controlling discharges from small industrial and commercial sewer users can be accomplished by requiring that all nondomestic sewer users obtain a pretreatment discharge permit. This requirement, which can be mandated through the sewer ordinance, would specify the discharge limits for all nondomestic sewer users. Once a permitting program is established, setting local wastewater discharge limits covering a wide range of pollutants provides a strong incentive for small industrial and commercial sewer users to implement pollution-prevention measures. Finally, by development of a judicious permit fee structure, all nondomestic sewer users can be encouraged to adopt pollution-prevention strategies.

The last approach to encouraging P2 (i.e., mass-based local limits) can result in significant cost savings to nondomestic sewer users if implemented appropriately. By replacing pollutant concentration with pollutant mass loading as the compliance parameter (as well as mass-based surcharge rates), nondomestic sewer users would be encouraged to internally identify and control those processes that result in the generation of the specific pollutant. Moreover, mass-based local discharge limits would encourage sewer uses to seriously consider water-conservation options within their facility. For example, the use of an evaporative recovery system to concentrate and recycle pollutants, as well as minimizing water use, is a pollution-prevention approach commonly used in the metal finishing industry (Fig. 4.5).

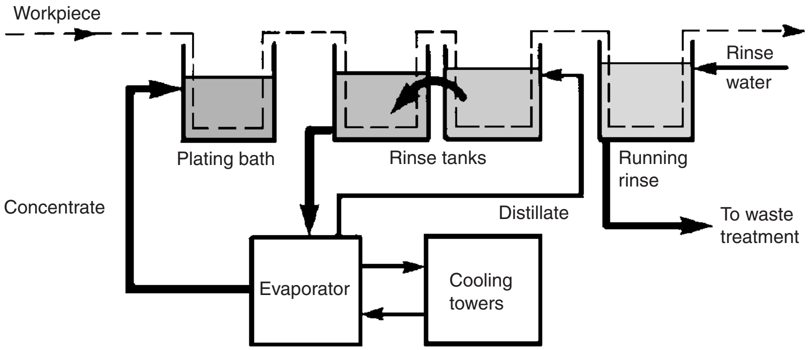


Figure 4.5 Evaporative recovery system for pollutant recycling in the metal plating industry.

4.3.1 Pollution-prevention plans

POTW pretreatment personnel can promote pollution prevention as a means of complying with pretreatment standards by requiring industrial and commercial users to develop and submit pollution-prevention plans as part of the pretreatment permitting process. Pollution-prevention plans contain detailed and systematic assessments of a facility's ability to reduce the volume and toxicity of discharges through pollution-prevention activities [3,39]. At a minimum, a facility pollution-prevention plan would consist of the following items:

- A process flow diagram indicating where toxic pollutants enter and exit the industrial processes
- An estimate of the amount of regulated waste generated by each industrial process
- An evaluation of current and past pollution-prevention activities
- Identification of technically and economically feasible pollution-prevention opportunities
- A P2 implementation timetable

POTW personnel can assist their industrial and commercial sewer users in developing pollution-prevention plans by identifying pollution-prevention opportunities during facility inspections. New or expanding facilities are the nondomestic sewer users most likely to benefit from pollution-prevention planning. Moreover, facilities that have failed to meet discharge limits with traditional treatment technologies will be more inclined to participate in pollution-prevention planning than a facility that successfully meets pretreatment discharge standards.

Industrial pollution prevention encompasses a myriad of approaches to reduce or eliminate toxic and/or hazardous substances from the discharged wastewater stream. Several of the standard approaches include (1) source reduction, (2) material substitutions, (3) product changes, and (4) recycling. Each of these approaches is described briefly in the following sections.

4.3.2 Source reduction

Source reduction is defined as any practice that reduces the amount of any toxic and/or hazardous material, pollutant, or contaminant entering any waste stream (including fugitive emissions) prior to recycling, treatment, or disposal [3,39]. Source reduction can result in significant cost savings to the industry through raw materials conservation and reduction in discharge fees. Source reduction encompasses operational practices, technology changes, input material substitutions, and product changes.

Securing management commitment to pollution prevention is an important first step toward instituting an effective source-reduction program. Once management support is obtained, the next step is to generate a process flow diagram that identifies the inputs and outputs of all toxic and/or hazardous substances from a facility.

With a comprehensive understanding of the input and output mass flows from a process, POTW pretreatment personnel can assist industries in identifying and implementing a source-reduction program. Two of the most effective approaches to source reduction are (1) improved housekeeping practices and (2) technological advances in production [3,39].

4.3.2.1 Improved housekeeping practices. In general, a significant reduction in pollution generation can be recognized from a minimal investment in improved housekeeping practices. These practices include (1) improving inventory control, (2) preventing accidental spills, (3) segregating waste streams, and (4) scheduling production runs that maximize production and minimize waste generation.

Maintaining an orderly inventory system and proper storage conditions can reduce pollutant generation from product deterioration, inefficient materials use, and spills. Implementing a materials tracking system that monitors materials use by individual employees or work groups enables managers to identify production teams with efficient materials use. Other improved housekeeping practices include regularly scheduled preventive maintenance operations that reduce the potential for leaks and spills.

Segregating hazardous and nonhazardous waste streams avoids making the entire waste stream hazardous, thus reducing the volume of waste requiring treatment and/or disposal. Waste segregation also enhances an industry's ability to reclaim recyclable materials [39]. Finally, modification of production schedules to minimize required equipment changeovers is an effective housekeeping practice that can reduce the quantity of waste generated by equipment cleaning.

4.3.2.2 Technological advances in industrial operations. Technological advances can range from minor modifications of existing industrial processes to major investments in new equipment. Technological advances involve changes in any of the following areas:

- Equipment, layout, or piping
- Increased automation
- Production processes/operational conditions

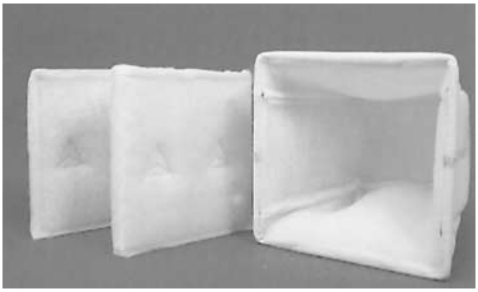


Figure 4.6 Dry particulate filters used in industrial painting and depainting operations. (*Courtesy of TRI-DIM Filter Corporation.*)

An example of an equipment change that could affect biosolids quality is the substitution of dry particular filters for waterfall (i.e., wet scrubber) paint booths in the control of particulate emissions from industrial painting and paint removal operations (Fig. 4.6). Elimination of the scrubber system would result in a reduction in metals loadings to the industrial wastewater discharge [57]. It should be noted that once they are exhausted (as indicated by a significant increase in pressure drop), the dry filters are normally disposed as a hazardous waste.

Of the many pollutants entering the municipal wastewater treatment plant, heavy metals have received the greatest attention particularly with regard to their impact on biosolids quality. Because of the importance of heavy metals in the wastewater stream and the effect of their accumulation in sludge on the available biosolids beneficial use options, efforts to reduce metals loading to the sewer system typically focus on those industrial users that have the greatest potential to discharge heavy metals. One of the most important industries in terms of heavy metals discharges to the municipal sewer is metal finishing. Metal finishing encompasses various process industries, including (1) metal plating, (2) semiconductor processing, (3) printed circuit board manufacturing, etc. The following sections illustrate general approaches to source reduction of metals discharged by metal finishing as well as other related industries.

4.3.3 Source reduction in the metal finishing industry

Approaches to reducing metal discharges from the metal finishing industry include (1) material substitution, (2) waste segregation, (3) process modifications, and (4) capture/concentration techniques. Material substitution involves eliminating the use of toxic materials within the finishing process, such as the substitution of hexavalent chromium and cyanide bearing cleaners/deoxidizers with nontoxic materials. Examples of chemicals often used as functional substitutes

TABLE 4.14 Chemical Substitutes Used in the Metal Finishing Industry*

Toxic chemical	Chemical substitute	Comments
NaCN (fire dip)	Muriatic acid	Substitute is slow-acting
Copper cyanide plating bath	Copper sulfate	Requires good preplate cleaning of surface
Chromic acid pickles, deoxidizers, and bright dips	Sulfuric acid	Nonfuming
Cyanide cleaners	Trisodium phosphate	Good degreasing characteristics when hot and in an ultrasonic bath

*Adapted from refs. [8,13,24,48].

for toxic pollutants (including cyanide) in the metal finishing industry are listed in Table 4.14.

Although materials substitution is an ideal approach for reducing heavy metals concentrations in the sewer discharge, there are several potential disadvantages, including (1) the unavailability of a suitable material substitute, (2) the potential negative effect of a material substitute on product quality and/or production rate, and (3) changes in process/product economics [24].

After eliminating as many toxic pollutants as possible through materials substitution, additional reduction in potential heavy metals sewer discharges can be achieved by segregating heavy metal–containing streams from nonpolluting streams within the facility. The segregation process, which will likely require some physical changes within the facility, is an example of an improved housekeeping practice. Effective segregation of heavy metal–containing process streams may allow the discharging of the nonpolluting streams directly to the sewer (although, in some cases, pH adjustment may be necessary).

Once the nonpolluting solutions have been segregated, solutions containing heavy metals must be treated prior to discharge to the sewer. To minimize treatment costs to the industrial facility, the volume of the metal-contaminated solution should be reduced. The largest volume of heavy metal–contaminated wastes generated within metal finishing operations is produced by chemical drag-out. Drag-out, which is caused by the adherence of residual solution from a concentrated chemical bath to the workpiece prior to rinsing operations, results in a large volume of contaminated rinsewater. To remove residual concentrated solution from the workpiece, gravity-induced drainage must overcome the adhesive force that exists between the solution and the metal surface. Although numerous techniques are available to control drag-out, the effectiveness of each will vary as a function of the metal finishing process, operator cooperation, racking, barrel design, transfer dwell time, and finished part configuration [24,27].

The use of wetting agents and longer workpiece withdrawal/drainage times are two techniques that significantly control drag-out and are used commonly in the metal finishing industry. Wetting agents reduce drag-out by lowering the surface tension of concentrated bath solutions. Lowering the surface tension reduces both the drainage time and the edge effect (i.e., the bead of liquid adhering to the part edge). By proper use of wetting agents, drag-out can be reduced by more than 50 percent [8,24,27].

With slower withdrawal rates and/or longer drain times, drag-out of process solution may be reduced significantly. Where high-temperature finishing solutions are used, slow withdrawal of the rack also may be necessary to prevent evaporative "freezing" that can actually increase drag-out. Figure 4.7 illustrates the drainage rates for plane and bent pieces. Drainage for all shapes typically is complete within 15 seconds after withdrawal, indicating that this is minimum drain time for most workpieces. In addition to wetting agents and changes in withdrawal rates/drying times, drag-out may be reduced by several other techniques, including the use of (1) low-concentration plating

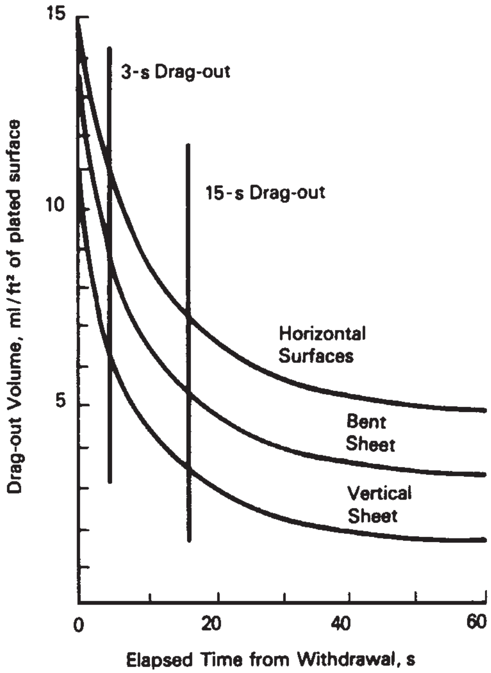


Figure 4.7 Drainage rate of drag-out for various workpieces.

solutions, (2) low-impurity plating baths, (3) high-temperature baths, (4) fog sprays or air knives, (5) proper racking, and (6) capture/concentration with full or partial reuse of drag-out. Each of these techniques is briefly described in the following paragraphs

Low-concentration finishing solutions reduce the mass of chemicals being transferred to the rinsewater by reducing the mass of material that adheres to the workpiece. A potential disadvantage of using low-concentration finishing solutions for metal plating is a lowered plating efficiency. To offset a low plating efficiency, higher current densities and/or closer process control may be required to maintain product specifications.

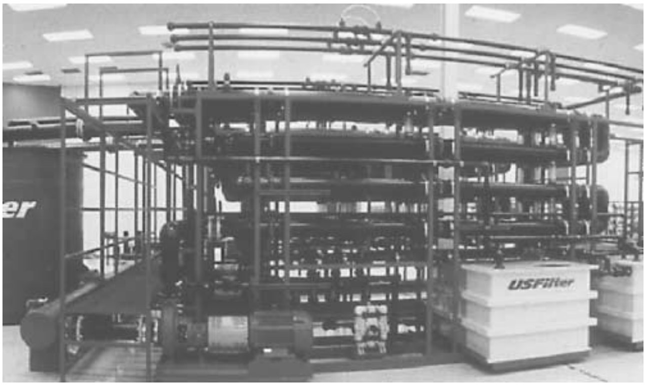
Use of metal finishing baths with high levels of impurities (e.g., carbonate accumulation in cyanide baths) can increase drag-out by as much as 50 percent. The increased drag-out stems from the increase in solution viscosity caused by the significant concentration of contaminants. High solution viscosity reduces the drainage rates, which, in turn, increases drag-out. To minimize increases in solution viscosity, efforts should be made to filter concentrated metal finishing solutions continuously or as part of the regularly scheduled maintenance operations. There are many processes that are now available commercially to continuously remove impurities from plating bath solutions. Most of these systems use precipitation and/or filtration to separate impurities from the concentrated metal finishing solution (Fig. 4.8).

Another approach used to lower both solution surface tension and viscosity of concentrated metal finishing solutions is to employ high-temperature plating baths. Disadvantages in using high-temperature plating baths include (1) rapid solution decomposition, (2) higher energy consumption, and (3) an undesirable dry-on pattern on the workpiece.

In addition to low-concentration plating solutions and/or thermal reduction of solution surface tension/viscosity, physical separation methods are effective in removing residual plating solution from the workpiece prior to rinsing. Fog sprays located just above the solution's surface effectively dilute and drain the adhering drag-out solution, thus reducing the concentration and mass of chemicals lost. Air knives, also located just above the solution surface, reduce the volume of drag-out by mechanically scouring the adhering liquid from the workpiece. By rapidly discharging deionized water (i.e., fog sprays) or air (i.e., air knives), as much as 75 percent of the drag-out can be returned to the finishing solution.

In addition to use of chemical addition and/or physical separation of adhering solution, significant reduction of drag-out also may be achieved by altering the rack positioning of the workpiece. The rationale behind this approach is based on the fact that every workpiece has at least one racking position in which drag-out will be at a minimum. The

(a)



(b)

Figure 4.8 (*a*) Precipitation/filtration system used to continuously remove impurities from plating bath solutions. (*Courtesy of US Filter/JWI.*) (*b*) Microfiltration system used for continuous removal of particulate impurities from concentrated metal finishing solutions. (*Courtesy of US Filter/Memtek.*)

optimal workpiece orientation will provide faster drainage and less drag-out. In general, drag-out can be minimized by (1) racking parts with major surfaces vertically oriented, (2) ensuring that workpieces are not racked directly over one another, and (3) orienting parts so that the smallest surface area of the piece leaves the bath surface last [24,27].

4.3.4 Recycling

Recycling options include both the reuse and reclamation of materials such as solvents, metals, detergents, inks, and other chemicals used in manufacturing processes. Reuse approaches involve substitution of spent input materials for new materials in the manufacturing process, whereas reclamation programs focus on the recovery of valuable material from the spent input materials for incorporation in some other process or product [39].

Recycling can be integrated into an industrial process through a closed-loop system, or it can be accomplished separately using a centralized on-site waste recycling system or commercial materials recycler. Wastes reprocessed or reclaimed can be used on site and/or sold to other industries [39]. For example, recycling in the paint manufacturing industry includes the reformulation of off-specification products into other products or the recycling of rinse tank cleanup solvents [57].

With regard to reuse systems in the metal plating industry, drag-out often can be captured and reused within the plating bath. In a simple drag-out reuse system, a static tank is employed to capture drag-out. A rinse tank that will contain only trace amount of pollutants follows the static drag-out tank(s). The drag-out recycle system is easily applicable to hot plating baths, where the plating bath evaporation rate equals or exceeds the recycle or pour-back rate. The greater the number of counterflow drag-out tanks used in the recycle system, the smaller is the recycle rate necessary to achieve adequate workpiece rinsing. Figure 4.9*a* illustrates a drag-out reuse system in which there is complete drag-out reuse, whereas Fig. 4.9*b* depicts the partial drag-out reuse concept. It should be noted that partial drag-out reuse systems normally employ an additional trickling water supply and drain system.

In addition to reuse of drag-out, reclamation technologies typically employed in the metal finishing industry include (1) electrolytic recovery, (2) evaporation, (3) ion exchange, and (4) reverse osmosis. Each of these technologies is described briefly in the following paragraphs.

Electrolytic recovery employs special electroplating equipment to remove heavy metals, thus lowering the concentration of pollutants in discarded process baths and rinsewaters (Fig. 4.10). There are three variations of the electrolytic heavy metal recovery process in use in the metal finishing industry, including (1) methods used for recovery of rinsewaters with minimum by-product value, (2) recovery on high-surface-area substrate with high metal by-product value, and (3) recovery of high-purity metals in sheets or slabs (i.e., electrowinning). Schematic diagrams of each of the various systems are provided in Fig. 4.11.

Figure 4.12 depicts several commercially available electrolytic heavy metal recovery systems. The design and operational details of electrolytic heavy metal recovery systems are beyond the scope of this text. Readers interested in specific details of electrolytic metal recovery technologies are referred to the following references [8,9,13].

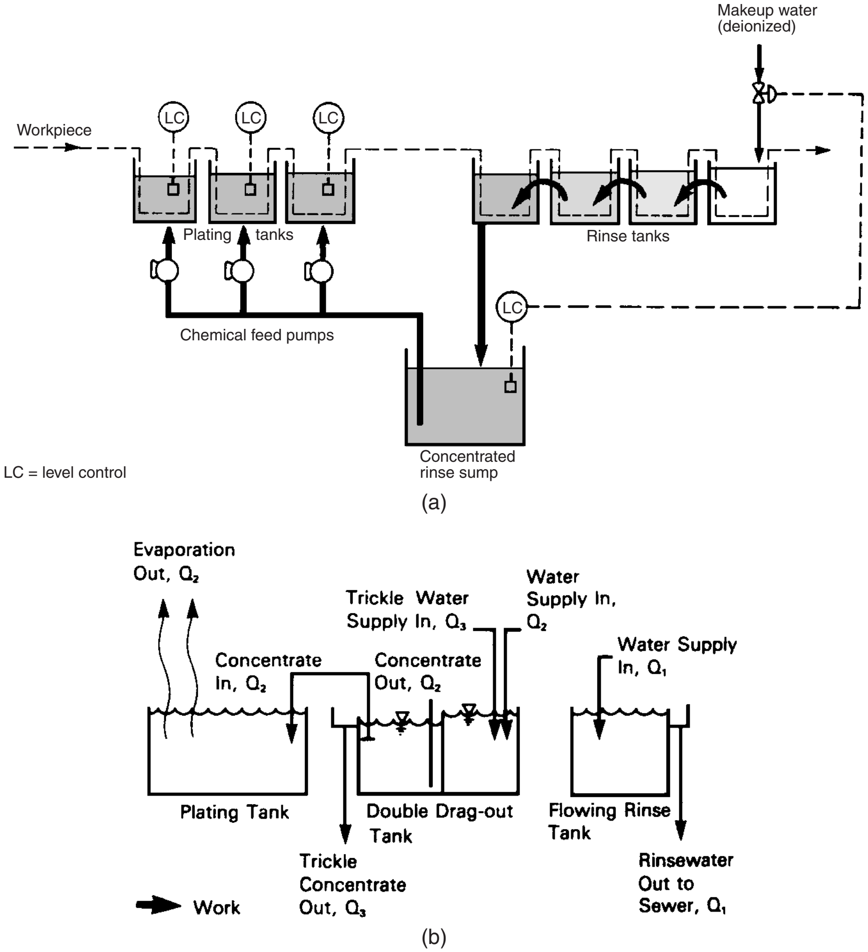


Figure 4.9 (*a*) Schematic diagram of metal recovery system for complete drag-out reuse. (*b*) Schematic diagram of metal recovery system for partial drag-out reuse.

Evaporation involves the vaporization of liquid from a solution. To effectively recycle constituents from solution drag-out, atmospheric evaporation is used to remove excess water from a counterflow rinsewater system before returning the concentrated drag-out to the finishing solution (Fig. 4.13*a*). Because evaporation is an energy-intensive process, it is important to identify cost-effective approaches to conserve energy. For example, part of the heat for moisture evaporation may be recovered when atmospheric evaporators are used to increase evaporative losses from the plating bath. Partial recovery (with lower evaporation rates) results in reduced heat energy cost and slower accumulation of rinsewater impurities in the plating tank. If all the water

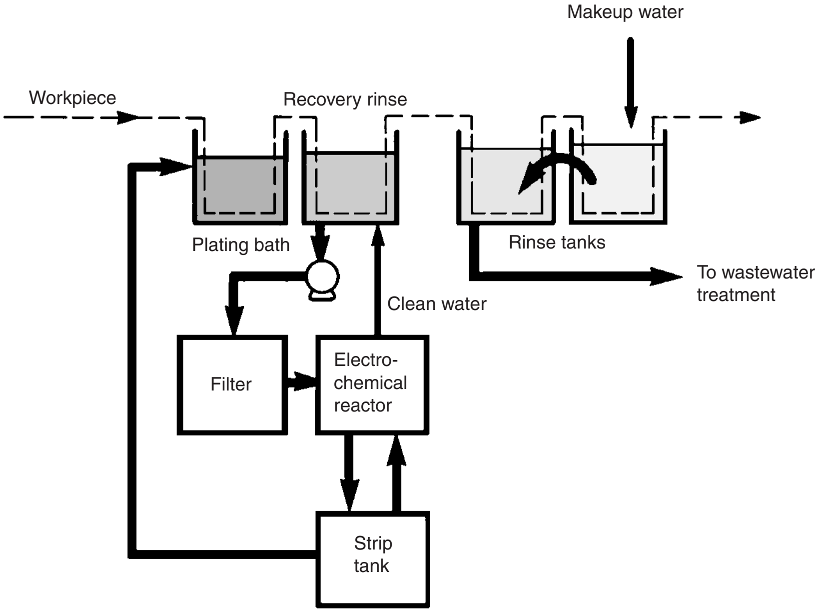


Figure 4.10 Schematic diagram of electrolytic system.

required for rinsing can be evaporated, then the system can be a “closed loop” with no water or drag-out being discharged to the drain (see Fig. 4.13*b*).

Ion exchange involves the capture of metal ions from a wastewater stream based on the characteristics of their electronic charge. Ion-exchange systems are designed to reversibly exchange ions in solution with ions retained on a reactive solid material called *resin.* A typical ion-exchange system has a fixed bed of specially designed resin that promotes the selective removal of metallic cations or anions (e.g., chromate) from rinsewaters. Unlike other metal recovery systems, ion exchange is unaffected by any dilution of the rinsewater being fed to the system. Schematic diagrams of ion-exchange systems for metals recycle and disposal are provided in Fig. 4.14*a* and *b,* respectively.

In reverse osmosis (RO), high pressure (typically 200–1200 lb/in^2) is used to separate water from inorganic (i.e., metal) salts using a semipermeable membrane. Rinsewaters pass through the membrane, leaving behind a solution with a smaller volume and a higher concentration of metals (called *retentate*). The retentate can be recycled to the metal finishing solution (although it may contain high levels of impurities). The recovered water (called *permeate*) can be recycled or treated downstream depending on its quality and the needs of the facility (Fig. 4.15).

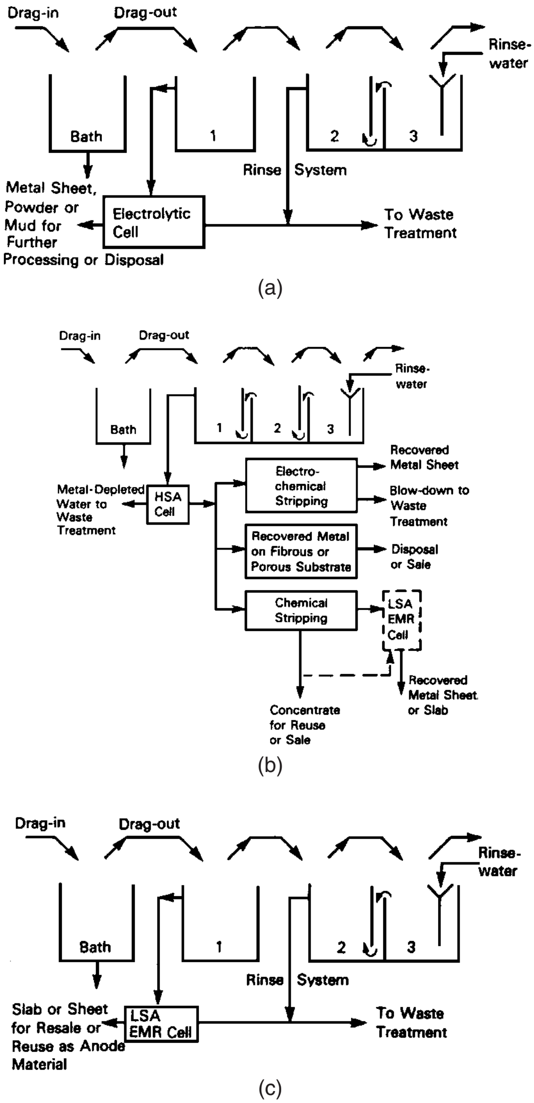
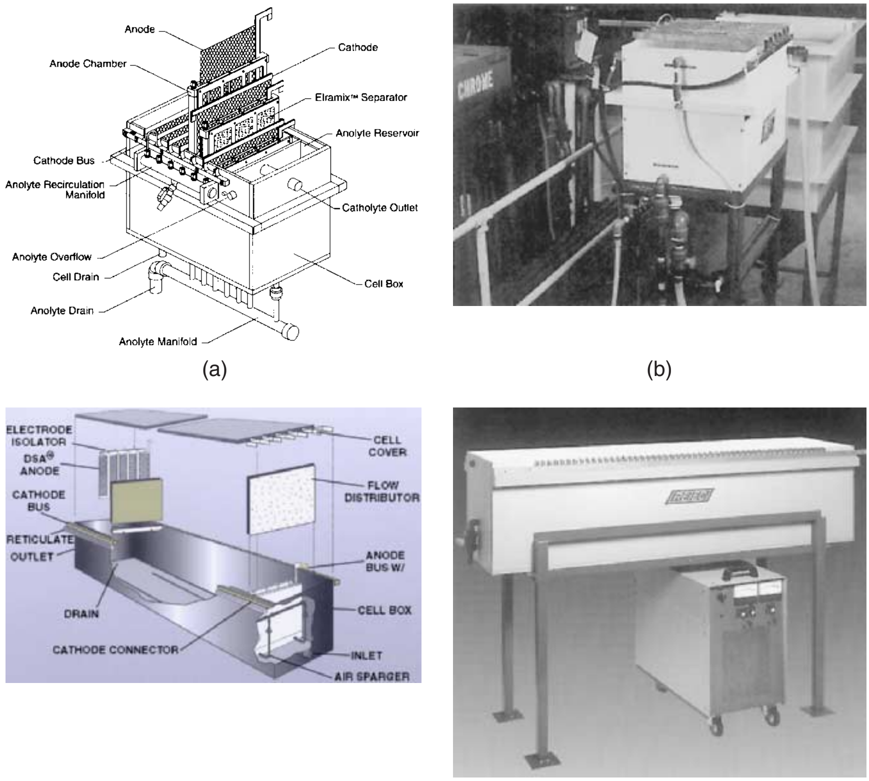


Figure 4.11 (*a*) Schematic diagram of recovery system designed for metals with low by-product value. (*b*) Schematic diagram of system for the recovery of metals with high by-product value. (*c*) Schematic diagram of an electrowinning system.

Although the reverse osmosis process is highly efficient in separating metals, the semipermeable membranes can be fouled (i.e., plugged) or damaged (i.e., holes in membrane with resulting contamination of permeate). To prevent fouling of the membrane, rinsewaters may need to be pretreated to remove oxidizing materials (including manganese,



(a) (b)

(c) (d)

Figure 4.12 (*a*) Schematic diagram of electrolytic chromium recovery cell. (*Courtesy of US Filter/Memteck Products.*) (*b*) Electrolytic chromium recovery cell. (*Courtesy of US Filter/Memteck Products.*) (*c*) Schematic diagram of electrolytic recovery system for recovery of various heavy metals. (*Courtesy of US Filter/Memteck Products.*) (*d*) Electrolytic recovery system for recovery of various heavy metals. (*Courtesy of US Filter/Memteck Products.*)

calcium, lead, and ion salts), particulates, oil and grease, and microorganisms. Typical pretreatment technologies include (1) microfiltration, (2) ultrafiltration, (3) clarification, (4) granular medium filtration, and (5) chlorination. Figure 4.16 illustrates a microfiltration system that may be used as a pretreatment step for reverse osmosis treatment of rinsewaters or as a stand-alone system to remove impurities from metal finishing solutions.

Other, less commonly used heavy metal reuse and recovery technologies employed in the metal finishing industry include (1) carbon adsorption, (2) crystallization, and (3) electrodialysis [24]. Details on these technologies can be found in refs. [8,27]. Although all the technologies identified are available commercially, it should be noted that each technology will vary in complexity, metal removal efficiency, and

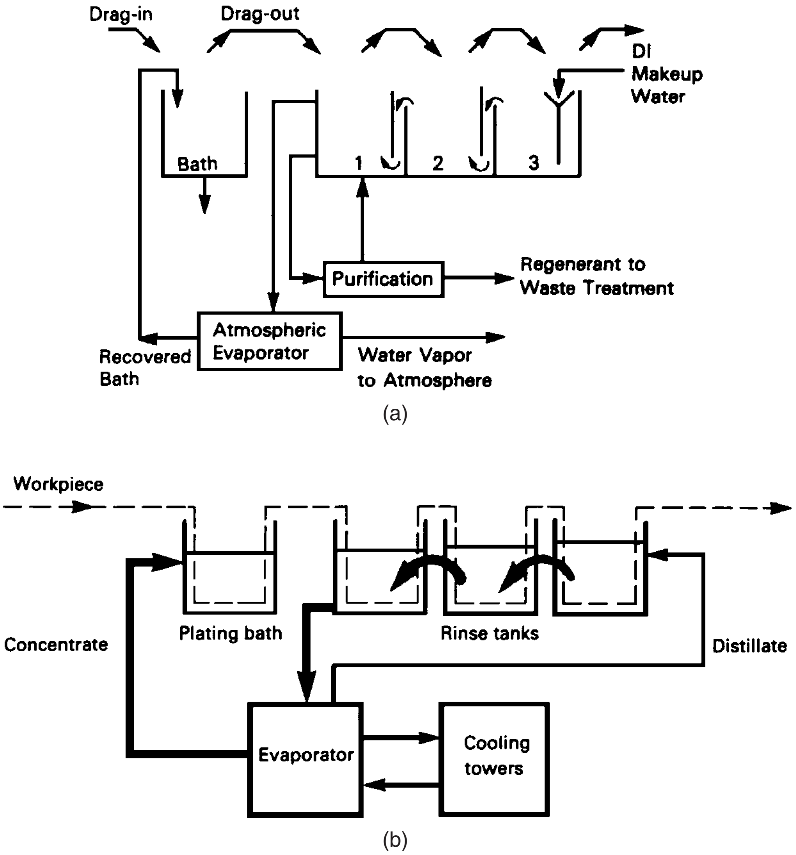
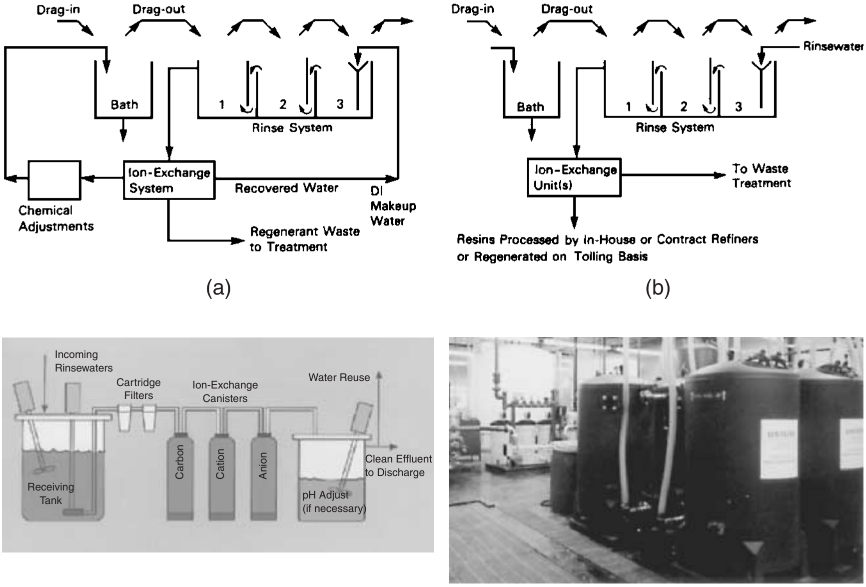


Figure 4.13 (*a*) Schematic diagram of an atmospheric evaporator. (*b*) Schematic diagram of a "closed loop" evaporation system.

cost. In negotiating the pretreatment permit discharge limits with the industrial sewer user, POTW pretreatment personnel should inform the facility about the various source-reduction options but allow the facility the flexibility of choosing its own approach to limiting the metals content of its sewer discharge.

4.3.5 Product changes

Product changes reduce the toxic materials load to the POTW by altering the product so that the use of the problem pollutant is significantly reduced or is no longer required in the manufacturing process. For example, product changes that result in a reduction in the generation of hazardous wastes in the commercial printing industry are illustrated in image processing. Typical compounds found in wastewater generated



(a) (b)

(c) (d)

Figure 4.14 (*a*) Schematic diagram of an ion-exchange system for metal capture and recycle. (*b*) Schematic diagram of an ion-exchange system for metal capture and disposal. (*c*) Schematic diagram of an ion-exchange system. (*Courtesy of US Filter Recovery Services.*) (*d*) Ion-exchange system. (*Courtesy of US Filter Recovery Services.*)

from image processing include silver together with hazardous organic chemicals. Product changes include the use of electrostatic films rather than silver-containing films. Electrostatic films are non-silver-containing films that use an electrostatic charge (rather than chemicals) to render the film light sensitive, thereby reducing wastewater pollutants. Within the metal plating industry, an example of a product change is the use of aluminum ion vapor deposition in lieu of cadmium plating in the corrosion protection of steel [48]. Hazardous materials are not used in this process, nor are any hazardous wastes generated.

Although a proactive approach to reducing the discharge of industrial contaminants, implementation of product changes has several potential disadvantages, including (1) identification of a reliable and effective product change, (2) negative impact of product change on product quality and/or production rate, and (3) increased production costs.

4.3.6 Pollution-prevention resources

With a statutory mandate to incorporate pollution prevention into all federal environmental programs, the U.S. Environmental Protection Agency (EPA) has established the Office of Pollution

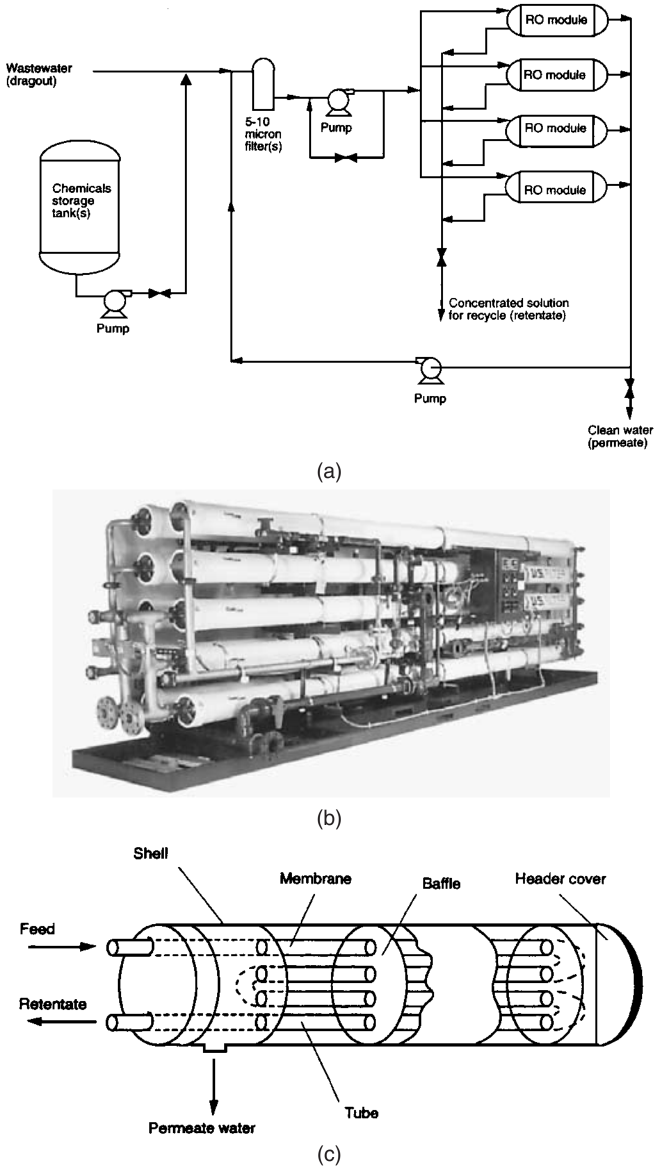


Figure 4.15 (*a*) Schematic diagram of a reverse osmosis system used to treat drag-out. (*b*) Photograph of a reverse osmosis system for rinsewater recycle. (*Courtesy of US Filter/Memteck Products.*) (*c*) Schematic diagram of a tubular reverse osmosis module.

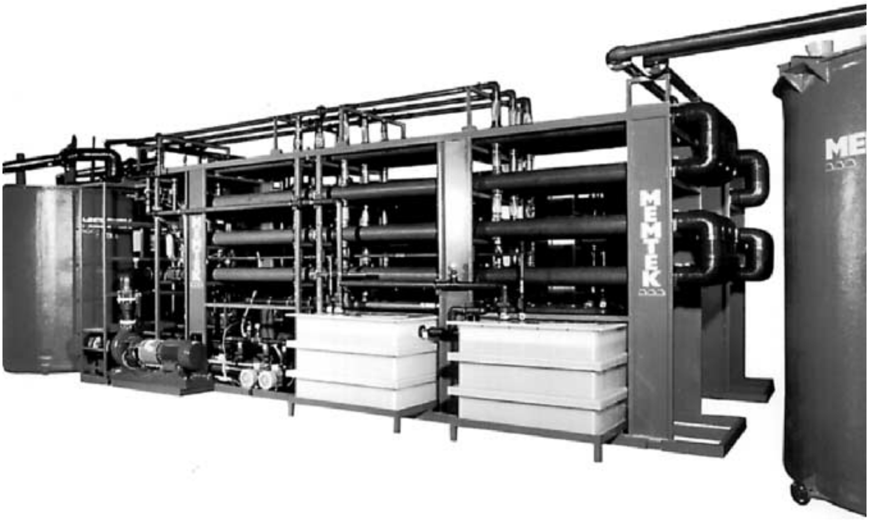


Figure 4.16 A microfiltration system used as a pretreatment step to a reverse osmosis system. (*Courtesy of US Filter/Memtek Products.*)

Prevention and Toxics (OPPT) and the Pollution Prevention Information Clearinghouse (PPIC). OPPT has primary responsibility for the implementation of the 1990 Pollution Prevention Act and to ensure that all EPA policies and programs explicitly address pollution prevention in their implementation [39]. While OPPT manages the regulatory implementation of pollution prevention for the EPA, PPIC functions as a national repository for technical, policy, programmatic, legislative, and financial information on pollution prevention. PPIC maintains the EPA's pollution-prevention Internet home page and provides pollution-prevention documents and fact sheets to the public free of charge. The PPIC pollution-prevention resources can be accessed on the World Wide Web at the following address: *http://www.epa.gov/opptintr/library/libppic.htm.*

4.4 Facility Inspections

One of the most effective approaches for encouraging pollution prevention among nondomestic sewer users is to identify pollution-prevention opportunities for the affected industry during scheduled facility inspections. For a permitted facility, POTW personnel should review information relating to the facility's compliance history. Compliance data will allow POTW personnel to focus preinspection efforts on pollution-prevention options that address the facility's general compliance problems. With an understanding of the specific industrial processes, types of inputs they require, and the waste streams they generate, POTW personnel can help

TABLE 4.15 Three-Step Approach to Assist POTW Personnel to Identify Potential Areas for Implementation of Pollution-Prevention Measures*

1. Construct a general process flow diagram of the operation identifying all inputs and outputs to the process, including raw materials inputs, product outputs, material recovery, and waste streams.
2. Perform a materials balance to identify process steps that result in significant material loss.
3. Identify areas other than the process areas, such as material storage, where significant losses can occur.

*Adapted from ref. [39].

identify potential problem areas and initiate discussion with facility personnel about implementing pollution-prevention measures.

Pretreatment personnel should consult and coordinate with the appropriate federal, state, and local agencies prior to embarking on a major pollution-prevention initiative. A coordinated effort with other federal, state, and local programs could avoid unnecessary duplication of effort among environmental and public health agencies. In addition, a coordinated effort will be met with less confusion and suspicion on the part of targeted industries. A three-step approach that can assist POTW pretreatment personnel in identifying industrial process areas where pollution-prevention measures could reduce toxic and/or hazardous loadings to the sewers is provided in Table 4.15.

Based on the general process flow diagram, POTW pretreatment personnel can track the POC from its point of origin in the raw materials inputs to the resulting products and waste streams. To provide confidence in the process tracking analysis, the mass of input materials should be approximately the sum of the mass of materials output in the product plus the amount contained in the recovered materials. A significant difference between POC materials input and output from the process indicates losses of materials that should be investigated. Examples of typical processes employed in industrial coating operations that lend themselves to mass-balance analyses are illustrated in Fig. 4.17. The difference between the mass of coating applied to and the mass of coating associated with the finished product represents the amount of material that leaves the process as waste.

In addition to the process areas, POTW pretreatment personnel should investigate storage areas, pumping stations, laboratories, boiler areas, etc. to determine whether good housekeeping practices are being applied to prevent or minimize the discharge of pollutants to the POTW. Special attention should be paid to potential discharges to floor drains and whether further improvements in existing housekeeping practices may be appropriate. Based on knowledge of the industry,

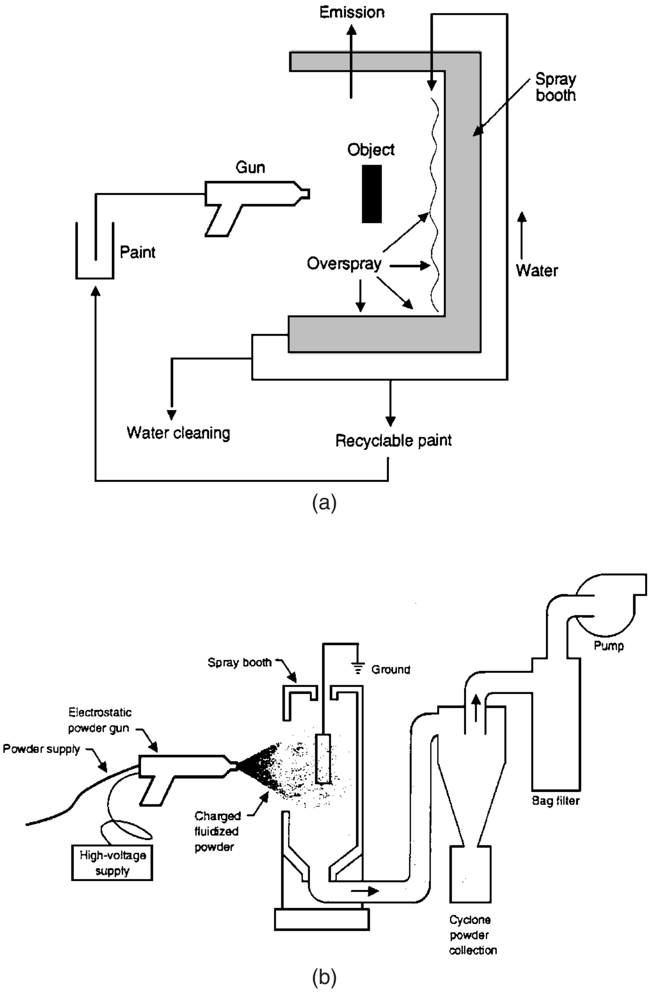


Figure 4.17 (*a*) High-volume, low-pressure (HVLP) coating application in a waterfall spray booth. (*b*) Electrostatic powder coatings in a dry spray booth. Note that in the electrostatic powder coating system no wastewater is generated. (*Courtesy of Eastern Research Group.*)

POTW pretreatment personnel can identify any periodic maintenance activities, such as equipment or tank cleaning, boiler blowdown, and motor fluid changes, that can result in a significant quantity of wastes being discharged to the sewer.

Once a preliminary assessment of the facility is conducted, the POTW pretreatment investigators should focus on collecting as much information as possible about the pollution-prevention opportunities available for the facility under investigation. This information can be used to educate facility owners and operators about the usefulness of pollution-prevention measures. The final decision regarding the implementation of any pollution-prevention measure will be made by the facility based on technical as well as economic factors.

POTWs interested in inspecting unpermitted industrial and commercial facilities may have greater difficulty in obtaining facility-specific process data. Options in such cases include

- Reviewing industrial waste survey data
- Contacting other federal, state, and local environmental public health program offices that may have collected facility-specific information
- Requesting process data and information directly from the facility

4.4.1 Inspection procedures

Most facility inspections begin with an introductory preinspection meeting where POTW pretreatment personnel can inform facility personnel about pollution prevention as a means of reducing toxic and/or hazardous discharges to the sewers and achieving long-term compliance with pretreatment discharge limits [39]. Topics to cover in the opening meeting include

- What pollution prevention is and why it is important to the POTW. POTW personnel should emphasize how the specific facility may benefit from implementing a pollution-prevention program.
- Current and potential future sewer user compliance concerns based on existing and anticipated POTW compliance needs and how pollution prevention could help address these concerns.
- A review of the types of pollution-prevention programs the facility has already implemented and the level of success that the facility has had with those measures.

Following the introductory meeting, the actual facility inspection provides an opportunity for pretreatment personnel to observe operations. Identification of pollution-prevention opportunities can be accomplished by (1) making observations and asking insightful questions, (2) offering appropriate advice, and (3) highlighting pollution prevention in the exit inspection meeting.

During actual facility inspections, the POTW pretreatment personnel should recognize the potential of cross-media transfers of pollutants. For example, use of wet scrubbers to remove particulate air emissions will transfer toxic pollutants to the facility's wastewater discharge. Coordination among officials in the local hazardous and solid waste, air, and water programs can assist POTW pretreatment personnel in identifying and uncovering these cross-media transfers.

4.5 Pretreatment Noncompliance

As part of their normal pretreatment permitting and inspection activities, POTWs can encourage pollution prevention among nondomestic sewer users, but they cannot require specific measures beyond those considered best management practices (BMPs). In response to user noncompliance, however, a POTW can require specific pollution-prevention measures as part of a mutually agreed on compliance schedule with the nondomestic sewer user.

In the corrective action plan, POTWs can require facilities in noncompliance to (1) conduct pollution-prevention planning, (2) identify cost-effective pollution-prevention measures, and (3) develop an implementation schedule with interim and final milestones. The implementation schedule can then be incorporated into a binding compliance schedule. The noncompliant sewer user can be required to evaluate pollution-prevention options but should be allowed the flexibility to develop a corrective action plan that includes the most effective mix of pollution-prevention measures and traditional treatment options [39].

4.6 Problems

4.1 The Rumbek City Wastewater Treatment Plant (WWTP) desires to reduce the nickel concentration in its wastewater effluent by installing a chemical flocculation system. The result of sampling the plant influent and effluent metal concentrations over a 5-day period has generated the following mass flow rates for nickel (Ni). Given the metal-removal data, estimate the average daily and mean daily nickel removal efficiencies.

Sample (day)	Influent mass (lb Ni/day)	Effluent mass (lb Ni/day)
1	30	7
2	22	6
3	41	9
4	27	10
5	33	7

4.2 The Tubmanburg County Sewer Improvement District has completed the start-up of a new activated sludge wastewater treatment plant. Since the plant effluent will be discharged into an irrigation ditch during the summer months and a nearby wetland during the remainder of the year, the district manager is concerned about the ability of the facility to remove copper from its effluent wastewater flow. Given the following data from a 5-day trial test, estimate the average daily and mean daily copper (Cu) removal efficiencies.

Sample (day)	Average flow rate (million gal/day)	Influent (Cu) conc. (mg/liter)	Effluent (Cu) conc. (mg/liter)
1	2.2	14.7	0.9
2	2.7	19.2	1.3
3	1.9	28.5	1.4
4	2.4	21.6	1.0
5	2.6	25.3	1.6

4.3 In order to establish local industrial wastewater discharge limits, the Numan County Water Reclamation Plant desires to statistically characterize the overall selenium (Se) removal efficiency at the facility. If the following data represent the daily selenium removal efficiencies recorded over a 3-week period, estimate the value of the removal efficiencies corresponding to the 9-decile positions.

Day	Removal efficiency (% Se)	Day	Removal efficiency (% Se)
1	68.6	12	91.0
2	57.3	13	51.3
3	72.1	14	41.4
4	58.7	15	64.2
5	67.5	16	65.3
6	85.3	17	78.1
7	91.4	18	74.6
8	56.8	19	90.5
9	70.6	20	45.7
10	86.1	21	83.9
11	69.3		

4.4 The Sokoto City Wastewater Treatment Plant has identified trichloroethylene (TCE) as a pollutant of concern (POC) from its wastewater pollutant scan. If the average wastewater flow rate is 12 million gal/day, estimate the maximum allowable headworks loading of TCE in pounds per day based on effluent wastewater quality. Assume that the TCE maximum regulatory discharge concentration is 0.005 mg/liter and that the average TCE removal efficiency at the plant is 98 percent.

4.5 The Sokoto City Wastewater Treatment Plant (see Prob. 4.4) has decided to install an activated carbon filter to increase the TCE removal efficiency at the facility to 99.9 percent. If the maximum regulatory discharge concentration and average plant flow rate remain unchanged at 0.005 mg/liter and 12 million gal/day, respectively, estimate the maximum allowable headworks loading of TCE in pounds per day based on effluent wastewater quality.

4.6 The Sikasso City Water Reclamation Plant is currently generating 26,000 gal of liquid biosolids having a solids content of 3.0 percent. If the maximum zinc concentration permissible in locally land-applied biosolids is 2800 mg/kg (dry-weight basis), estimate the maximum headworks loading for zinc based on biosolids quality. Assume that the average wastewater flow rate to the facility is 6 million gal/day and that the average zinc removal efficiency at the plant is 97 percent.

4.7 The Carnot County Sewer Improvement District is currently treating 10 million gal/day of municipal wastewater. If the regulatory authorities have established a maximum cadmium concentration in the wastewater effluent and land-applied biosolids at 0.01 mg/liter and 39 mg/kg (dry-weight basis), respectively, estimate the maximum headworks loading for cadmium based on both effluent and biosolids quality. Assume that the facility is generating 19,000 gal of liquid biosolids having a 4.5 percent solids content and that the following table provides the 9-decile levels computed from the daily cadmium removal efficiencies from the treated wastewater.

Decile	Cadmium removal efficiency (%)
1	31.2
2	45.3
3	56.4
4	61.2
5	72.1
6	78.9
7	84.1
8	89.4
9	98.2

4.8 The Turkana City Water Reclamation Plant is generating 38,000 gal/day of liquid biosolids having a solids content of 3.8 percent. If the local regulatory authorities have set a maximum lead concentration in the wastewater effluent and land-applied biosolids at 0.05 mg/liter and 250 mg/kg (dry-mass basis), respectively, estimate the maximum headworks loading for lead based on both effluent and biosolids quality. Assume that the wastewater flow rate is 15 million gal/day and that the table on the next page provides the 9-decile levels computed from the daily lead removal efficiencies.

4.9 The Eldoret City Wastewater Treatment Plant desires to establish a maximum allowable headworks loading for benzo(*a*)pyrene based on biosolids quality. From sampling both wastewater and biosolids, it has been determined that the average daily loading of benzo(*a*)pyrene to the facility and benzo(*a*)pyrene concentration in biosolids are 1.29 lb/day and 0.093 mg/kg (dry-weight basis), respectively. If the local health department has established a maximum concentration of benzo(*a*)pyrene in land-applied biosolids at 0.001 mg/kg, estimate the maximum allowable headworks loading for benzo(*a*)pyrene.

Decile	Lead removal efficiency (%)
1	29.5
2	36.4
3	48.5
4	57.2
5	68.9
6	80.1
7	85.2
8	91.3
9	97.9

4.10 The Odongo County Sewer Improvement District desires to establish a maximum allowable headworks loading for dioxin based on biosolids quality. The local health department has established a maximum concentration of dioxin in land-applied biosolids at 0.0003 mg/kg (dry-weight basis). Estimate the maximum allowable headworks loading for dioxin if, from sampling both wastewater and biosolids, it has been determined that the average daily loading of dioxin to the facility and dioxin concentration in biosolids are 0.52 lb/day and 0.047 mg/kg (dry-weight basis), respectively.

4.11 The Makurdi City Wastewater Treatment Plant is currently generating 18,000 gal/day of liquid biosolids having a 3.5 percent solids content. If a required land-application site of 120 acres was estimated based on nutrient considerations, estimate (A) the maximum concentration of nickel permissible in biosolids if the site is to be operated for 30 years and (B) the theoretical maximum allowable headworks loading for nickel. Assume that the maximum nickel loading in land-applied biosolids is 420 kg/ha (374.8 lb/acre) and that the POTW has an average nickel-removal efficiency of 97.0 percent.

4.12 The Gemena County Watershed Protection District has established a maximum arsenic loading in land-applied biosolids at 41 kg/ha (36.6 lb/acre). If the local wastewater treatment plant is currently generating 12,000 gal/day of biosolids having a 10 percent solids content, estimate (A) the maximum concentration of arsenic permissible in biosolids if the site is to be operated for 25 years and (B) the theoretical maximum allowable headworks loading for arsenic. Assume that, based on the nitrogen content of the biosolids, the area for biosolids land application was estimated to be 140 acres and that the average arsenic-removal efficiency at the facility is 96 percent.

4.13 The Onyullo Galvanizing Company, Inc., is currently permitted to discharge 1.5 lb/day of zinc into the municipal sewer from its industrial operations. The Onyullo Galvanizing Company would like to expand its operations, which will result in increasing its current wastewater flow rate of 30,000 to 75,000 gal/day to the municipal sewer. If the zinc concentration in the new discharge flow rate is to remain unchanged, estimate a surcharge rate for the Onyullo Galvanizing Company if the company's present sewer charge is $6500 per year.

4.14 Omondi Electroplaters, Inc., is currently discharging 125,000 gal/day of industrial wastewater into the public sewer system. Because of a planned expansion, Omondi Electroplaters has filed a permit request to increase its industrial discharge rate to 425,000 gal/day. In the permit application, Omondi Electroplaters states that it will install an ion-exchange system to reduce the average chromium concentration in its industrial wastewater flow from its current level of 0.85 mg/liter to 0.10 mg/liter prior to discharge into the municipal sewer. If the current sewer charge for Omondi Electroplaters is $23,800 per year, estimate the minimum surcharge rate that should be imposed on the facility as a result of its expansion.

4.15 The Nakuru County industrial park is currently discharging 250,000 gal of industrial wastewater per day from its circuit board manufacturing facility into the local municipal sewer system. The local municipal wastewater treatment plant (POTW) has identified copper as the limiting pollutant for land application of its biosolids. The local public health department has established a maximum copper concentration in any land-applied biosolids at 2100 mg/kg (dry-weight basis). Moreover, to reduce the copper loading to the local POTW, the sewer district has decided that no single industrial sewer user can discharge more than 2 percent of the total permissible copper loading to the POTW headworks. Assuming that the POTW produces 42,000 gal of biosolids per day with a solids content of 3 percent and that the average POTW copper removal efficiency is 98 percent, estimate the maximum copper concentration in the wastewater effluent discharged by the Nakuru County industrial park.

4.7 References

1. Bastian, R. K. (1997), "Biosolids Management in the United States," *Water Environment and Technology,* pp. 45–50.
2. Bastian, R. K. (1997), "The Biosolids (Sludge) Treatment, Beneficial Use and Disposal Situation in the USA," *European Water Pollution Control,* pp. 62–79.
3. Bishop, P. L. (1999), *Pollution Prevention: Fundamentals and Practice,* McGraw-Hill, New York.
4. Brobst, R. B. (1999), "EPA's Biosolids Data Management System and Plans for Evaluating Biosolids Quality," WEF/ASSA Joint Residuals and Biosolids Management Conference, Charlotte, NC.
5. Curran, M. (1996), *Environmental Life-Cycle Assessment,* McGraw-Hill, New York.
6. Eckenfelder, W. W. (1989), *Industrial Water Pollution Control,* 2d ed., McGraw-Hill, New York.
7. Garvey, D., C. Guarino, and R. Davis (1993), "Sludge Disposal Trends Around the Globe," *Water/Engineering & Management,* pp. 17–20.
8. Hartinger, L. (1994), *Handbook of Effluent Treatment and Recycling for the Metal Finishing Industry,* ASM International.
9. Kerri, K. (1993), *Treatment of Metal Waste Streams,* California State University, Hornet Foundation, Inc.
10. Kerri, K. (1994), *Industrial Waste Treatment Field Study Program,* California State University Grant no. T900690010.
11. LaGrega, M. D., P. L. Buckingham, and J. C. Evans (1994), *Hazardous Waste Management,* McGraw-Hill, New York.
12. Loehr, R., W. Jewell, J. Novak, W. Clarkson, and G. Friedman (1979), *Land Application of Wastes,* Vols. 1 and 2, Van Nostrand Reinhold, New York.
13. Lowenheim, F. A. (1974), *Modern Electroplating,* John Wiley & Sons, New York.

14. McCabe, W. L., J. C. Smith, and P. Harriot (1993), *Unit Operations of Chemical Engineering,* McGraw-Hill, New York.
15. National Research Council (1996), *Use of Reclaimed Water and Sludge in Food Crop Production,* National Academy Press, Washington.
16. Parkin, W. P. (1993), *The Complete Guide to Environmental Law,* Specialty Technical Publishers.
17. Peavy, H. S., D. R. Rowe, and G. Tchobanoglous (1985), *Environmental Engineering,* McGraw-Hill, New York.
18. Perry, R. H., and C. H. Chilton (1973), *Chemical Engineers Handbook,* McGraw-Hill, New York.
19. Porter, M. C. (1990), *Handbook of Industrial Membrane Technology,* Noyes Publications.
20. Tchobanoglous, G. (1991), *Wastewater Engineering: Treatment Disposal and Reuse,* 3d ed., McGraw-Hill, New York.
21. U.S. Environmental Protection Agency (1979), "Fate of Priority Pollutants in Publicly Owned Treatment Works," EPA/440/1-79/300, Washington.
22. U.S. Environmental Protection Agency (1979), "Inspectors Guide for Evaluation of Municipal Wastewater Treatment Plants," EPA/430/9-79/010, Washington.
23. U.S. Environmental Protection Agency (1979), "NPDES Compliance Sampling Manual," PB81-153215, Washington.
24. U.S. Environmental Protection Agency (1985), "Environmental Regulations and Technology: The Electroplating Industry," EPA/625/10-85/001, Washington.
25. U.S. Environmental Protection Agency (1985), "Handbook Estimating Sludge Management Costs," EPA-625/6-85010, Washington.
26. U.S. Environmental Protection Agency (1987), "Guidance Manual on the Development and Implementation of Local Discharge Limitations under the Pretreatment Program," PB92-129188, Washington.
27. U.S. Environmental Protection Agency (1987), "Meeting Hazardous Waste Requirements for Metal Finishers," EPA/625/4-87/018, Washington.
28. U.S. Environmental Protection Agency (1989), "POTW Sludge Sampling and Analysis Guidance Document," EPA/833/B-89/100, Washington.
29. U.S. Environmental Protection Agency (1990), "40 CFR Part 503, National Sewage Sludge Survey: Availability of Information and Data and Anticipated Impacts on Proposed Regulations," *Federal Register* 55(218): 47210–47283.
30. U.S. Environmental Protection Agency (1990), "Guidance for Writing Case-By-Case Permit Requirements for Municipal Sewage Sludge," EPA/505/8-90/001, Washington.
31. U.S. Environmental Protection Agency (1990), "Guides to Pollution Prevention: The Paint Manufacturing Industry," EPA/625/7-90/005, Washington.
32. U.S. Environmental Protection Agency (1990), "Guides to Pollution Prevention: The Fabricated Metal Products Industry," EPA/625/7-90/006, Washington.
33. U.S. Environmental Protection Agency (1991), "Cooperative Testing of Municipal Sewage Sludges by the Toxicity Characteristic Leaching Procedure and Compositional Analysis," EPA/430/09-91-007, Washington.
34. U.S. Environmental Protection Agency (1991), "Residential and Commercial Toxic Pollutant Loadings and POTW Removal Efficiency," PB93-209872, Washington.
35. U.S. Environmental Protection Agency (1991), "Guides to Pollution Prevention: The Fiberglass-Reinforced and Composite Plastics Industry," EPA/625/7-91/014, Washington.
36. U.S. Environmental Protection Agency (1992), "Determining Wastewater User Service Charge Rates: A Step-by-Step Manual," EPA/832/B-92/003, Washington.
37. U.S. Environmental Protection Agency (1992), "Environmental Regulations and Technology: Control of Pathogens and Vector Attraction in Sewage Sludge," EPA/625/R-92/013, Washington.
38. U.S. Environmental Protection Agency (1992), "Guides to Pollution Prevention: Metal Casting and Heat Treating Industry," EPA/625/R-92/009, Washington.
39. U.S. Environmental Protection Agency (1993), "Guides to Pollution Prevention: Municipal Pretreatment Programs," EPA/625/R-93/006, Washington.

40. U.S. Environmental Protection Agency (1993), "Regulatory Impact Analysis of the Part 503 Sewage Sludge Regulation," EPA/821/R-93/006, Washington.
41. U.S. Environmental Protection Agency (1993), "Standards for Use or Disposal of Sewage Sludge. Final Rule, 40 CFR Part 503," *Federal Register* 58(32): 9248–9415.
42. U.S. Environmental Protection Agency (1993a), "Technical Support Document for Land Application of Sewage Sludge," Vol. I, PB93-110575; Vol. II, PB93110583, Washington.
43. U.S. Environmental Protection Agency (1993b), "Standards for the Use or Disposal of Sewage Sludge," *Federal Register* 58(32): 9248–9415.
44. U.S. Environmental Protection Agency (1994), "Biosolids Recycling: Beneficial Technology for a Better Environment," EPA/832/R-94/009, Washington.
45. U.S. Environmental Protection Agency (1994), "Guides to Pollution Prevention: Organic Coating Removal," EPA/625/R-93/015, Washington.
46. U.S. Environmental Protection Agency (1994), "Guides to Pollution Prevention: Alternatives to Chlorinated Solvents for Cleaning and Degreasing," EPA/625/R-92/016, Washington.
47. U.S. Environmental Protection Agency (1994), "Guides to Pollution Prevention: Organic Coating Removal," EPA/625/R-93/009, Washington.
48. U.S. Environmental Protection Agency (1994), "Guide to Cleaner Technologies: Alternative Metal Finishes," EPA/625/R-94/007, Washington.
49. U.S. Environmental Protection Agency (1994), "Land Application of Sewage Sludge: A Guide for Land Appliers on the Requirements of the Federal Standards for the Use or Disposal of Sewage Sludge, 40 CFR Part 503," EPA/831/B-93/002b, Washington.
50. U.S. Environmental Protection Agency (1995), "A Guide to the Biosolids Risk Assessments for the EPA Part 503 Rule," EPA/832/B-93/005, Washington.
51. U.S. Environmental Protection Agency (1995), "Part 503 Implementation Guidance," EPA/833/R-95/001, Washington.
52. U.S. Environmental Protection Agency (1995), "Process Design Manual: Land Application of Sewage Sludge and Domestic Septage," EPA/625/R-95/001, Washington.
53. U.S. Environmental Protection Agency (1995), "Process Design Manual: Surface Disposal of Sewage Sludge and Domestic Septage," EPA/625/R-95/002, Washington.
54. U.S. Environmental Protection Agency (1996), "Capsule Report: Evaporation Process," EPA/625/R-96/008, Washington.
55. U.S. Environmental Protection Agency (1996), "Capsule Report: Reverse Osmosis Process," EPA/625/R-96/009, Washington.
56. U.S. Environmental Protection Agency (1996), "Technical Support Document for the Round Two Sewage Sludge Pollutants," EPA/822/R-96/003, Washington.
57. U.S. Environmental Protection Agency (1996), "Manual Pollution Prevention in the Paints and Coatings Industry," EPA/625/R-96/003, Washington.
58. U.S. Environmental Protection Agency (1999), "Environmental Regulations and Technology: Control of Pathogens and Vector Attraction in Sewage Sludge," EPA/625/R-92/013, Washington.

Source: Biosolids Engineering

Chapter
5

Transport, Storage, and Facilities Design

5.0 Introduction

The fundamental objective of all wastewater treatment plant operations is to remove pollutants from wastewater and to consolidate them for further processing and/or disposal. Depending on the characteristics of the influent wastewater and the type of wastewater treatment to which it is subjected, the volume of the solids generated can vary significantly. Within the publicly owned treatment works (POTWs), several types of solids accumulate and must be collected periodically and transported, including screenings, grit, scum, and biosolids/sludge. Since the collection and transportation of these various types of solids from point to point within a POTW for treatment, storage, and disposal, as well as the transport of biosolids to beneficial-use sites, represents a significant cost to overall POTW operations, transportation options must be evaluated thoroughly prior to both construction of wastewater treatment plants and implementation of a biosolids beneficial-use system. Although all the information in this chapter pertains to the transport of biosolids/sludge within and outside the POTW, much of the information is also applicable to the transport of screenings, grit, and scum [37].

5.1 Transportation of Biosolids/Sludges

Transportation of biosolids/sludges is accomplished primarily by the use of pumps, conveyors, or hauling equipment. Within a wastewater treatment plant, liquid biosolids/sludges must be transported during various operations including (1) primary and secondary wastewater treatment, (2) thickening operations, (3) stabilization processing, and (4) dewatering operations.

The transport of liquid biosolids/sludges within the wastewater treatment plant normally is accomplished through the use of pipelines under pressurized (i.e., pumped) or gravity flow conditions. In contrast, dewatered biosolids/sludges are transported within a wastewater treatment plant using conventional industrial solids conveyance equipment (e.g., carts, front-end loaders, belt conveyors, screw conveyors, etc.) [19,37].

When biosolids/sludges are to be transported from a wastewater treatment plant to another site for further processing and/or final disposal, potential modes of transportation include pipeline, truck, railroad, barge, or various combinations of these. Figure 5.1 illustrates the possible combinations of these transportation modes where land application of biosolids is the final disposal option. The transportation method(s) chosen and its inherent costs depend on several factors, including

- Quality and quantity of biosolids/sludge to be transported
- Transportation distance
- The flexibility in the transportation method chosen
- Environmental and public acceptance factors

The following sections examine the principal transport systems for biosolids/sludges within and external to a wastewater treatment plant.

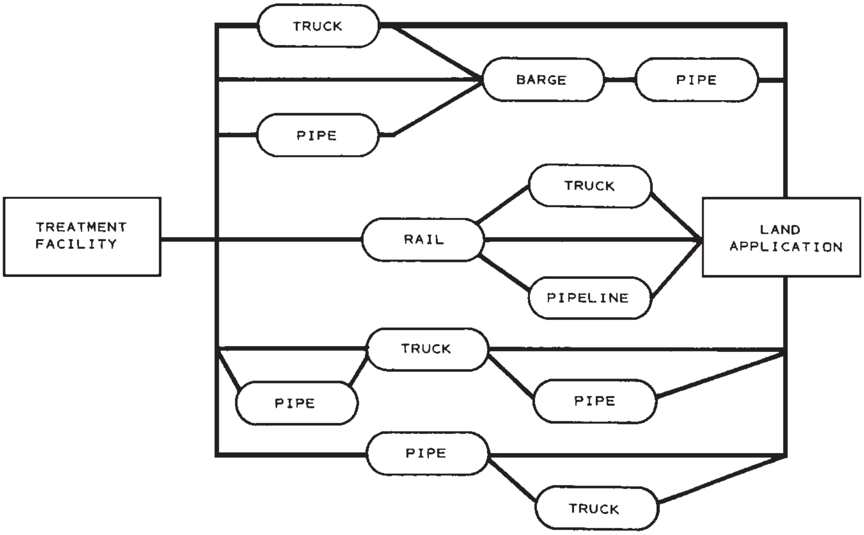


Figure 5.1 Combinations of various biosolids/sludge transport methods.

5.2 Pipeline Transport

Unless biosolids/sludges have been dewatered, they can be transported most efficiently and economically through pipelines. In many circumstances, the flow of liquid biosolids/sludge within a wastewater treatment facility is characterized by gravity flow. Under these conditions, the pipe may be flowing either full or partially full. In most situations characterized by gravity flow, the solids content is low (i.e., less than 5 percent solids), which allows the flow velocity and discharge rate to be estimated using standard head-loss equations suitable for describing water flow [15,27,37].

The flow velocity and discharge rate of biosolids/sludge under full-pipe gravity flow (e.g., flow of primary sludge to a thickener) may be estimated by equating the total available head (e.g., difference in elevation between the inlet and outlet hydraulic surfaces) to the head losses generated within the pipeline, including: (1) friction losses, (2) exit losses, and (3) minor losses [Eq. (5.1)].

$$\text{Total available head (ft, m)} = \text{friction losses} + \text{exit losses} + \text{minor losses} \qquad (5.1)$$

NOTE: All energy losses have units of feet (or meters) of water.

Assuming that the gravity flow is characterized by turbulent conditions within the pipe, friction losses h_f may be estimated by any of the standard empirical relationships used to estimate head-losses during water flow, i.e., the Darcy-Weisbach, Manning, and Hazen-Williams equations [Eqs. (5.2) through (5.8)].

Darcy-Weisbach equation: $$h_f = \frac{fLV^2}{D2g} \qquad (5.2)$$

where h_f = head loss, m (ft)
f = friction factor
L = length of pipe, m (ft)
V = mean velocity of flow, m/s (ft/s)
D = diameter of pipe, m (ft)
g = acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)

Equation (5.2) may be modified to explicitly account for the discharge rate Q by noting that the flow velocity V is equal to the discharge rate divided by the cross-sectional area of the pipe. Therefore, for a circular pipe, Eq. (5.2) may be rewritten as Eq. (5.3):

$$h_f = \frac{8fLQ^2}{\pi^2 gD^5} \qquad (5.3)$$

where Q is the discharge flow rate [m^3/s (ft^3/s)].

TABLE 5.1 Values of Coefficients to Use in the Friction-Loss Equations*

Manning Equation	
Pipe material	*n* value
Uncoated cast iron	0.013
Coated cast iron	0.012
Commercial wrought iron pipe, galvanized	0.014
Riveted and spiral steel pipe	0.015
Concrete pipe	0.013
Hazen-Williams Equation	
Pipe material	*C* value
Pipes extremely straight and smooth	140
Pipes very smooth	130
New riveted steel, vitrified clay	110
Old cast iron	100
Old riveted steel	95

*Adapted from ref. [27].

TABLE 5.2 Head-Loss Coefficients *K* for Estimating Minor Losses*

$$\text{Minor head losses} = K\left(\frac{V^2}{2g}\right)$$

Cause of minor loss	*K* value
90° elbow	0.7
45° elbow	0.6
Swing check valve (fully open)	0.6–2.5
Flow from storage tank	
Pipe projecting into tank	0.78
Pipe flush with tank wall	0.50
Slightly rounded entrance	0.23
Well-rounded entrance	0.04
Gate valve	
3/4 closed	24.0
1/2 closed	5.6
1/4 closed	1.2
Fully opened	0.2
Other valves	Depends on valve shape and degree of opening

*Adapted from refs. [14,27].

The pipe friction factor f has been found to vary with flow velocity, pipe roughness, pipe size, and other parameters. The relationship among these parameters is presented graphically in Moody diagrams [27]. If the absolute roughness ε is known or can be estimated, Eq. (5.4) may be used to estimate the friction factor under turbulent flow conditions.

$$1/\sqrt{f} = 2 \log (D/2\varepsilon) + 1.74 \tag{5.4}$$

where ε is the absolute pipe roughness (length per length of pipe wall; ref. [27]). All other terms are as previously defined.

Manning equation:

$$h_f = \frac{10.3n^2Q^2L}{D^{16/3}} \quad \text{(SI units)} \tag{5.5}$$

$$h_f = \frac{4.66n^2Q^2L}{D^{16/3}} \quad \text{(U.S. customary units)} \tag{5.6}$$

where n is the coefficient of roughness (Manning equation; Table 5.1). All other terms are as previously defined.

Hazen-Williams equation:

$$h_f = \frac{10.7Q^{1.85}L}{C^{1.85}D^{4.87}} \quad \text{(SI units)} \tag{5.7}$$

$$h_f = \frac{4.73Q^{1.85}L}{C^{1.85}D^{4.87}} \quad \text{(U.S. customary units)} \tag{5.8}$$

where C is the coefficient of roughness (Hazen-Williams equation; Table 5.1). All other terms are as previously defined.

Exit losses refer to the change in fluid velocity V that occurs during discharge of the biosolids/sludge flow. In most cases, gravity flow results in the discharge of the biosolids/sludge to an open or closed container. In this case, the final velocity of the fluid is zero, and the exit losses may be estimated as the loss of velocity head (i.e., $V^2/2g$).

Finally, *minor head losses* refer to the frictional energy losses associated with the movement of the biosolids/sludge through bends, valves, or other appurtenances within the pipe. The minor head losses are a function of the velocity head (i.e., $V^2/2g$) and can be approximated by multiplying the velocity head by an appropriate head-loss coefficient. Head-loss coefficients for various appurtenances are provided in Table 5.2. Example 5.1 illustrates the approach for estimating the velocity and discharge rate for biosolids/sludge gravity full-pipe flow.

Example 5.1 The Garvey County Water Reclamation Plant is presently operating its aerobic digester to accomplish both sludge stabilization and thickening. Once the solids blanket reaches a prescribed depth, the settled solids are discharged by gravity through 600 ft of 10-in-diameter (0.833-ft-diameter) steel pipe to a sludge drying bed. Estimate the velocity V and discharge rate Q of the sludge if the hydraulic surface of the aerobic digester is 30 ft above the outfall of the discharge pipe and the pipeline contains two 45° elbows and one gate valve that is one-quarter closed when discharging. Assume that the Darcy-Weisbach friction factor is 0.02 and that the minor loss coefficients for the elbows and gate valve are 0.6 and 1.2, respectively.

solution

Step 1. Use Eq. (5.1) and the Darcy-Weisbach equation [Eq. (5.2)] to solve for the discharge velocity V.

Total available head, ft (m) = friction losses + exit losses + minor losses

$$30\text{ ft} = \frac{fLV^2}{D2g} + \frac{V^2}{2g} + 2\left(K_{\text{elbow}}\frac{V^2}{2g}\right) + K_{\text{gate valve}}\frac{V^2}{2g}$$

$$30\text{ ft} = \frac{V^2}{2g}\left(\frac{fL}{D} + 1 + 2K_{\text{elbow}} + K_{\text{gate valve}}\right)$$

$$30\text{ ft} = \frac{V^2}{2\,(32.2\text{ ft/s}^2)}\left[\frac{0.02 \cdot 600\text{ ft}}{0.833\text{ ft}} + 1 + 2\,(0.6) + 1.2\right]$$

Solving for velocity V gives

$$V = 10.42\text{ ft/s}$$

Step 2. Estimate the discharge rate Q by multiplying the flow velocity V by the cross-sectional area of the pipe.

$$\begin{aligned} Q &= \text{area} \cdot \text{velocity} \\ &= \frac{\pi\,(0.833)^2}{4}\,10.42\text{ ft/s} \\ &= 5.68\text{ ft}^3\text{/s (2549 gal/min)} \end{aligned}$$

NOTE: 1 ft^3 = 7.48 gal.

In the case of pipes flowing partially full, the relationship between fluid velocity and discharge rate must be determined through the use of empirical graphs or tables [27]. The basic approach involves solving for these flow parameters as a function of the relative depth of pipe flow and pipe slope. Examples of this approach are illustrated in the analysis of drainage systems for biosolids land application systems (Chap. 7).

When biosolids/sludges are to be transported from lower to higher elevations within the wastewater treatment plant, a pumping system must be designed. In addition to estimating the elevation difference, the frictional losses must be estimated in order to determine the minimum pumping requirements necessary to maintain a specific discharge rate. With the aid of pumping, biosolids/sludges can be transported through short pipelines at solids contents of up to 20 percent as well as through pipelines of distances of over 10 mi (16 km) at solids contents as high as 8 percent [37].

Empirical approaches have been developed to estimate head-losses in biosolids/sludge pumping systems. The accuracy of these approaches is normally adequate for pipeline design, particularly when the biosolids/sludge solids content is below 3 percent solids (i.e., 30,000 mg/liter). However, as the pipe length, percentage total solids, or percentage volatile solids increases, the accuracy of these approaches decreases.

The simplest approach for estimating the frictional losses during pressurized biosolids/sludge flow is to multiply the friction loss estimated from the water flow head-loss equations (h_f) by a factor specific for the type of biosolids/sludge being pumped. Multiplication factors suitable for pumping biosolids/sludges at solid contents as high as 10 percent are provided in Fig. 5.2. The following factors must be considered before using Fig. 5.2 in pipeline design:

1. Flow velocities should be between 2.5 ft/s (0.8 m/s) and 8.0 ft/s (2.4 m/s).
2. Frictional losses associated with the suction side of the pipe, and transient startup conditions are not accounted for in this approach and therefore must be evaluated independently.
3. Grease or other materials do not obstruct pipe flow.

Example 5.2 illustrates the use of Fig. 5.2 in estimating the head-loss in biosolids/sludge pumping systems.

Example 5.2 Allen County Wastewater Treatment Plant periodically discharges the entire contents of one of its anaerobic digesters to sludge-holding lagoons through 1500 ft of 6-in-diameter steel pipe. Determine the head loss generated in the pipeline if the digester volume is 150,000 gal and the contents must be pumped to the sludge lagoons over one 8-hour shift. Assume that the anaerobic digester sludge has a solids content of 7 percent and that a Hazen-Williams coefficient of 110 is suitable to describe the quality of the pipe.

solution

Step 1. Estimate the volumetric flow rate of sludge in cubic feet per second (ft^3/s):

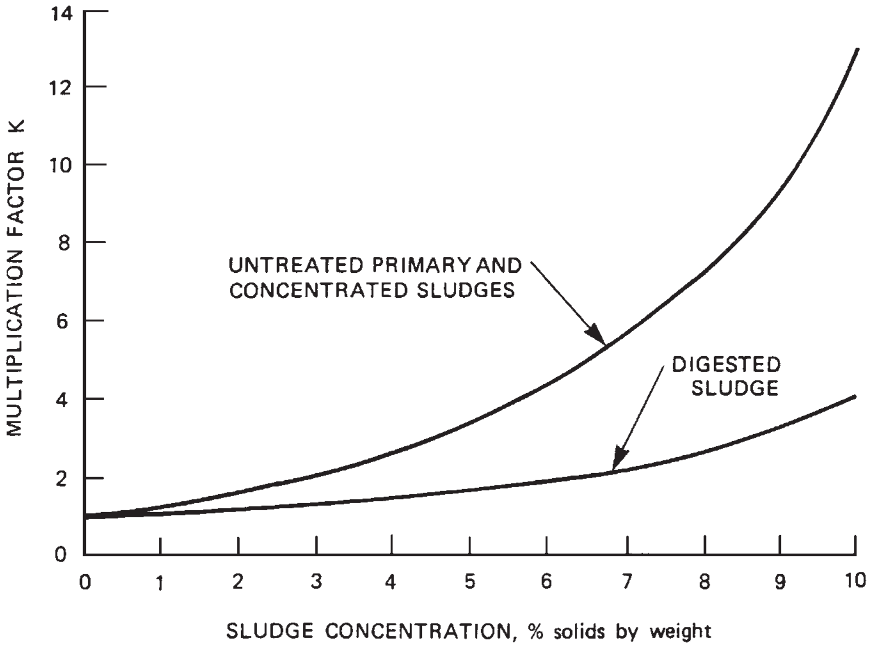


Figure 5.2 Multiplication factors *K* for predicting biosolids/sludge head losses.

$$\text{Volumetric flow rate } Q\ (\text{ft}^3/\text{s}) = 150{,}000 \text{ gal}/8 \text{ h} \cdot \text{ft}^3/7.48 \text{ gal} \cdot \text{h}/3600 \text{ s}$$

$$= 0.70 \text{ ft}^3/\text{s}$$

Step 2. Using the Hazen-Williams equation [Eq. (5.8)], estimate the head loss based on water flow:

$$h_f = \frac{4.73Q^{1.85}L}{C^{1.85}D^{4.87}}$$

$$= \frac{4.73\ (0.7 \text{ ft}^3/\text{s})^{1.85} 1500 \text{ ft}}{(110)^{1.85}\ (0.5 \text{ ft})^{4.87}}$$

$$= 17.9 \text{ ft (of } H_2O)$$

Step 3. From Fig. 5.2, a digested sludge having a solids content of 7 percent has a *K* factor of approximately 2. Therefore, the estimated head loss generated in the pumping of the digested sludge is approximately 35.8 ft of H_2O (i.e., 17.9 ft multiplied by 2), or 15.5 lb/in^2.

NOTE: 1 lb/in^2 = 2.31 ft of H_2O pressure.

Implicit in this simplified approach to estimating head-loss is the assumption that the biosolids/sludge pipe flow is turbulent. Under these conditions, the behavior of biosolids/sludge flow is similar to that of water flow. However, unlike pressurized water flow, laminar flow is common in pressurized biosolids/sludge flow (particularly at high solids concentrations). Because of its dependency on viscosity forces, the head-losses generated within biosolids/sludge pipelines are significantly greater than those which occur during water flow under laminar flow conditions [15].

Further complicating the estimation of head-losses generated during biosolids/sludge laminar flow is the fact that while water is a newtonian fluid, biosolids/sludges are nonnewtonian. From theoretical considerations, the head-loss generated during the pumping of a newtonian fluid is directly proportional to the flow velocity and specific fluid properties and may be estimated by application of the Pouseille equation [Eq. (5.9)]. Moreover, the fluid properties (e.g., viscosity, density, etc.) of a newtonian fluid are only functions of temperature and are independent of flow characteristics.

$$h_f = \frac{32\mu LV}{\rho g D^2} \tag{5.9}$$

where h_f = head (i.e., energy) loss, m (ft)
L = length of pipe, m (ft)
V = mean velocity of flow, m/s (ft/s)
μ = dynamic viscosity, $\mathrm{N \cdot s/m^2}$ ($\mathrm{lb_f \cdot s/ft^2}$)
ρ = fluid density, $\mathrm{kg/m^3}$ ($\mathrm{slug/ft^3}$)
D = diameter of pipe, m (ft)
g = acceleration due to gravity, 9.81 $\mathrm{m/s^2}$ (32.2 $\mathrm{ft/s^2}$)

In addition to the simplified approach for estimating head-loss during the transport of newtonian fluids, the flow velocity that characterizes the transition from laminar to turbulent flow is characterized by the Reynolds number, which may be estimated using Eq. (5.10) [15,37]:

$$\mathrm{Re} = \frac{\rho VD}{\mu} \tag{5.10}$$

where Re = Reynolds number (dimensionless)
ρ = density of biosolids/sludge, $\mathrm{slug/ft^3}$ ($\mathrm{kg/m^3}$)
V = average velocity ft/s (m/s)
D = diameter of pipe, ft (m)
μ = dynamic viscosity, $\mathrm{lb_f \cdot s/ft^2}$ ($\mathrm{N \cdot s/m^2}$)

In contrast to water flow under laminar flow conditions, the head-loss h_f generated during biosolids/sludge flow neither is proportional to fluid velocity nor is the onset of turbulent flow conditions predictable from knowledge of a characteristic Reynolds number. Moreover, as the duration and/or intensity of biosolids/sludge pumping increases, there is a gradual reduction in fluid viscosity. Once pumping is stopped, the fluid viscosity is recovered. This variation in fluid viscosity as a function of pumping time and/or intensity is a characteristic of thixotropic fluids [15,22,37]. The effects of thixotropic behavior on biosolids/sludge pumping systems are summarized as follows:

1. No flow occurs unless the generated pressure is sufficient to exceed a specific yield stress τ_0.
2. The onset of turbulent conditions will occur at a much higher flow velocity than what is required for water flow.
3. In fully developed turbulent flow, the head-losses generated in biosolids/sludge flow are approximately the same as for water.
4. In the laminar flow region, biosolids/sludge flow behaves as a Bingham plastic and is characterized by a yield stress τ_0 and a coefficient of rigidity η.

Biosolids/sludge behave like a Bingham plastic, a material with a linear relationship between shear stress and shear rate, only after flow begins. A Bingham plastic is described by two constants: (1) yield stress τ_0 and (2) coefficient of rigidity η. Typical ranges of yield stress and coefficient of rigidity for various biosolids/sludge solids contents are provided in Table 5.3. It should be noted that the range of these parameters is highly variable at constant solids contents, and, therefore, laboratory analysis should be conducted to achieve specific values.

To estimate the head-loss generated during the pipeline transport of biosolids/sludge flow under laminar conditions, Fig. 5.3 may be used to estimate an appropriate friction factor *f*. With respect to biosolids/sludge flow under laminar flow conditions, the friction factor *f* is a function of two dimensionless numbers, i.e., Reynolds (Re) and Hedstrom (He) numbers, both of which are defined by Eqs. (5.11) and (5.12), respectively.

$$\mathrm{Re} = \frac{\gamma V D}{\eta} \qquad \text{(U.S. customary units)} \qquad (5.11a)$$

TABLE 5.3 Yield Stress τ_0 and Coefficient of Rigidity η for Biosolids/Sludge*

Solids content	Yield stress τ_0 (lb/ft^2)	Coefficient of rigidity η (lb/ft·s)
2	0.018–0.035	0.0025–0.0053
4	0.085–0.170	0.0071–0.0150
6	0.110–0.220	0.0140–0.0280
8	0.014–0.260	0.0200–0.0420
10	0.016–0.340	0.0280–0.0580

*Adapted from ref. [37].

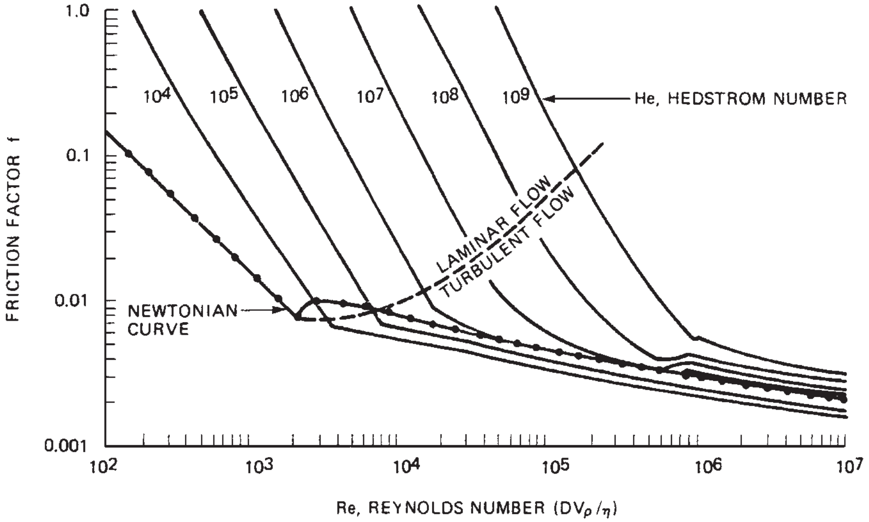


Figure 5.3 Friction factor for biosolids/sludge flow analyzed as a Bingham plastic.

$$\mathrm{Re} = \frac{\rho VD}{\eta} \qquad \text{(SI units)} \tag{5.11b}$$

where Re = Reynolds number (dimensionless)
ρ = fluid density, slug/ft^3 (kg/m^2)
γ = specific weight, lb/ft^3 (kN/m^3)
V = mean velocity of flow, ft/s (m/s)
D = diameter of pipe, ft (m)
η = coefficient of rigidity, lb$_m$/ft · s (kg/m · s)

$$\mathrm{He} = \frac{D^2\tau_0 g\gamma}{\eta^2} \qquad \text{(U.S. customary units)} \tag{5.12a}$$

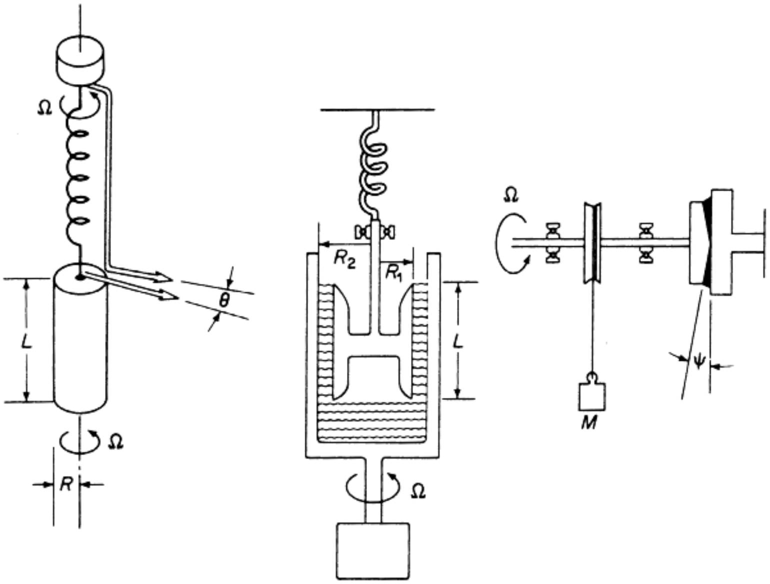
$$\mathrm{He} = \frac{D^2\tau_0\rho}{\eta^2} \qquad \text{(SI units)} \tag{5.12b}$$

where He = Hedstrom number (dimensionless)
τ_0 = yield stress, lb$_f$/ft^2 (N/m^2)
γ = specific weight, lb/ft^3 (kN/m^3)
η = coefficient of rigidity (lb$_m$/ft · s) (kg/m · s)
g = acceleration due to gravity, 32.2 ft/s^2
D = diameter of pipe, ft (m)

It is critical to note that while the Reynolds number for newtonian fluid flow [Eq. (5.10)] is based on viscosity, the Reynolds number for nonnewtonian biosolids/sludge flow is a function of the coefficient of rigidity η. The reason for this is that the viscosity in biosolids/sludge flow is highly variable, and therefore, Eq. (5.10) is not adequate to characterize flow behavior. In addition to the modification of the Reynolds number equation, the friction factor estimated from Fig. 5.3 will be significantly less than the friction factor f estimated by the use of Moody diagrams for standard water flow [27].

To estimate Re and He for biosolids/sludge flow, the yield stress τ_0 and the coefficient of rigidity η must be known or estimated. Although a reasonable amount of data are available for estimating the yield stress τ_0 and the coefficient of rigidity η for anaerobically digested sludges, little data exist for other types of biosolids/sludges [37]. Without literature values, the best approach to obtain these constants is to measure them in the laboratory from samples of biosolids/sludge in a rotational viscometer. Schematic diagrams of various rotational viscometer designs are provided in Fig. 5.4.

Of the three major types of rotational viscometer designs, use of the coaxial cylinder system is the most suitable for biosolids/sludge analysis. In this system, the biosolids/sludge sample is placed between two concentric cylinders, one of which rotates. The torque (or shear stress) on the cylinder is measured as a function of rota-



(a) (b) (c)

Figure 5.4 Rotational viscometers: (a) long cylinder in infinite volume of fluid; (b) coaxial cylinder (cup and bob); (c) cone plate. (*Adapted by permission from ref.* [22]).

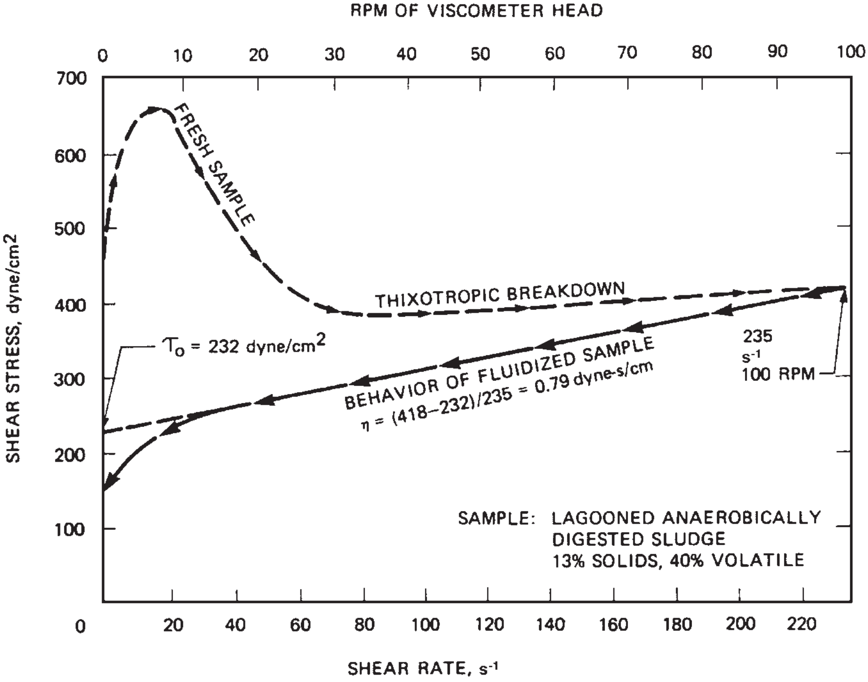


Figure 5.5 Rotational viscometer test results for biosolids and sludge [22].

tional speed that simulates the shearing rate that occurs during biosolids/sludge pumping. Rotational viscometer testing allows the development of a shear stress–shear rate curve that can be used to estimate the yield stress τ_0 and the coefficient of rigidity η. Representative rotational viscometer data for a lagooned anaerobically digested sludge are depicted in Fig. 5.5.

In the viscometer test shown in Fig. 5.5, the rotational speed was increased gradually from zero to 100 revolutions per minute (rpm) and then decreased slowly. The torque on the inner cylinder was measured and converted to shear stress (dyne/cm^2). The upper curve illustrates the loss in shear stress with increasing shear rate (i.e., rotational speed), which is characteristic of thixotropic fluids. The behavior of the lower curve (decreasing rotational speed) reflects the flow characteristics after the thixotropic effects of pipeline startup have been overcome.

For pipeline design, the results from the lower curve in Fig. 5.5 should be employed because passage through the pump effectively fluidizes the biosolids/sludge. The yield stress τ_0 from the viscometer test is estimated as the shear stress that occurs at a zero shear rate (i.e., 232 dyne/cm^2), while the coefficient of rigidity η is the slope of the straight portion of the lower curve (i.e., 0.79 dyne · s/cm).

Once the Reynolds (Re) and Hedstrom (He) numbers are calculated using Eqs. (5.11) and (5.12), the friction factor f may be estimated from Fig. 5.3. Using the friction factor, the frictional head-loss (i.e., pressure drop) across the length of pipe may be estimated using Eq. (5.13) or Eq. (5.14):

$$\Delta P = \frac{2f\gamma LV^2}{Dg} \qquad \text{(U.S. customary units)} \qquad (5.13a)$$

$$\Delta P = \frac{2f\rho LV^2}{g} \qquad \text{(SI units)} \qquad (5.13b)$$

where ΔP = pressure drop (head loss) in pipe, lb_f/ft^2 (N/m^2)
f = friction factor (from Fig. 5.3)
L = length of pipe, ft (m)
ρ = fluid density, slug/ft^3 (kg/m^2)
γ = specific weight, lb/ft^3 (kN/m^3)
V = mean velocity of flow, ft/s (m/s)
D = diameter of pipe, ft (m)
g = acceleration due to gravity, 32.2 ft/s^2

It should be noted that Eqs. (5.11) through (5.13) apply to the entire range of biosolids/sludge flow from zero flow velocity to the fully turbulent range [37]. Example 5.3 illustrates the use of these equations and Fig. 5.3 in predicting the head-loss in biosolids/sludge pipeline systems.

Example 5.3 A digested sludge having a solids content of 6 percent is to be pumped through 5000 ft of 6-in steel pipe. From viscometer tests, the digested sludge has an approximate yield stress τ_0 of 0.025 lb_f/ft^2 and a coefficient of rigidity η of 0.022 lb/ft·s. Estimate the head-loss generated in the pipe if the pumping rate is to be maintained at 0.5 ft^3/s. Assume that the specific weight of sludge is approximately the same as that for water at 68°F (20°C) (i.e., 62.4 lb/ft^3).

solution

Step 1. Estimate the flow velocity V from knowledge of the discharge rate and pipe diameter:

$$V\ (\text{ft/s}) = \frac{\text{discharge rate (ft}^3\text{/s)}}{\text{cross-sectional area of pipe (ft}^2)}$$

$$= \frac{0.5\ \text{ft}^3/\text{s}}{\pi\ (\text{diameter})^2/4} = \frac{0.5\ \text{ft}^3/\text{s}}{\pi\ (0.5\ \text{ft})^2/4}$$

$$= 2.55\ \text{ft/s}$$

Step 2. Estimate the Reynolds (Re) and Hedstrom (He) numbers from the viscometer data:

$$\mathrm{Re} = \frac{\gamma VD}{\eta}$$

$$= \frac{62.4\ \mathrm{lb/ft^3} \cdot 2.55\ \mathrm{ft/s} \cdot 0.5\ \mathrm{ft}}{0.022\ \mathrm{lb/ft \cdot s}} = 3.6 \cdot 10^3\ \text{(dimensionless)}$$

$$\mathrm{He} = \frac{D^2 \tau_0 g \gamma}{\eta^2}$$

$$= \frac{(0.5\ \mathrm{ft})^2 \cdot 0.025\ \mathrm{lb/ft^2} \cdot 32.2\ \mathrm{ft/s^2} \cdot 62.4\ \mathrm{lb/ft^3}}{(0.022\ \mathrm{lb/ft \cdot s})^2}$$

$$= 2.6 \cdot 10^4\ \text{(dimensionless)}$$

Step 3. Using the calculated values for Re, He, and Fig. 5.3, estimate the friction factor *f*. From Fig. 5.3, the flow behavior is near the transition region with a friction factor *f* of approximately 0.08.

Step 4. Estimate the head loss generated during sludge pumping using Eq. (5.13*a*):

$$\Delta P = \frac{2f\gamma LV^2}{Dg}$$

$$= \frac{2 \cdot 0.08 \cdot 62.4\ \mathrm{lb/ft^3} \cdot 5000\ \mathrm{ft} \cdot 2.55\ \mathrm{ft/s}}{(0.5\ \mathrm{ft}) \cdot 32.2\ \mathrm{ft/s^2}}$$

$$= 7907\ \mathrm{lb_f/ft^2}\ (54.9\ \mathrm{psi, or}\ 126.8\ \mathrm{ft\ of}\ H_2O)$$

NOTE: 1 psi = 2.31 ft of H_2O, 1 psi = 144 $\mathrm{lb_f/ft^2}$.

One caution in the use of Fig. 5.3 in estimating the friction factor *f* is that the functional relationships depicted in this figure ignore pipe roughness. If the head-loss equations for water flow (e.g., Darcy-Weisbach, Manning, or Hazen-Williams equations) predict a higher frictional energy loss than that generated by employing the friction factor obtained from Fig. 5.3, pipe roughness is dominant, and the biosolids/sludge flow is fully turbulent. Under these circumstances, the head-loss for the biosolids/sludge flow is approximately the same as in water flow. To ensure uninterrupted pumping of biosolids/sludge, a safety factor on the order of 1.5 is recommended for estimating the maximum frictional head-loss during normal operating conditions [37].

5.2.1 General pipeline design guidance

Proper preplanning of a pipeline is critical for successful biosolids/-sludge transport operations. In addition to pump malfunctions or a plugged pipeline, the sequence of unit operations will have a significant impact on the effectiveness of pipeline transport. For example, the system design engineer may have to decide whether or not to place the sludge digesters before or after the pipeline. Since experience has shown that reduction in volatile solids significantly reduces head-loss, the pumping of digested sludge is preferred to the pumping of raw or undigested sludge.

In addition to the concerns associated with increased head-loss, the pumping of raw sludges may cause additional transport problems related to thickening, odors, and corrosion at the receiving point due to septic conditions developed in the pipeline. If it is necessary to pump raw sludge long distances, the environmental impact of this operation will be reduced if the pipeline contents are discharged directly into anaerobic digesters [37].

The impact of thixotropy on the required startup pressures for any long-distance biosolids/sludge pipelines, as well as the permanent loss of yield stress τ_0 that can occur with increasing duration and/or intensity of pumping, must be accounted for in the pipeline design. Permanent loss in yield stress occurs in biosolids/sludge flow when pumping imparts a sufficient amount of fluid shear to physically disrupt the floc particles. If this occurs, it may be difficult to later thicken or dewater the biosolids/sludges [27,28,37].

Finally, the following general design guidance should be followed when planning biosolids/sludge pipeline transport systems:

1. Provide two pipes unless a single pipe can be shut down for several days without causing problems in the wastewater treatment system.
2. Consider external corrosion protection and pipe loads during physical construction of the pipeline. Electrical return currents, acid soils, saline groundwater, and other factors may cause serious difficulty unless special corrosion-control measures are used (e.g., cathodic protection).
3. Provide an efficient mechanism for adding controlled amounts of water to either dilute the biosolids/sludge or to flush the line.
4. Provide an effective means for inserting and retrieving a pipeline cleaning tool, if needed.
5. The pipeline route should be chosen for ease of maintenance.
6. At points of high elevation in the pipeline, air or gas relief values should be provided. Note that in addition to causing odors, air and gases from biosolids/sludge pipelines may be toxic, flammable, explosive, and corrosive.
7. If the biosolids/sludge flow is to be pumped at solids contents of more than 3 percent, the pumps and pipeline should be designed for high and variable friction head-losses.

8. If centrifugal pumps are used, flow rates may be variable because of the varying flow resistance properties of the biosolids/sludge. To minimize interuptions in biosolids/sludge transport, appropriately sized storage facilities should be an integral component of the pipeline design.
9. Positive-displacement pumps may experience excessive pressures during startup of a long biosolids/sludge pipeline. To minimize this problem, variable-speed drives should be provided, and the pumps should be started at low speeds.
10. For long pipelines, a booster pumping station may be required. Because of the difficulty in matching booster pumping rates to the biosolids/sludge flow, adequately sized holding tanks should be provided.
11. The adverse impacts of waterhammer are best controlled by limiting the flow velocity to no more than 3 ft/s (0.9 m/s).

5.2.1.1 Pipeline materials. Pipelined constructed of various materials may be used for transporting biosolids/sludge. Typical pipeline materials include steel, cast iron, concrete, fiberglass, and polyvinyl chloride (PVC). For a long-distance high-pressure biosolids/sludge pipeline, steel pipe is the most common construction material [1,14,19,37].

One of the major concerns in pipeline design is pipe stress and the potential for pipe rupture [19]. In addition to stresses caused by fluid pressure, temperature fluctuations may cause stress in pipe. Temperature fluctuations occur during flushing and/or the use of hot pressurized water for grease removal. Biosolids/sludge pipelines should be constructed from materials designed to accommodate such stresses.

In addition to pipe stress, corrosion can be a significant problem for biosolids/sludge pipelines unless properly considered during system design. External corrosion is a function of the pipeline material and can be controlled by applying a suitable pipe coating and/or providing cathodic protection [37]. Laboratory tests have indicated that with proper design, only moderate internal corrosion rates should be expected in long-distance biosolids/sludge pipelines [19]. Moreover, if most of the grit and other abrasive materials are removed from the digested biosolids/sludge, internal wear due to friction is not a significant factor in biosolids/sludge pipelines.

5.2.1.2 Pipe sizes. Conventional biosolids/sludge piping should not be smaller than 6 in (150 mm) in diameter. Pipe sizes need not be larger than 8 in (200 mm) unless the flow velocity exceeds 5 ft/s (1.5 m/s), in which case the pipe is sized to maintain a reduced velocity. Gravity-flow piping should not be less than 8 in (200 mm) in diameter. It is common practice to install a number of cleanouts in the form of plugged tees or crosses instead of elbows so that the lines can be rodded, if necessary [19]. Pump connections should not be smaller than 4 in (100 mm) in diameter.

Grease has a tendency to coat the inside of piping used in transporting primary sludge. Grease accumulation results in a decrease in the effective diameter of the pipe and a significant increase in the required discharge pressure. For this reason, low-capacity positive-displacement pumps are designed for discharge pressures much greater than the theoretical estimate when used in long-distance biosolids/sludge pipelines. Centrifugal pumps, with their larger capacity, are used to pump a more dilute sludge over shorter distances. In some wastewater treatment plants, provisions have been made for melting the grease by circulating hot water or steam through the pipeline [37].

5.2.2 Biosolids/sludge pumps

Biosolids/sludges can range in consistency from a watery scum to a thick pastelike slurry. Because of the range of biosolids/sludge fluid properties, choosing an appropriate pump for a specific application should be done carefully. Important operational characteristics of any pump are (1) pump horsepower and (2) pump efficiency. The *pump horsepower* is the energy transferred from the pump to the fluid and is estimated as the product of the discharge flow rate and the pump pressure (or total head). The pump horsepower may be estimated by employing Eq. (5.14):

$$\text{Pump horsepower (hp)} = \frac{\gamma Q H_p}{550 \text{ ft} \cdot \text{lb}_f/\text{s} \cdot \text{hp}} \tag{5.14}$$

where γ = specific gravity of biosolids/sludge
Q = discharge flow rate, ft^3/s
H_p = total head delivered by pump, ft (elevation head plus frictional losses)
550 = conversion of $\text{ft} \cdot \text{lb}_f/\text{s}$ to hp

The *efficiency* of a pump is a measure of its hydraulic performance relative to the energy input applied to the pump shaft by the motor (i.e., brake horsepower). The pump efficiency e is defined by Eq. (5.15). Example 5.4 illustrates the use of Eqs. (5.13) and (5.14) in estimating pump efficiency.

$$\text{Pump efficiency } e = \frac{\text{pump horsepower (hp)}}{\text{brake horsepower (hp)}} \cdot 100 \tag{5.15}$$

where brake horsepower = energy delivered by the motor to the pump.

Example 5.4 An electric motor imparts 15 hp of mechanical energy to a centrifugal pump (i.e., brake horsepower) that is maintaining a discharge rate of 0.5 ft^3/s of sludge through a 1600-ft steel pipe. Estimate the efficiency of the pump if the frictional head losses have been estimated to be 45 ft of H_2O at this flow rate and the maximum elevation difference occurring within the pipeline is 130 ft. Assume that the specific weight of biosolids/sludge is 62.4 lb/ft^3.

solution

Step 1. Estimate the total pressure head generated by the pump H_p:

$$\begin{aligned} H_p &= \text{frictional losses} + \text{elevation difference} \\ &= 45 \text{ ft} + 130 \text{ ft} \\ &= 175 \text{ ft} \end{aligned}$$

Step 2. Estimate the pump horsepower using Eq. (5.14):

$$\begin{aligned} \text{Pump horsepower (hp)} &= \frac{\gamma Q H_p}{550 \text{ ft} \cdot \text{lb}_f/\text{s} \cdot \text{hp}} \\ &= \frac{62.4 \text{ lb/ft}^3 \cdot 0.5 \text{ ft}^3/\text{s} \cdot 175 \text{ ft}}{550 \text{ ft} \cdot \text{lb}_f/\text{s} \cdot \text{hp}} \\ &= 9.93 \text{ hp} \end{aligned}$$

Step 3. Estimate the pump efficiency using Eq. (5.15):

$$\begin{aligned} \text{Pump efficiency } e &= \frac{\text{pump horsepower (hp)}}{\text{brake horsepower (hp)}} \cdot 100 \\ &= \frac{9.93 \text{ hp}}{15.0 \text{ hp}} \cdot 100 = 66 \text{ percent} \end{aligned}$$

A critical specification in choosing a biosolids/sludge pump is the required net positive suction head (NPSH). The NPSH refers to the maximum depth from which the biosolids/sludge can be drawn from the suction side of the pump. In addition to altitude and barometric pressure, the NPSH for any pump will be a function of the frictional losses associated with the suction side of the pump [19,27]. Excessively long suction pipelines can result in a suction pressure that is close to the vapor pressure of water, causing cavitation to occur in the pump. Cavitation not only reduces pump capacity but also results in erosion of the internal components of the pump [27].

Due to fluid thixotropy and the increased head-losses associated with biosolids/sludge flow, the required NPSH in biosolids/sludge pump systems is significantly less than in water flow. Ideally, the biosolids/sludge pump should be several feet below the liquid level of the tank from which the biosolids/sludges are being pumped (i.e., positive suction pressure).

Pumps that are currently used in biosolids/sludge pipelines include (1) centrifugal, (2) torque flow, (3) plunger, (4) piston, (5) progressive cavity, (6) diaphragm, (7) rotary, (8) ejector, and (9) gas lift. Each of these pump designs is described briefly in the following subsections.

5.2.2.1 Centrifugal pumps. A centrifugal pump generates pressure by transferring energy to a fluid using a rotating impeller within a confined circular casing. Depending on the application, centrifugal pumps can be designed with features for maximum pumping performance. For example, a common design specification for centrifugal pumps is the number of stages. A single-stage centrifugal pump is equipped with one impeller, whereas a multistage pump consists of two or more impellers within a single casing. The impeller consists of a number of vanes that force the fluid to the outside of the casing by centrifugal force (Fig. 5.6*a*). The principal advantages cited for centrifugal pumps in transporting biosolids/sludges include (1) relatively smooth and even flow and (2) easily controlled discharge pressure and flow rate [1,14].

To minimize clogging, centrifugal pumps used for biosolids/sludge transport have fewer but larger and less obstructed vane passageways in the impeller compared with pumps used to transport clean water. A variation of the standard centrifugal pump design is the nonclog centrifugal pumps, which commonly are used to transport biosolids/sludge (see Fig. 5.6*b*). The larger passageways and greater clearances in nonclog centrifugal pumps result in increased reliability at a cost of lower pump efficiency [37].

Although throttling of the discharge valve to control the discharge pressure is possible in centrifugal pumps, it is not a recommended practice because it reduces energy efficiency while increasing the potential for permanent damage to the discharge valve. Therefore, it is essential that centrifugal pumps requiring discharge pressure (or capacity) adjustment are equipped with variable-speed drives. Maintaining constant discharge flow rates in multiple-centrifugal-pump systems may be achieved by equipping each pump with a discharge flowmeter and using the flowmeter signal in conjunction with the variable-speed drive to control the speed of the pump [1,19].

A major challenge in employing centrifugal pumps for biosolids/sludge flow is choosing the correct size. At any given speed, centrifugal pumps operate most efficiently within a narrow range of discharge pressures. Because of the variability in biosolids/sludge consistency, discharge pressures can change drastically. To determine the proper operating speeds and motor power required for centrifugal pumps, characteristic system-head versus capacity curves should be compared for (1) the highest density of fluid anticipated, (2) average conditions, and (3) water. These system curves should be plotted on a graph of the pump curves for a range of available speeds. The maxi-

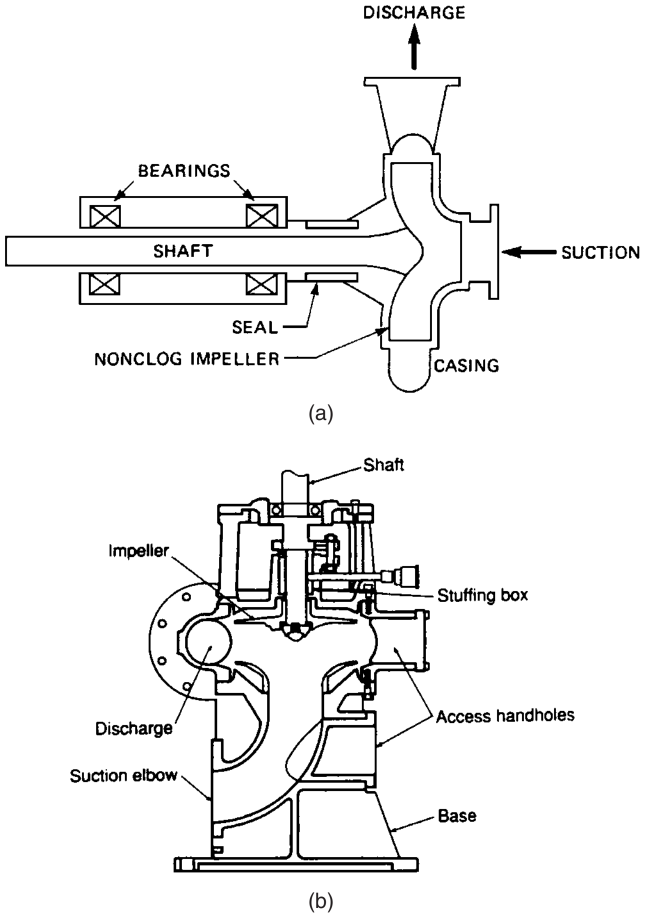


Figure 5.6 Schematic diagrams of centrifugal pumps: (*a*) standard design; (*b*) nonclog design.

mum and minimum speeds of a particular pump are obtained from the intersection of the pump curves with the system curves at the desired capacity. Where the maximum speed head-capacity curve intersects the system curve for water determines the power required, whereas the intersection of the pump curves with the system curve for average conditions is used to estimate the hours of operation, average speeds, and power costs [27].

Propeller or mixed-flow centrifugal pumps are sometimes used for low-pressure applications (e.g., return activated sludge) because of higher pump efficiencies [14]. When being considered for this type of application, such pumps must be of sufficient size to provide internal clearances capable of passing the types of solids normally found within the activated-sludge system. Such pumps should not be used

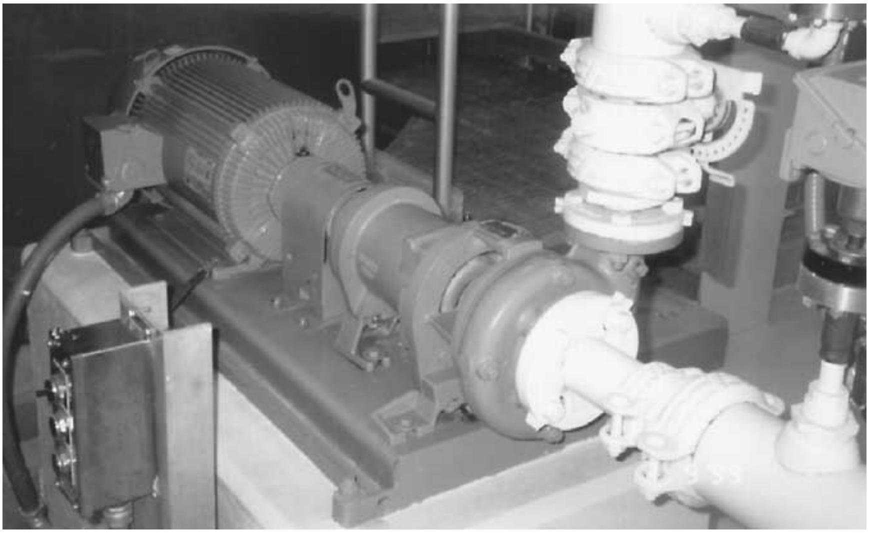


Figure 5.7 Photograph of a centrifugal pump.

in activated-sludge systems that are not preceded by primary sedimentation facilities.

5.2.2.2 Torque-flow pumps. A torque-flow pump, also known as a *recessed impeller* or *vortex pump,* is a centrifugal pump in which the impeller is open faced and recessed well back into the pump casing. The rotating impeller develops a vortex in the flow so that the main propulsive force is the fluid itself (Fig. 5.8*a*). The size of particles that can be handled by this type of pump is limited only by the diameter of the suction or discharge openings.

Because there are no close tolerances between the impeller and the casing, abrasive wear is reduced. The tradeoff for increased pump longevity and reliability is that the torque-flow pump is relatively inefficient (e.g., *ca.* 45 percent pump efficiency is typical versus 65 percent efficiency for centrifugal pumps). Torque-flow pumps can operate only over a narrow discharge pressure range for a given speed, so the operating conditions must be evaluated carefully. Variable-speed control is recommended where these pumps are expected to operate over a wide range of discharge pressure conditions.

5.2.2.3 Plunger pumps. Unlike impeller-type pumps, the plunger pump is an example of a positive-displacement pump. These types of pumps are designed to generate pressure by forcing the fluid out of the pump under the motion of a solid body displacing the fluid. In the case

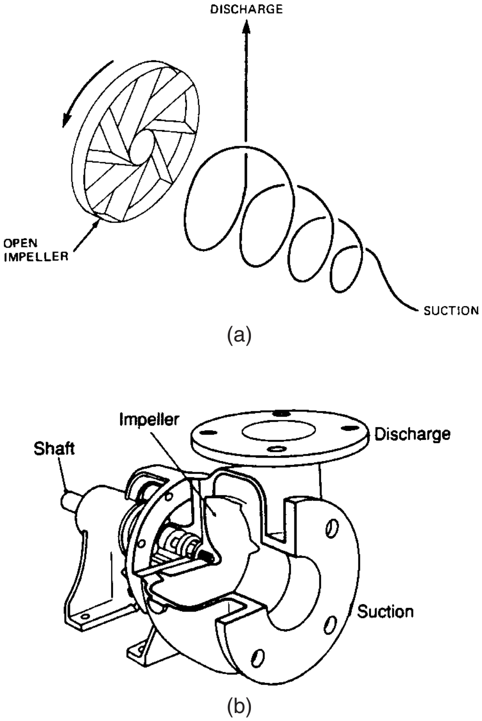


Figure 5.8 (*a*) Flow pattern in torque-flow pumps. (*b*) Schematic diagram of a torque-flow pump.

Figure 5.9 Photograph of a torque-flow pump.

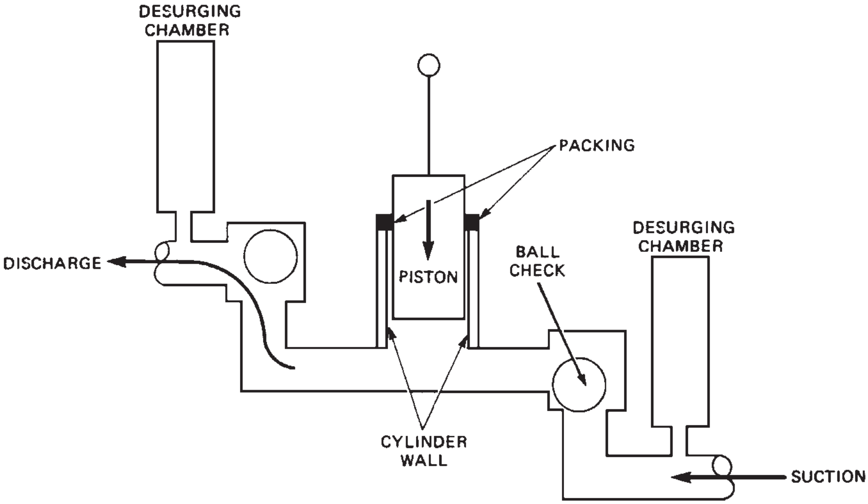


Figure 5.10 Schematic diagram of a plunger pump.

of plunger pumps, the solid body consists of a piston driven by an exposed drive crank (Fig. 5.10). Plunger pumps have constant capacity regardless of variations in head-loss and can handle biosolids/sludge with solids contents up to 15 percent [37].

Shortening the stroke of the plunger can be used to decrease the discharge flow rates in plunger pumps. Since pump operation is more satisfactory under full-stroke conditions, many plunger pumps can be equipped with variable-pitch V-belt drives for control of discharge flow rates.

Plunger pumps come with one, two, or three plungers (called *simplex, duplex,* or *triplex* units) with capacities of 40 to 60 gal/min (2.5–3.8 liters/s) per plunger (Fig. 5.11). Plunger pumps should be designed for a minimum discharge pressure of 80 ft (24 m) in small plants and 115 ft (35 m) or more in large plants because grease accumulations in the biosolids/sludge pipelines will cause a progressive increase in system pressure [37].

In addition to being able to pump high solids flows, plunger pumps may operate with up to 10 ft (3 m) of suction lift [19,28]. However, increasing the suction lift will significantly reduce the solids content of the biosolids/sludge flow that can be pumped. The use of a plunger pump with a suction pressure higher than the discharge pressure is not practical because the flow will be forced past the check valves.

5.2.2.4 Piston pumps. Piston pumps are positive-displacement pumps and therefore are similar in action to plunger pumps. The piston pump consists of a guide piston and a fluid-power piston (Fig. 5.12). Like

Figure 5.11 Photograph of a plunger pump.

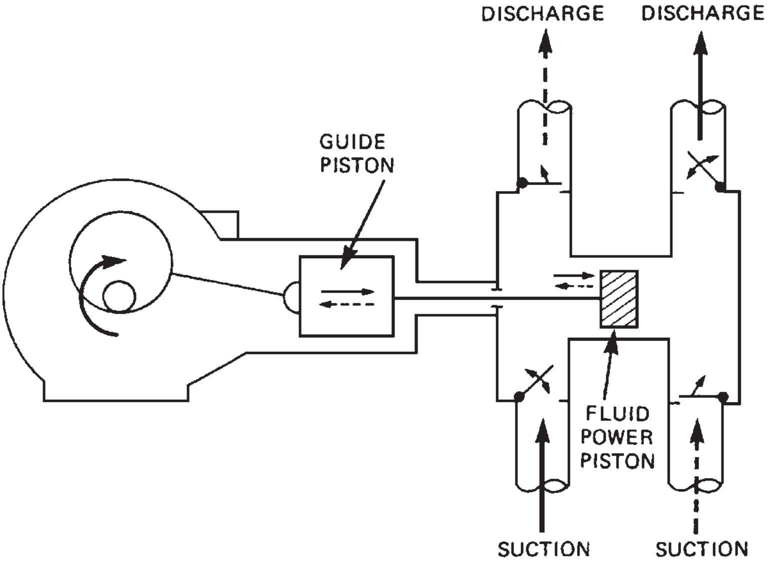


Figure 5.12 Schematic diagram of a piston pump.

plunger pumps, piston pumps are capable of generating high pressures at low flow rates [37].

High-pressure piston pumps are used in high-pressure applications such as pumping biosolids/sludges long distances. High-pressure piston pumps use separate power pistons or membranes to separate the drive mechanism from contacting the flow (Fig. 5.13).

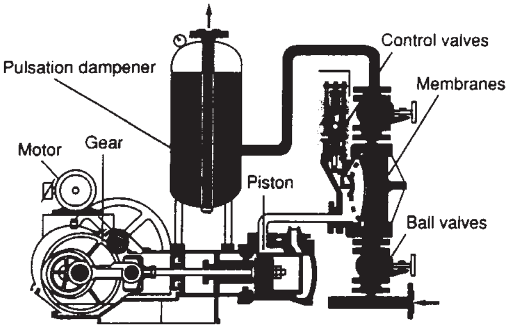


Figure 5.13 Diagram of a high-pressure piston pump.

Advantages cited for the use of high-pressure piston pumps in long-distance transport of biosolids/sludges include (1) discharge pressures of up to 2000 psig (13,800 kN/m^2) can be generated, (2) solids of diameters up to the discharge pipe diameter can be passed, and (3) pumping requirements can be met using a single stage system [37].

A variation of the piston pump called the *combination piston/hydraulic diaphragm pump* has been developed for use when reliability and close control are needed. The combination piston/hydraulic diaphragm pump uses a fluid-power piston driving an intermediate hydraulic fluid (e.g., clean water), which, in turn, pumps the biosolids/sludges into a diaphragm chamber (Fig. 5.14). The speed of the hydraulic fluid drive piston can be controlled to provide pump discharge conditions ranging from constant flow rate to constant pressure [14,19].

5.2.2.5 Progressive cavity pumps. The progressive cavity pump is a positive-displacement pump that consists of a single-threaded rotor that operates with an interference clearance in a double-threaded helix stator. A volume or “cavity” moves progressively from suction to discharge when the rotor is rotating (Fig. 5.15). A progressive cavity pump is self-priming at suction lifts up to 28 ft (8.5 m), but it must not operate dry because it will burn out the rubber stator. The progressive cavity may be operated at flow rates in excess of 1200 gal/min (75 liters/s) and at discharge pressures in excess of 450 ft (137 m) H_2O [37].

The total pressure generated by a progressive cavity pump is divided equally between the number of cavities created by the threaded rotor and the helix stator. Because wear between the rotor and stator is high, maintenance costs for this type of pump can be significant. However, flow rates are controlled easily with these pumps, and pulsation is minimal (Fig. 5.16).

For transport of primary sludges, an in-line grinder normally precedes these pumps, particularly where grit levels are significant.

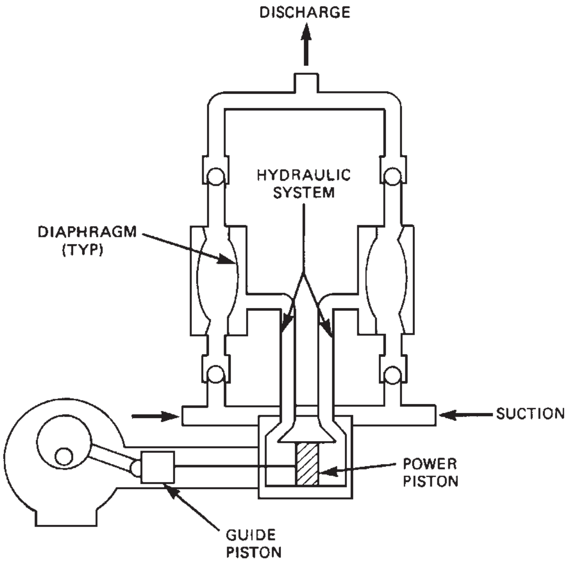


Figure 5.14 Schematic diagram of a combination piston/hydraulic diaphragm pump.

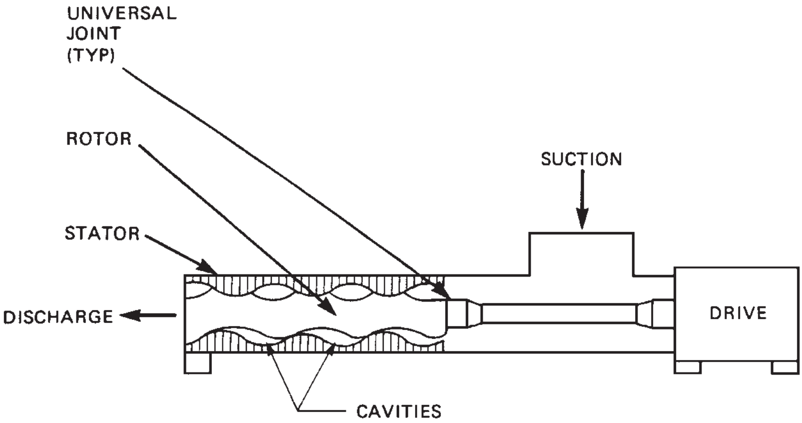


Figure 5.15 Schematic diagram of a progressive cavity pump.

Advantages cited for use of progressive cavity pumps include (1) easily controlled flow rates, (2) minimal pulsation, and (3) simple operation.

5.2.2.6 Diaphragm pump. The diaphragm pump is a positive-displacement pump that uses a flexible membrane that is pushed or pulled to contract or enlarge an enclosed cavity (Fig. 5.17). Biosolids/sludge flow is directed through this cavity by check valves that may be a ball or flap type [14,37].

Figure 5.16 Photograph of a progressive cavity pump.

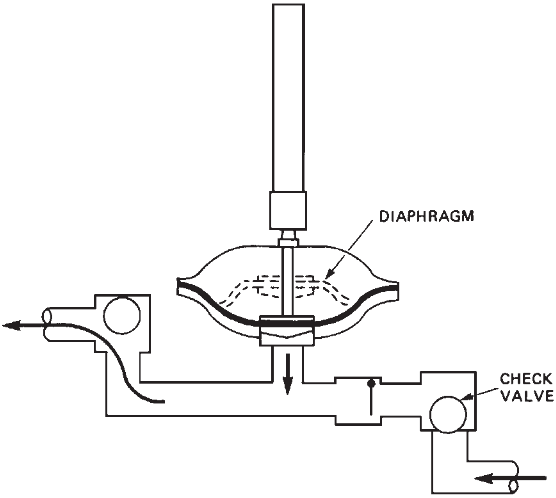


Figure 5.17 Schematic diagram of a diaphragm pump.

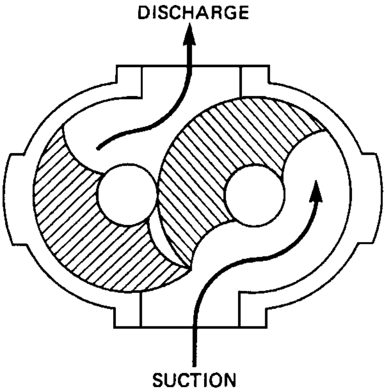
(a)

(b)

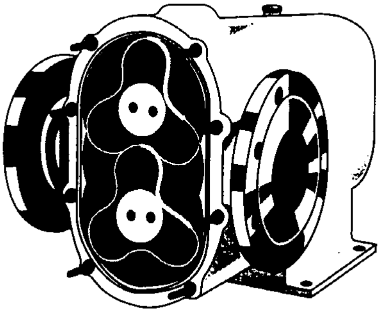
Figure 5.18 Photographs of diaphragm pumps: (*a*) single chamber; (*b*) double chamber.

The capacity of the diaphragm pump is altered by changing either the length of the diaphragm stroke or the number of strokes per minute. Pump capacity can be increased and flow pulsations reduced by providing two pump chambers and using both strokes of the diaphragm for pumping (Fig. 5.18). Diaphragm pumps have minimal maintenance requirements because there are no seals, shafts, rotors, stators, or pacing in contact with the fluid. Diaphragm life is more a function of the discharge head and the total number of flexures than of the abrasiveness or viscosity of the pumped fluid.

Movement of the diaphragm may be accomplished mechanically (i.e., rod or spring) or hydraulically (air or water). The power required to operate air-driven diaphragm pumps is typically double that required for a mechanically driven pump of similar capacity [37]. However, hydraulically operated (air or water) diaphragms generally outwear mechanically driven diaphragms [19]. Moreover,



(a)



(b)

Figure 5.19 (*a*) Rotary pump flow behavior. (*b*) Schematic diagram of a rotary lobe pump.

hydraulically driven diaphragm pumps are suitable for operation in hazardous explosion-prone areas. However, in some locations, high-humidity intake air will cause icing problems to develop at the air-release valve. A compressed-air dryer should be used in the air-supply system when such conditions exist.

5.2.2.7 Rotary pumps. Rotary pumps are positive-displacement pumps that consist of a casing containing gears, cams, screws, vanes, or similar elements activated by rotation of the drive shaft (Fig. 5.19*a*). Fluids are trapped within the spaces between the rotating elements and casing and forced through the pump into the discharge opening. Rotational speed and shearing stresses are low in these pumps, and unlike other positive-displacement pumps, the flow is continuous rather than pulsating. However, like other positive-displacement pumps, rotary pumps must be protected against pipeline obstructions.

An example of a rotary pump commonly used to transport biosolids/sludges is the rotary lobe pump (see Fig. 5.19*b*). Rotary lobe pumps contain two rotating, synchronous lobes that push the fluid

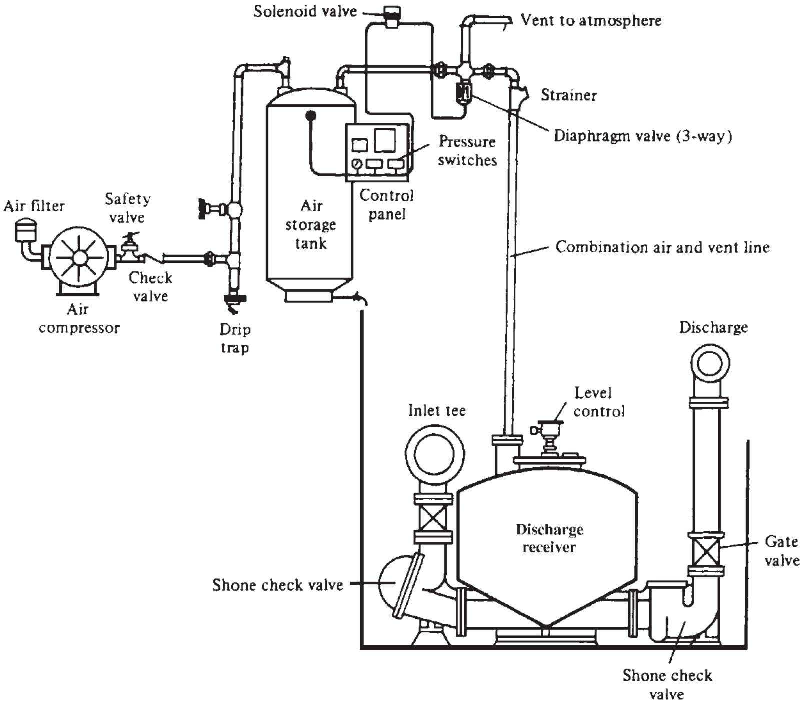


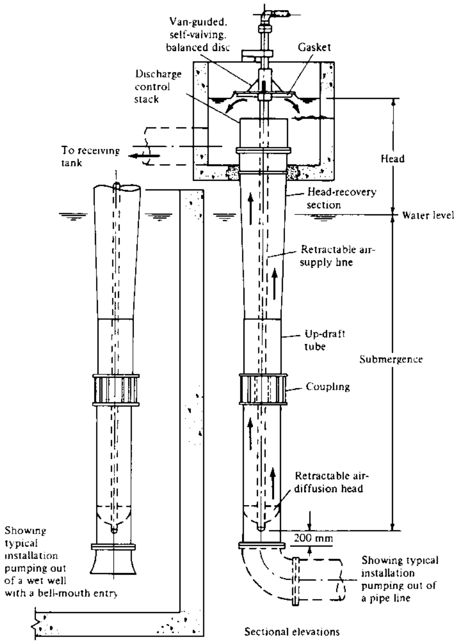
Figure 5.20 Schematic diagram of an ejector pump.

through the pump. An advantage cited for the rotary lobe pump in biosolids/sludge pumping is that lobe replacement is less costly than rotor and stator replacement for progressive cavity pumps [37].

5.2.2.8 Ejector pumps. Ejector pumps use a discharging pot that is intermittently pressurized and discharged by the force imparted by compressed air (Fig. 5.20). These pumps require a positive suction pressure and typically will discharge to a vented holding tank or basin [37].

If ejector pumps are to be used to discharge biosolids/sludges to an anaerobic digester, the air associated with the discharged fluid could produce an explosive gas mixture. Special precautions should be taken to ensure that the pump cannot bleed excessive quantities of air into the digester.

5.2.2.9 Gas-lift pumps. Gas-lift pumps use low-pressure gas released within a confined riser pipe with an open top and bottom to transport biosolids/sludges. The released gas bubbles rise, dragging the fluid out of the riser pipes (Fig. 5.21). Air is commonly used in gas-lift pumps, in which case the pump is called an *airlift pump* [19,37].



(a)

(b)

Figure 5.21 (*a*) Schematic diagram of an airlift pump. (*b*) Photograph of an airlift pump.

The principal advantage of these pumps is the complete absence of moving parts. Gas-lift pumps are usually limited to lifts of less than 10 ft. The capacity of a gas-lift pump can be varied by changing the gas pressure and/or flow rate. Gas-lift pumps using digester gas typically are employed to circulate the contents of anaerobic digesters, whereas airlift pumps are sometimes used for pumping return activated sludge (RAS). Because of their low lifting capability, gas-lift pumps are sensitive to suction and discharge head variations and to variations in the depth of gas released.

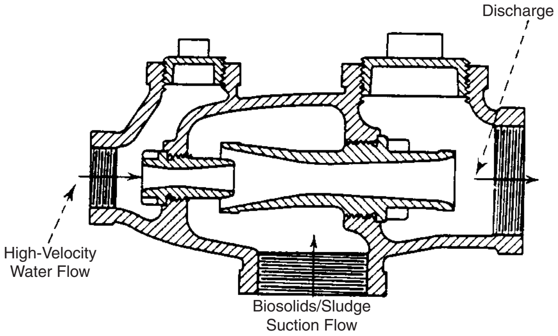


Figure 5.22 Schematic diagram of a water eductor.

5.2.2.10 Water eductors. Water eductors use the suction force created when high-pressure water is passed through a Venturi to induce biosolids/sludge flow [27,37]. Like airlift pumps, water eductors have no moving parts (Fig. 5.22). Although water eductors have been used for removal of grit from aerated grit chambers, their application in biosolids/sludge transport is not widespread [37].

5.2.2.11 In-line grinders. In-line grinders are used to reduce the size of biosolids/sludge particles to minimize the potential for clogging and to eliminate problems with operation of downstream processes (Fig. 5.23). Some of the processes that must be preceded by in-line grinding are listed in Table 5.4 [27,37].

Historically, in-line grinders have had high maintenance requirements [27]. However, newer designs of slow-speed systems have been found to be more durable and reliable. The newer designs include improved bearings and seals, hardened-steel cutters, overload sensors, and mechanisms that reverse the cutter rotation to clear obstructions or shut down the unit if an obstruction cannot be cleared.

5.2.2.12 Sludge degritting. At wastewater treatment plants in which grit-removal facilities are not used ahead of primary sedimentation tanks or where grit-removal facilities are not adequate to handle peak grit loads (e.g., storm event), it may be necessary to remove the grit before further processing of the sludge. The most effective method of degritting sludge is through the application of centrifugal forces in a flowing system to achieve grit separation. Such separation is achieved through the use of a cyclone degritter (Fig. 5.24).

In cyclone degritters, the sludge is applied tangentially to a cylindrical feed section that imparts a centrifugal force. The heavier grit particles move to the outside of the cylindrical section and are discharged through the lower conical section. The degritted organic sludge is discharged through a separate outlet to downstream treatment processes (typically a thickener).

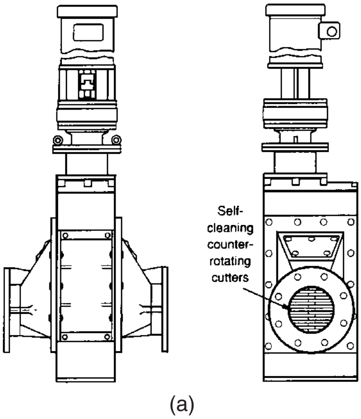


Figure 5.23 (*a*) Schematic diagram of an in-line grinder. (*b*) Photograph of an in-line grinder.

TABLE 5.4 Operations and Processes Normally Preceded by In-Line Sludge Grinding*

Operation or process	Purpose of grinding
Pumping with progressive cavity pumps	To prevent clogging and reduce wear
Solid bowl centrifuges	To prevent clogging; large units may not require grinding
Belt filter press	To prevent clogging of the sludge distribution system, to prevent warping of rollers, and to provide more uniform dewatering
Heat treatment	To prevent clogging of high-pressure pumps and heat exchangers

*Adapted from ref. [27].

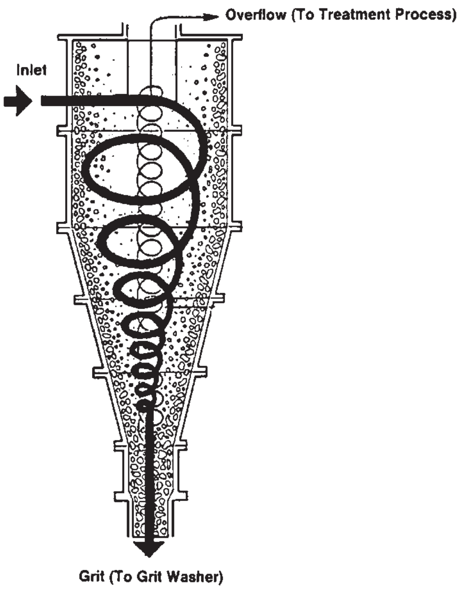


Figure 5.24 Schematic diagram of a cyclone degritter.

The efficiency of the cyclone degritter is a function of the operating pressure and the concentration of organic matter in the sludge. To obtain effective grit removal, the sludge should be relatively dilute. As the solids content of the sludge increases, the particle size that can be removed decreases. The general relationship between the influent sludge solids content and the effectiveness of grit removal for primary sludges is illustrated in Table 5.5.

5.2.2.13 Sludge blending. Sludge is often blended at the wastewater treatment plant to produce a uniform mixture to downstream operations. Sludge-processing operations with short detention times (e.g., dewatering operations, heat treatment, and incineration) require that the influent sludge be relatively homogeneous for effective treatment [27,37]. Sludge from primary, secondary, and advanced wastewater treatment operations may be blended using the methods described in Table 5.6.

5.2.3 Pumping stations

Biosolids/sludge pumping stations often are required both internally to the wastewater treatment plant and for long-distance pipelines. The main purpose of a pumping station is to house the pumps and auxiliary equipment required to pump biosolids/sludges through pipelines. Modern pumping stations usually are automated

TABLE 5.5 Grit-Removal Efficiency Using a Cyclone Degritter for Primary Sludge*

Primary sludge solids content (%)	Mesh size of grit removed†,‡
1	150
2	100
3	65
4	28–35

*Adapted from refs. [27,37].

†A 12-in (0.3 m) hydroclone operated at 6 psig (42 kN/m^2) with a sludge flow rate of 205 gal/min (12 liters/s).

‡Ninety-five percent or more of indicated particle size is removed.

TABLE 5.6 Typical Methods Used to Blend Sludge at Various Locations within the Wastewater Treatment Plant*

Primary settling tanks	Secondary and/or advanced wastewater treatment sludges can be returned to the primary settling tanks, where they will settle and mix with the primary sludge.
Pipes	This procedure requires careful control of sludge sources and feed rates to ensure adequate mixing. Without careful control, wide variations in sludge consistency may occur.
Sludge processes with long retention times	Aerobic and anaerobic digesters (complete-mix type) can blend the feed sludges uniformly.
Separate blending tanks	This practice provides the best opportunity to control the quality of the blended sludges.

*Adapted from refs. [27,37].

for routine operation. The smaller pumping stations typically used for biosolids/sludge transport normally are unattended and require little attention other than a daily check on the proper functioning and lubrication of the equipment.

Pumping station substructures should be made of reinforced concrete. The exterior walls below grade and wet-well walls below the maximum high wet-well level should be coated with tar to prevent leakage. The ground floor of the station must be set above the flood plane of the surrounding area to eliminate the possibility of flooding the station.

Pumping stations must have facilities for removal and/or servicing of the equipment. Typically, an overhead bridge crane, monorail, or lifting hooks are used [24,27]. In addition, to ensure that a suitable system for moving heavy equipment exists, doors must be made large enough for the removal of equipment, and floor openings or floor hatches must be provided for transferring equipment from lower floors.

5.2.3.1 Wet wells. Biosolids/sludges enter the pumping station through the wet well. Because of the potential generation of explosive and toxic gases, wet wells must be vented to the atmosphere. Wet wells should be divided into two or more sections so that a portion of the station may be taken out of service for inspection and cleaning without shutting down the entire pipeline. Each section of the wet well should have an individual inlet and slide gate to direct the flow from that section when it is taken out of service. The sections, however, must be interconnected so that the total storage capacity of the station is available to prevent short cycling of the pumps.

The minimum required storage volume depends on the type of pump operation, constant- or variable-speed. If constant-speed operation is selected, the wet-well storage volume must be adequate to prevent the short cycling of the pumps (i.e., frequent starting and

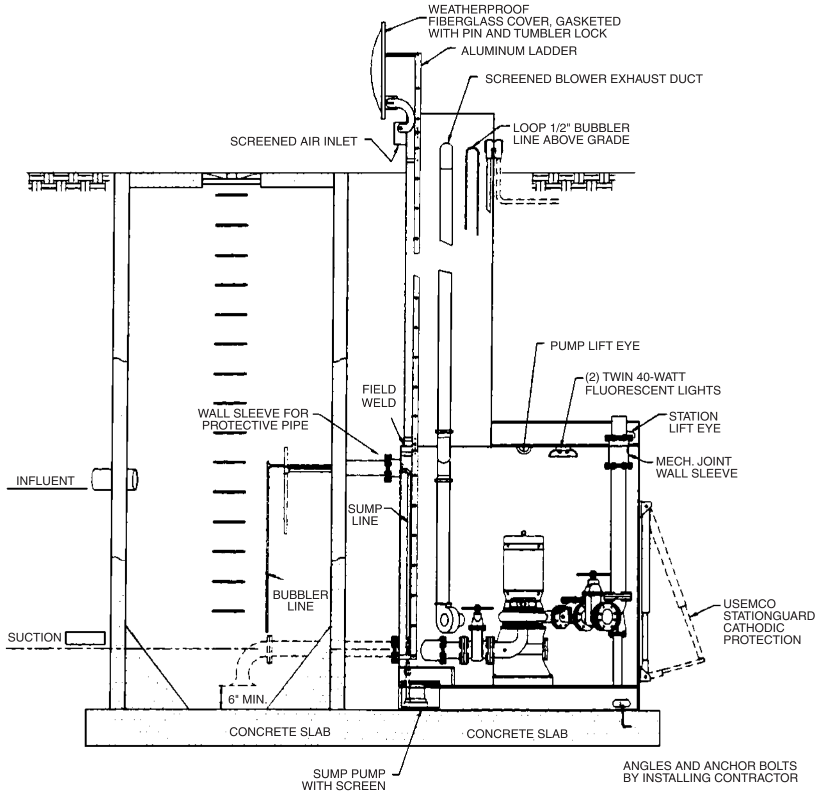


Figure 5.25 Schematic diagram of a pumping station for biosolids/sludge transport. (*Courtesy of USEMCO, Inc.*)

stopping of pump) as well as to provide sufficient submergence of the pump suction inlet (Fig. 5.25). On the other hand, if the pumps are operated using a variable-speed drive that automatically adjusts the pumping rate to match the inflow to the station, the required minimum wet-well storage volume is significantly smaller than in the constant-speed pump operating condition.

The time between pump starts is a function of the pumping rate and the quantity of flow entering the station. The minimum volume of the wet well between start and stop elevations for a single pump or a single-speed control step for multiple-speed operation is given by Eq. (5.16).

The minimum cycle time for a single-pump operation occurs when the inflow is exactly half the pump capacity. Under these conditions, the on and off times are equal. The pump is on a longer time and off a shorter time for larger inflows, and vice versa for smaller inflows. However, in both cases, the cycle time is greater.

$$\text{Wet-well volume (gal, m}^3\text{)} = \theta q/4 \tag{5.16}$$

where θ = minimum time (in minutes) of one pumping cycle (i.e., time between successive starts or changes in speed of a pump operating over the control range)

q = pump capacity (gal/min, m^3/min) or increment in pumping capacity where one pump is already operating and a second pump is started

If the wet-well volume computed using Eq. (5.16) requires an unusually large wet-well volume in a pumping station with two identical pumps, one of which is in standby mode, the wet-well volume can be reduced by 50 percent by installing an automatic alternator in the pump control circuit. An alternator, which starts and runs the pumps alternatively, has the effect of reducing the value of θ by half for a single pump and motor [24,27,37].

Located adjacent to the wet well, the dry well is used to house the pumps and the related suction and discharge piping and valves. A gutter should be located in the dry well along the wall separating the wet and dry wells to convey seepage, pump drainage, and floor washdown to a sump.

The pumps used in pumping stations are normally vertical-shaft, single-suction units installed in a dry well with motors mounted on a floor above the pump. The pumps should be set so that the high point of the casing is below the minimum level of the biosolids/sludge in the wet well. This setting ensures that air cannot enter the pump through the packing when the pump is not operating and that the pump is full when started automatically.

Since most pump maintenance involves the rotating parts of the pump (e.g., impellers, shaft, bearings, etc.), the pump should be accessible. Pumps are more accessible if the motor is positioned on an upper floor and the drives are attached to the pump shaft through a flexible universal joint. With this arrangement, removing only the flexible shaft is necessary to disassemble the pump [24].

The required discharge rate of biosolids/sludges determines the minimum capacity of the pumps and the pumping station. Ideally, pumps will withdraw biosolids/sludges from a large-volume containment system (e.g., a digester) at a steady rate. The design of pumping stations used to transport biosolids/sludge through long-distance pipelines should account for the following factors:

- Characteristics of biosolids/sludge, e.g., type of biosolids/sludge, solids content, degree of stabilization, viscosity, etc.
- Pressure that the pumps must overcome
- Need for standby reliability, i.e., how long can the pumping station be out of service for maintenance, power failure, etc.
- Anticipated pumping station life
- Need for future expansion of capacity
- Ease of operation and maintenance

Finally, proper pumping station design must provide a means to flush the pipeline to remove accumulated grease or other blockages. The pumping station should provide sufficient storage capacity for biosolids/sludges and standby power to handle at least 2 days' volume of flow when the pumping station is shut down for maintenance.

5.2.4 Cost estimation of pipeline transport systems

The cost of pipeline transportation is directly related to the capacity and length of the pipeline system. Variables affecting the cost of pipeline transportation are summarized by the following list:

- Type of biosolids/sludges
- Required pumping rate
- Solids content and viscosity
- Transportation distance
- Pipeline alignment
- Topography of the area through which the pipeline is to be constructed

If, after performing an initial design and cost estimate, a pipeline is selected for biosolids/sludge transport, the following issues must be addressed:

1. *Alternate routes.* Preliminary planning should be used to reduce the number of potential pipeline routes. Crossings can add significantly to the cost of a pipeline and to the complexity of construction. The shortest distance with the least elevation difference and fewest crossings should be the primary goal.
2. *Operating program.* A comparison of constant versus variable-speed pumps is important in determining the design flow through the pipeline. Variable-speed pumps allow for continuous operation and lower storage requirements. Although constant-speed pumping will require more storage for peak flow dampening, it is usually more energy efficient. The maximum and minimum flow velocities are also important considerations in biosolids/sludge pipeline design. Velocities significantly less than 3 ft/s can promote solids settling and decomposition, whereas velocities much higher than this will cause scouring and increase head-loss [37].
3. *Physical design.* Pipeline friction losses should be minimized because they contribute significantly to pumping costs. Abrupt changes in slope and direction should be avoided. Depending on the nature of the biosolids/sludge and the characteristics of the soil, corrosion-control features should be incorporated in the pipeline design. In addition, air and vacuum relief valves should be provided at high points in the line, drains at low points, cleanouts at abrupt changes in direction, and frequently spaced isolation valves to allow shutdown in case of pipe failure and/or the need for pipeline repair [27,37].
4. *Pumping facilities.* More than one pumping station may be needed if the pipeline distance is long. The number of pumping stations should be balanced with the size and number of pumps required to determine the most cost-effective combination. Pumps should be appropriate for the type of biosolids/sludge to be pumped, with standby pumping units provided.
5. *Emergency operation.* Several days' storage should be provided in case of equipment failure. Digesters can be used for this purpose, if available. Standby power normally should be provided if there are not two independent sources of electricity to the pumping station. Additional storage may be substituted for standby power under certain conditions, although continuous operation is preferable.

6. *Excavation condition verification.* Field tests should be used to establish or verify the subsurface soil conditions. Borings should be taken after the pipeline route has been established but prior to final design. The field tests should be used to isolate areas where special design considerations are needed. If highly unusual localized conditions exist, they should be avoided, if possible, or additional field tests made.
7. *Acquisition of right-of-way.* Right-of-way easements must be acquired for pipelines on private property. This process should be initiated in the early stages of the project. The preferable approach is to obtain access rights on easements owned or controlled by other utilities when possible. Otherwise, suitable land access must be obtained through negotiations with landowners.

5.3 Dewatered Biosolids/Sludge Conveyance

Dewatered or dried biosolids/sludges can be conveyed by most types of industrial solids-handling equipment (e.g., belt conveyors, screw conveyors, slides, chutes, etc.). Because the consistency of the solids may vary, the design of dewatered biosolids/sludge conveyance equipment should be based on a worst-case scenario.

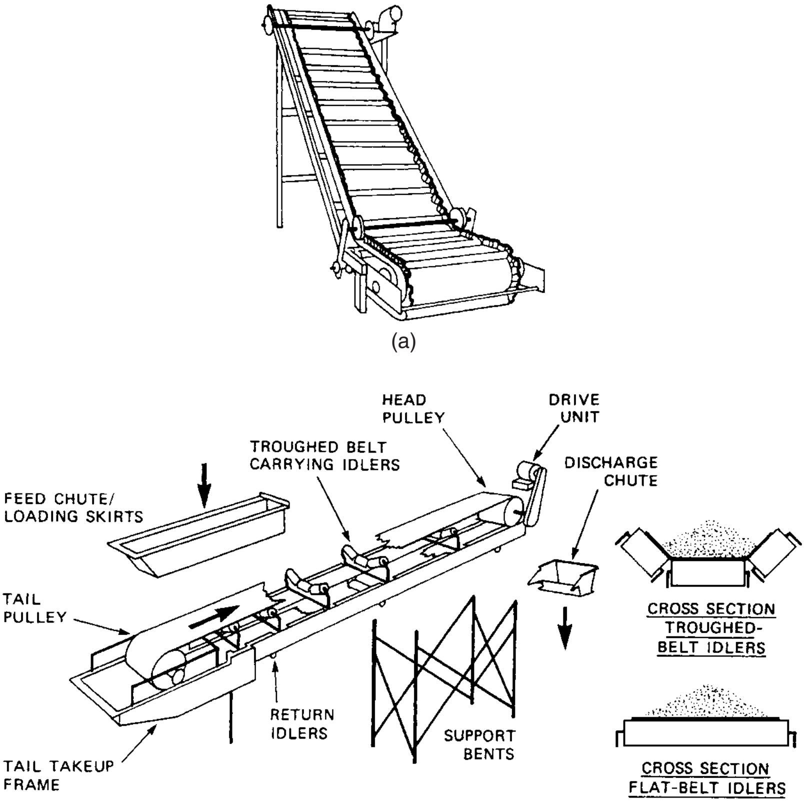
5.3.1 Conveyors

Conveyors transfer dry biosolids/sludges over relatively short distances. The principal types of conveyors may be classified as (1) belt conveyors and (2) screw conveyors. Each of these systems is described briefly in the following subsections.

5.3.1.1 Belt conveyors. Flat-belt conveyors are simple and reliable methods to transport dried or dewatered biosolids/sludge. Flat-belt conveyors are available in styles with integral pockets, sidewalls, and cleats that allow steep, high-capacity operation (Fig. 5.26*a*). Flat-belt conveyor designs use both roller and slider-bed supports.

A variation of the flat-belt conveyor is the troughed-belt conveyor (Fig. 5.26*b*). The troughed-belt conveyor may be equipped with load-cell weigh bridge sections for totalization of conveyed biosolids/sludge weight. Totalization is useful when an accurate solids balance must be calculated for a dewatering facility.

Biosolids/sludges concentrated to at least a semisolid state (*ca.* 15 percent solids) can be conveyed at about 18° maximum inclination on



(a)

(b)

Figure 5.26 Schematic diagram of (*a*) a flat-belt conveyor and (*b*) a troughed-belt conveyor.

troughed-belt conveyors [19,37]. Biosolids/sludges with a higher solids content may be transported at steeper slopes. Experience in the use of belt conveyors for the transport of biosolids/sludges has led to the general design and operational guidance provided in Table 5.7. Figure 5.27 is a schematic diagram of the basic inclined-belt conveyor design, while Fig. 5.28 depicts a flat-belt conveyor used to transport dewatered biosolids/sludge.

5.3.1.2 Screw conveyors. Screw conveyors are a reliable and economical method used for horizontal movement of biosolids/sludges or to convey dewatered biosolids/sludges up inclines (Fig. 5.29). The maximum slope over which the biosolids/sludge can be transported depends on the moisture content and consistency of the biosolids/sludge. Conservative sizing, abrasion-resistant construction materials, and

TABLE 5.7 Design and Operational Guidance for Belt Conveyance Systems*

Reduction of splashing at transfer point. The belt conveyance drop point should be enclosed and the drop distance minimized. Where long drops cannot be avoided, transfer chutes should have interior impact baffles to dissipate the momentum of falling biosolids/sludge.

Removal of biosolids/sludge from returning belts. Counterweighted rubber-bladed scrapers at head pulleys are not effective in scraping biosolids/sludge off return belts and often require significant maintenance. The use of adjustable-tension finger-type scrapers is recommended.

Ensuring minimum pulley slippage. Appurtenances that contact the dirty side of the belt should be avoided. Where long lifts are required, multiple short belts should be used instead of one long belt to avoid the need for gravity take-up. Figure 5.27 illustrates the recommended design layout of inclined-belt conveyors.

Importance of housekeeping facilities. Nonskid cover plates should be used for all access areas except those immediately over storage hoppers. Convenient hose stations should be located to serve all areas. Floors and slabs should be provided with exaggerated drainage slopes and should drain into sumps.

*Adapted from ref. [37].

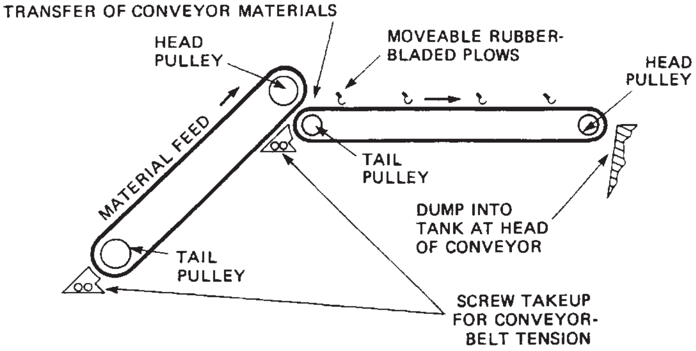


Figure 5.27 Recommended design layout for belt conveyors.

integral washdown systems within the enclosed housings are recommended for these systems [19,37].

Since biosolids/sludge can accumulate on the bearings and restrict flow, screw conveyors should not include internal intermediate bearings. Because of this design specification, screw-conveyor lengths normally are limited to 20 ft. Screw conveyors with reversible direction and/or several slide-gate discharge openings allow the point of conveyor discharge to be varied. Figure 5.30 depicts a screw-conveyor system equipped with multiple shafts enclosed in a single housing.

5.3.1.3 Chutes and inclined planes. Chutes and inclined planes employed for biosolids/sludge transport should be tested for minimum

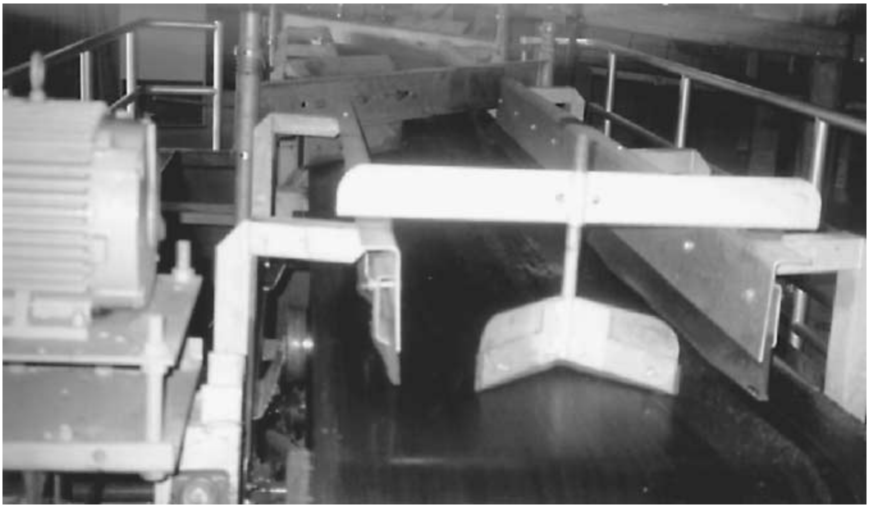


Figure 5.28 Photograph of a flat-belt conveyor for sludge. Note the sludge plow in the foreground.

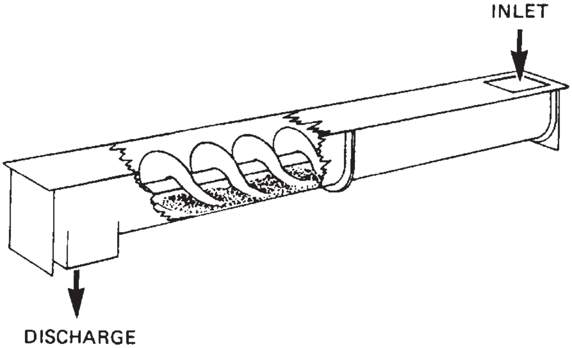


Figure 5.29 Schematic diagram of a screw conveyor.

inclination. In general, inclination for dewatered biosolids/sludges should be greater than 60°. For dried biosolids/sludge, the angle of inclination must be greater than the material's natural angle of repose [19,37].

5.4 Long-Distance Biosolids/Sludge Transportation

It is often necessary to transport biosolids/sludges long distances from the wastewater treatment plant (e.g., beyond 30 mi). In general, long-distance transport is limited to trucking, rail transport, and barging.

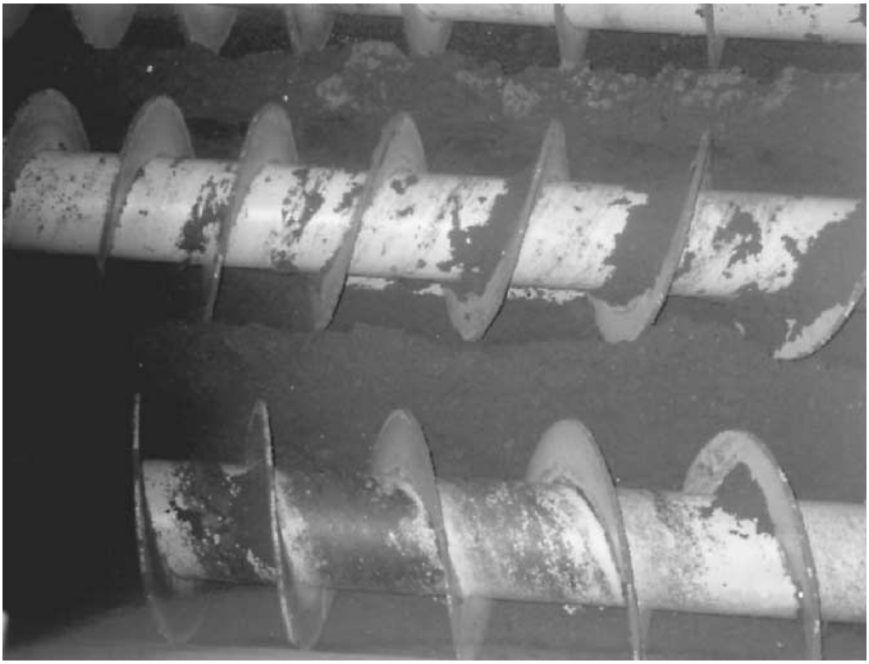


Figure 5.30 Photograph of a screw conveyor for sludge transport.

5.4.1 Truck transportation

For many wastewater treatment plants, trucking represents a cost-effective option for transport of both liquid and dewatered biosolids/sludge. Moreover, trucking provides flexibility not found in other modes of transport. For example, terminal points and routes can be changed readily for trucks at low cost relative to rail or barge transport.

For wastewater treatment facilities employing truck transport of biosolids/sludge, loading facilities should be accessible and positioned in a convenient location. Depending on the type of biosolids/sludge being hauled, hoppers, conveyor belts, or pipelines are needed to load the trucks. Equipment washdown facilities and adequate parking should be nearby. Similar facilities for truck unloading may be required for the biosolids/land-application site(s) and/or biosolids/sludge storage facility.

5.4.1.1 Biosolids/sludge hauling trucks. Because they must comply with overall vehicle weight, height, and gross weight restrictions, biosolids/sludge hauling trucks are similar in design to standard highway trucks [37,65]. Most of the variability seen in the design of trucks that transport biosolids/sludge is found in the containment-body configuration, with ease and speed of loading and unloading being of critical importance (Fig. 5.31).

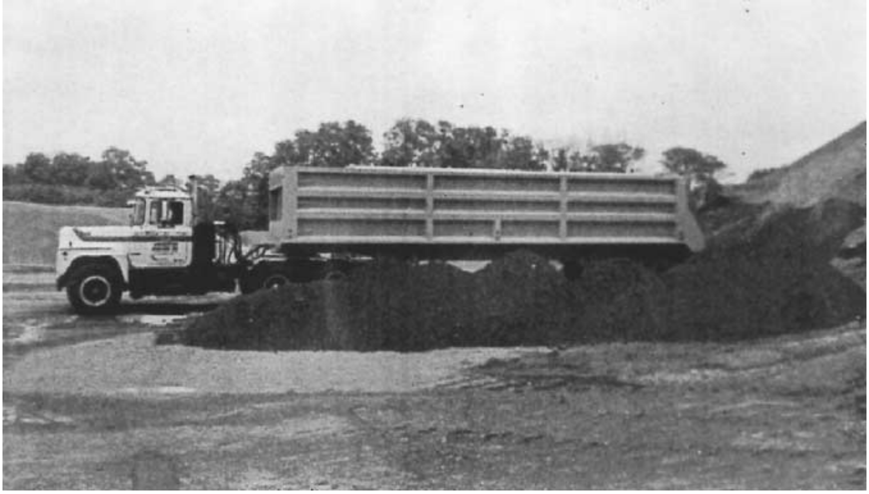


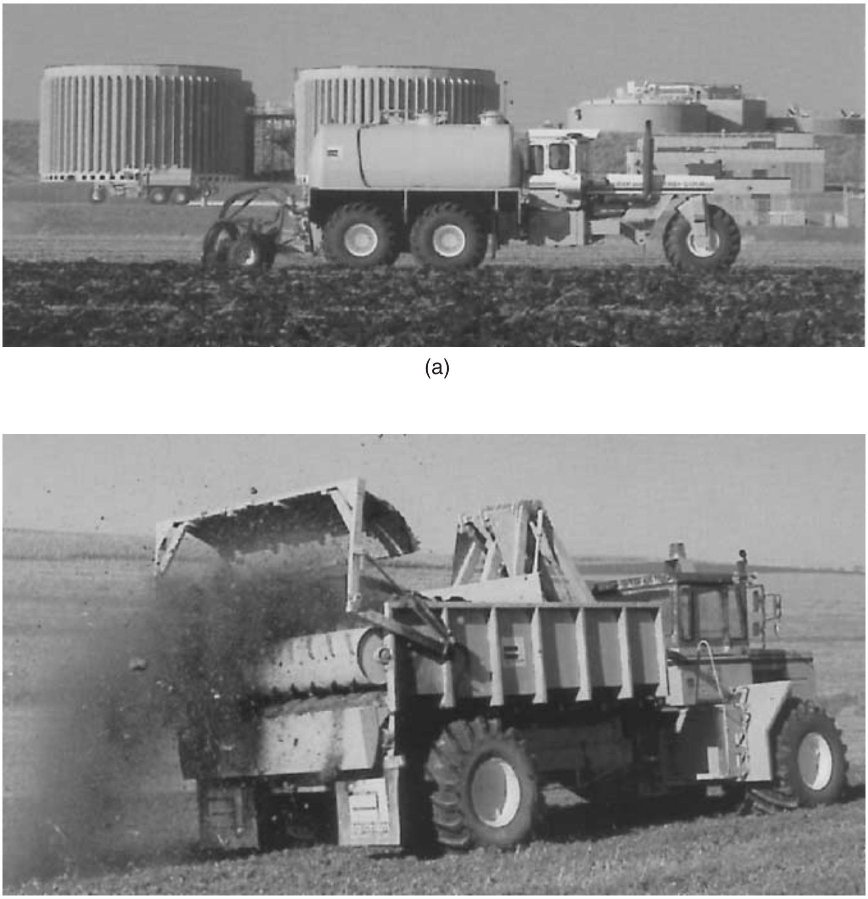
Figure 5.31 Photograph of a typical biosolids/sludge hauling truck.

Where it is determined that economic, environmental, and institutional considerations allow direct land application of biosolids, tank trucks are available equipped with specially designed spreaders, while some are equipped with subsoil injectors (Fig. 5.32). Use of dual-purpose trucks allows transport and disposal without an intermediate storage/pumping step.

Because of health considerations, spillage or leakage from biosolids/sludge hauling operations must be minimized. This requirement has meant a shift away from belly-dump vehicles even for dewatered materials. Tank-type trucks are the most common type of vehicle for hauling biosolids/sludges (Fig. 5.33). These vehicles require large hatch openings for loading purposes.

5.4.1.2 Contractual considerations for trucking. The choice between using wastewater treatment plant employees or contracting with a private company to haul biosolids/sludge has been found to be a function of economics as well as the size of the wastewater treatment plant. Smaller plants are more inclined to use both their own vehicles and staff to transport biosolids/sludge.

Contracting for truck transport of biosolids/sludge can be limited to providing only tractor units and driver services, with the trailers owned and maintained by the wastewater treatment plant. The benefit of this contractual approach is that treatment plant personal assigned to the biosolids/sludge handling operations are working in the immediate vicinity of the trailers and can relocate the trailers under a conveyer belt at the optimal time during biosolids/sludge processing.



(a)

(b)

Figure 5.32 Photographs of a typical biosolids/sludge hauling truck: (*a*) subsurface injection system; (*b*) surface-application system. (*Courtesy of Ag-Chem Equipment Co., Inc.*)

A common problem with truck hauling is scheduling. In general, the total cost of truck transport will decrease if the daily period of truck operation is increased. However, restrictions may be placed on truck operations, such as requiring specific routes or limiting operations to daylight hours.

Under normal conditions, an engineer will design the biosolids operating schedule to optimize the transportation of biosolids. The design engineer can easily develop an operating schedule for biosolids/sludge hauling based on the following:

- Quantity of biosolids/sludge to be hauled
- Average round-trip driving time requirement

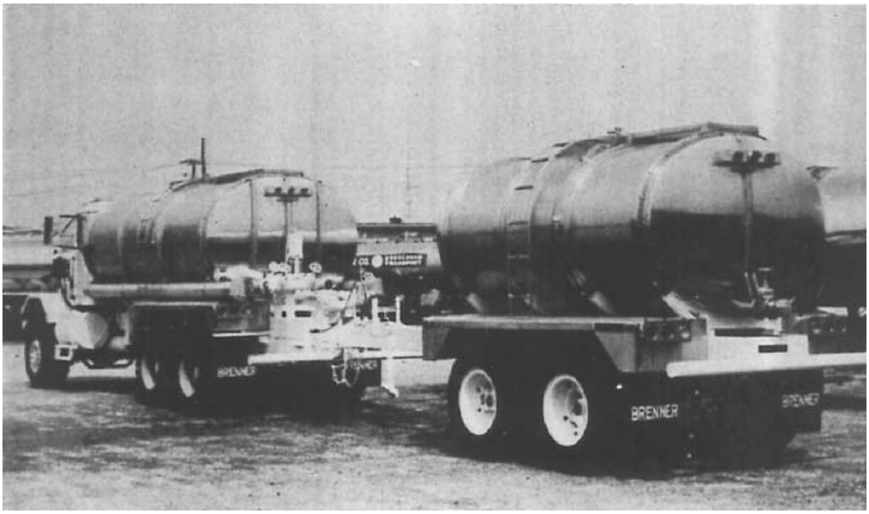


Figure 5.33 Photograph of a typical biosolids/sludge hauling truck.

- Biosolids/sludge loading and unloading time requirement
- Truck maintenance downtime
- Truck capacity
- Length of working shifts and number of laborers working

In designing biosolids/sludge handling facilities, it is desirable to provide several dump points at the facility with the capability to quickly shift from one to another. If trailers are used, the ability to fill several units before the tractor unit returns adds flexibility to scheduling and reduces storage requirements. Trucks can be used for hauling biosolids/sludges either to the final disposal site(s) or to an intermediate transfer point (e.g., railroad yards, barge loading area, etc.).

During the planning stage, the size and number of vehicles to be employed must be estimated. In determining the size and number of vehicles needed for transportation of biosolids from the treatment plant to the land-application site(s), the following factors should be considered:

- Quantity of biosolids, present and future
- Type of biosolids (liquid or dewatered)
- Distance from treatment plant to application site(s) and travel time
- Type and condition of roads
- Provisions for vehicle maintenance

- Scheduling of biosolids application
- Percent of time when the biosolids transport vehicles will be in productive use

The operating program for biosolids/sludge transportation can be simple or very complex. The haul distance should be minimized to reduce costs, travel time, and the potential for accidents. Fuel availability and costs can have a profound impact on the operation of biosolids/sludge transportation activities.

Travel time can be estimated from anticipated road speeds and the haul distance, allowing for differences in speed for various segments of the route and the anticipated traffic conditions. Periods of heavy traffic should be avoided from a safety standpoint. Unfavorable topographic features, road load limits, population patterns, etc. may influence routing such that the shortest haul distance is not the most favorable.

5.4.2 Rail transport

Rail transport is suitable for transporting biosolids/sludges of any solids concentration. Although rail transport has a lower energy cost per unit volume than trucking, rail transport suffers from many of the same problems as pipelines, namely, large unrecoverable capital expenditures and fixed terminal points. However, special circumstances may favor rail hauling of biosolids/sludge over either trucking or pipelines. For example, if biosolids/sludge are to be used to reclaim stripmined lands, a rail line that had been built to remove coal or ore may be used to transport biosolids.

The construction of a new railroad line may not be cost-effective or even possible for the sole purpose of transporting biosolids/sludges. New construction normally is limited to a short spur from a mainline railroad or the provision and/or expansion of small switching yards on a large treatment plant site. Since any attempt at constructing longer railroad lines solely for biosolids/sludge transport is impractical, this limits the overall route selection to railroad lines already in existence. This fact will limit either the selection of rail for biosolids/sludge transport or the choice of disposal sites.

5.4.2.1 Contractual considerations for railroads hauling of biosolids/sludge. Since a substantial long-term economic gain may be recognized, railroads are generally interested in providing biosolids/sludge transport. However, many railroads may be unfamiliar with biosolids/sludge handling. Similarly, many wastewater treatment plant engineers and operators are unfamiliar with railroad contracting procedures.

In the United States, a railroad company must haul railroad cars. Therefore, a contract must be negotiated with the railroad for biosolids/sludge transport. Since railroads are a regulated utility, the rate-quotation process can be complicated. Railroad hauling rates are of two general types: (1) class rate and (2) special-commodity rate. Class rates may be readily obtained by the wastewater treatment plant but are normally prohibitively expensive for biosolids/sludge transport. To obtain a special-commodity rate, the wastewater treatment plant must follow a prescribed process:

1. File an application with the railroad that includes a complete description of what is to be shipped, the frequency of shipping, the approximate loading and unloading time, and a statement of the transportation price the shipper (i.e., wastewater treatment plant) would be willing to pay per 100 lb (45.4 kg) net weight.
2. The local railroad reviews the load, distance, terrain, switching requirements, and carrier competition and calculates a commodity transport rate.
3. The local freight bureau publishes the commodity rate for a notice period of 30 days for review by other, possibly competing carriers and by one of the regional freight bureaus. The regional freight bureaus are conglomerations of the local ones, and they regulate and control prices between bureau jurisdictions.
4. Comments and appeals of commodity rates can be made to the Interstate Commerce Commission (ICC). An appeal concerning a proposed commodity rate will cause the rate to be suspended until the suspension board of the ICC can hear the case, at which time the carrier will have to justify the rate.

In addition to securing an acceptable price for shipping biosolids/sludge by rail, the wastewater treatment plant must make arrangements to obtain a suitable number of railcars to ensure that the transport system is uninterrupted. There are three methods of ensuring railroad car equipment availability: (1) leasing, (2) outright purchase, or (3) assigned service of the required number of cars through a hauling contract with the carrier [49]. Generally, an assigned service option is only available for solids or semisolid material (e.g., dewatered biosolids/sludge) that can be transported in hopper cars. Liquid biosolids/sludge must be carried in tank cars that are normally leased from a private tank car rental company. An exact determination of the number of cars required is site- and area-specific. It also should be recognized that the speed of railroad transport would depend in part on the track conditions and on the railroad's normal traffic schedule.

In addition to providing a sufficient number of cars, railroad transport of biosolids/sludge requires loading and storage equipment (e.g., tanks, pumps, piping for liquid biosolids/sludge, and hoppers and conveyors for dewatered material), railroad sidings, and unloading equipment. Car maintenance and storage normally will be the responsibility of the railroad car owners, but the wastewater treatment plant may be required to provide railroad car cleaning and washdown facilities.

5.4.3 Barge transportation

Barge transportation of biosolids/sludges is generally only practical for liquid biosolids/sludges that require long-distance transportation (typically over 30 mi). A major consideration in evaluating barge transportation is the proximity of the wastewater treatment plant to a suitable waterway. If the wastewater treatment plant is not close to a waterway, it may be desirable to locate a biosolids/sludge storage tank or lagoon near the barge loading dock. In most cases, it is desirable to load and meter the biosolids/sludge flow from a fixed pumping station located on a fixed wharf. Once loaded on the barge, the biosolids/sludge transit time will be affected by the (1) traffic on the waterway, (2) physical features such as drawbridges, locks, and height limitations, and (3) natural characteristics such as currents, tides, and wave heights [37]. A pump mounted on the barge itself accomplishes offloading of the biosolids/sludge. The transfer site should be located near a dock capable of mooring a suitably sized barge.

Only large wastewater treatment plants should consider ownership of the barge (tug or powered barge). For most wastewater treatment plants, the choices available for the hauling of biosolids/sludge are contracting for either complete barge transport services or for tug service alone. Full-service contracts are best for small operations with intermittent transport requirements. Moderate to large plants generally favor contract towing only with the barge(s) owned by the wastewater treatment authority. In certain cases, it may be possible for two or more wastewater treatment authorities to join in a common contractual agreement whereby biosolids/sludges for two or more facilities are transported by the one haul contractor.

An important element of barge transportation is a well-developed spill-prevention and cleanup program. Spills can result in serious water pollution and health problems. Biosolids/sludge spills should be contained immediately and transferred to storage tanks or another barge as quickly as possible to reduce risks to the public health and/or the environment. Paying careful attention to system design and operator training can minimize the risk of spills.

5.5 Storage of Biosolids/Sludge

Storage is an integral component of all biosolids/sludge management and disposal systems. Storage allows the various wastewater and sludge treatment processes to operate on schedules that optimize their individual performance. For example, although solids are generated within the wastewater treatment system 24 hours per day, in many cases it may be most convenient and cost-effective to operate certain sludge-processing systems only during the day shift [28,37]. This degree of operational flexibility is available only if sufficient storage is provided.

In addition to the need to provide adequate storage of solids within the wastewater treatment plant, storage also must be provided upstream from any biosolids beneficial-use system, since land application of biosolids occurs only at certain times of the year.

5.5.1 Types of storage

At the wastewater treatment plant, biosolids/sludge may be stored within (1) wastewater treatment systems, (2) sludge treatment and disposal systems, and (3) tanks, lagoons, drying beds, bins, and stockpiles. Regardless of the storage facility employed, there are basically three mechanisms through which solids may be stored:

1. *Single-phase concentration.* Solids accumulate within a completely mixed vessel, which is reflected in an increase in the suspended solids concentration. An example of this type of storage is the increase in mixed-liquor volatile suspended solids (MLVSS) concentration within the aeration basin of an activated-sludge treatment system when solids wasting is reduced or eliminated.
2. *Two-phase concentration.* Solids accumulate within the sludge blanket of a liquid/solids separation device. An example of this type of storage is the gradual increase in sludge blanket depth in primary or secondary sedimentation tanks as a result of reducing the rate of sludge withdrawal.
3. *Displacement.* Solids accumulate as a result of changing total system volume. An example of this type of storage occurs when sludge is pumped into an anaerobic digester equipped with a floating cover that is allowed to expand.

Although one solids storage mechanism may predominate in certain unit operations, in some cases, two or three of the described storage mechanisms may act in concert to increase the overall storage capacity of a wastewater treatment plant. For example, solids can accumulate within an anaerobic digester equipped with a floating cover by displacement as well as by two-phase concentration. In addition to the physical mechanisms within the treatment plant, biosolids/sludge storage may also be categorized by the solids detention time as follows:

1. *Equalization storage.* Solids are stored for 4 days or less.
2. *Short-term storage.* Solids are stored for 4 weeks or less.
3. *Long-term storage.* Solids are stored for more than 4 weeks.

Table 5.8 summarizes the available biosolids/sludge storage options at the wastewater treatment plant by solids type, primary mechanism of storage, and solids detention time.

5.5.1.1 Storage within wastewater treatment system. In general, solids should not be stored in wastewater treatment systems (e.g., primary and secondary wastewater treatment systems) unless storage provides operational benefits that clearly outweigh the risks to effluent water quality [7,18,28]. For many small wastewater treatment plants (i.e., those processing less than 2 million gal/day), most or all of the biosolids/sludge processing occurs only during the day shift. In this case, providing adequate solids storage within the wastewater treatment systems is essential. In most cases, when the wastewater treatment system is employed for solids storage, accumulation of solids is permitted to occur within the primary sedimentation system, aeration basin, and/or secondary sedimentation tank.

When two-phase concentration storage is being provided within the primary sedimentation tank, solids processing systems can operate at rates independent of the rate at which solids are removed from the wastewater. This characteristic of two-phase concentration storage is especially useful for small wastewater treatment plants that are not staffed continuously and for any size plant that experiences large diurnal or seasonal fluctuations in settleable solids [37].

The volume and solids content of sludge discharged from the primary sedimentation tank may be controlled through the use of (1) blanket-level instrumentation, (2) a programmable timer, and (3) density instrumentation. Ideally, all three devices can control primary sludge pump operation to ensure that effluent water quality is not compromised as a result of solids accumulation. For example, blanket-level instrumentation can be programmed to determine the time when the sludge discharge pump is activated. The control timer can then be used to set the cyclic period when the pumps can share the discharge piping (if necessary), while the density instrumentation can shut down the pumping system when the pumped sludge is below a preset density value [27].

TABLE 5.8 Solids Storage Options at the Wastewater Treatment Plant*

		Detention time†		
	Mechanism	Equalization	Short term	Long term
Storage within wastewater treatment process				
Primary clarification	Two-phase conc.	X	—	—
Aeration basin	Single-phase conc.	X	X	—
Secondary clarification	Two-phase conc.	X	—	—
Storage within sludge treatment processes				
Gravity thickener	Two-phase conc.	X	—	—
Anaerobic digester	Single, two-phase conc., displacement	X	X	—
Aerobic digester	Single, two-phase conc.	X	X	—
Composting	Two-phase conc., displacement	—	X	X
Drying bed	Two-phase conc., displacement	—	X	X
Faculties provided primarily for storage of liquid sludge				
Holding tanks	Single, two-phase conc., displacement	X	—	—
Facultative sludge lagoons	Two-phase conc.	—	X	X
Anaerobic liquid lagoons	Two-phase conc.	—	X	X
Aerated storage	Single, two-phase conc.	X	X	—
Facilities provided primarily for storage of dewatered sludge				
Sludge drying lagoons	Two-phase conc., displacement	—	—	X
Confined hoppers/bins	Displacement	X	X	—
Unconfined stockpiles	Displacement	X	X	X

*Adapted from ref. [37].

†X denotes the typical process solids detention time.

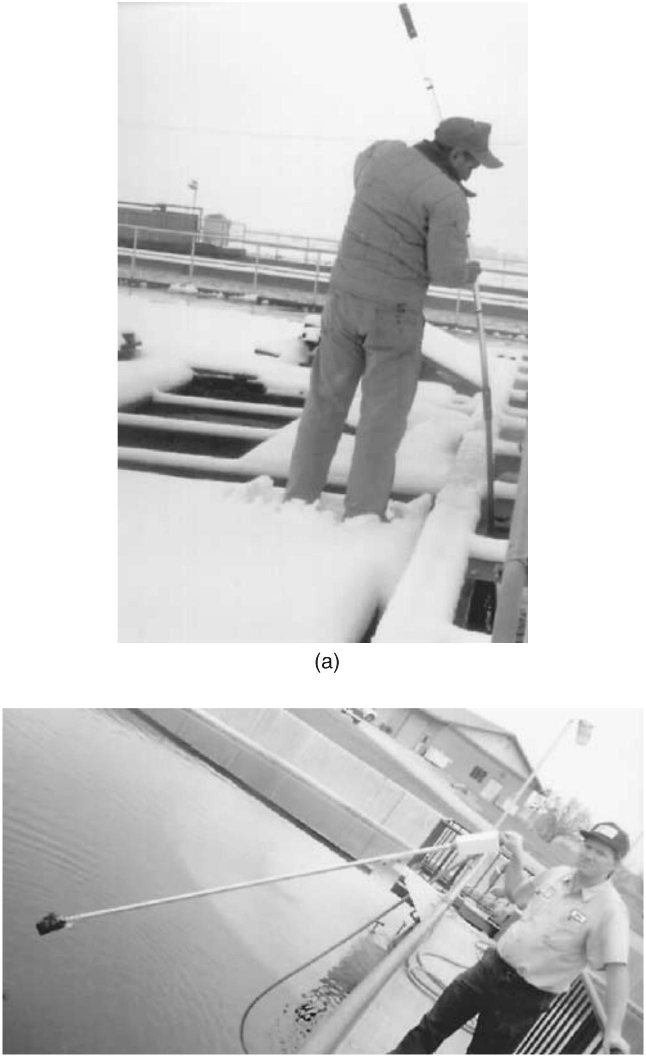
With respect to suspended-growth secondary wastewater treatment systems, solids are being stored through the single-phase concentration mechanism whenever there is an increase in the mixed-liquor suspended solids concentration. For wastewater treatment plants to take full advantage of the single-phase concentration storage capacity of the aeration basin, as well as the two-phase concentration storage capacity of the secondary sedimentation tank, both unit operations must be operated in concert to ensure protection of effluent water quality. In other words, since there is a finite solids storage capacity and a limiting solids transmission rate associated with a properly operating secondary sedimentation tank, both the solids loading rate and solids concentration entering the sedimentation tank will affect two-phase concentration storage capacity [37]. Exceeding the storage capacity of the secondary sedimentation tank will result in deterioration of effluent water quality. A rational design approach to optimize the solids storage capacity in secondary sedimentation facilities may be developed by employing the limiting solids flux theory [27].

Like activated-sludge systems, sedimentation tanks that follow fixed-film secondary treatment systems (e.g., trickling filters, rotating biologic contactors) also may be used to store solids. In all cases, blanket levels must be monitored to ensure that sludge storage levels are not being exceeded within the secondary sedimentation tanks (Fig. 5.34).

5.5.1.2 Storage within sludge treatment processes. Solids can be stored within sludge treatment processes with fewer potential adverse effects on water quality than if they were stored within the wastewater treatment system. The principal sludge processing operations that may be employed for effective solids storage include (1) gravity thickeners, (2) anaerobic and aerobic digesters, (3) air-drying beds (e.g., sand-drying bed, freeze-assisted sand-drying beds, reed beds, and paved beds), (4) lagoons, and (5) compost piles. Used judiciously, these processes can store solids for considerable lengths of time.

The goal of gravity thickeners is to separate liquid from primary and secondary sludge solids. The primary method of solids storage in these units is two-phase concentration storage. The concentrated sludge may be stored within the gravity thickeners for several days (i.e., equalization storage) without production of noxious odors if temperatures are moderate and chemicals that reduce septicity are applied.

In contrast to gravity thickeners, anaerobic digesters may provide storage using all three storage mechanisms. Anaerobic digesters equipped with floating covers have the flexibility to store sludge volumes equivalent to 20 to 125 percent of their design liquid volume by displacement storage when allowing the floating cover to rise (Fig. 5.35). Alternatively, the digester influent can be thickened to a higher solids content, thereby increasing storage capacity by the single-phase



(a)

(b)

Figure 5.34 Photographs of (*a*) an operator monitoring sludge blanket level in a secondary sedimentation tank with a transparent plastic tube; (*b*) an operator using an ultrasonic system to continuously monitor sludge blanket depth.

Figure 5.35 An anaerobic digester equipped with a floating cover.

concentration mechanism [34]. Secondary fixed-covered digesters can be used for two-phase concentration storage by means of liquid-solids separation as well as displacement storage. However, fixed-cover digesters must be protected from excessive vacuum or pressure conditions whenever an attempt is made to achieve displacement storage. Another important advantage of employing anaerobic digesters for solids storage is that they can be used to equalize peak sludge loadings, enabling downstream dewatering processes to operate at their optimal conditions. Example 5.5 illustrates the use of digester storage capacity to handle peak solids loadings.

Example 5.5 The Wallace County Water Reclamation Facility is a combined sewer wastewater treatment facility that currently employs a series of belt filter presses to dewater its primary sludge as well as a floating roof anaerobic digester to achieve solids stabilization. Under normal operating conditions, the primary sedimentation tanks produce a sludge with a solids content of 4 percent, whereas the belt filter presses typically process a solids loading rate of 90,000 lb/day from the digester (which is 60 percent of the maximum solids loading capacity of the belt filter press, i.e., maximum loading is 150,000 lb/day). From historical records, the peak 5-day solids loading during a storm event is 220,000 lb/day. To protect the dewatering system from being overloaded, what possible measures can be taken by the plant operator during significant storm events?

solution

Step 1. The key operational objective during the storm event is to prevent overloading of the belt filter presses. The steps that the operator can take to accomplish this objective include the following:

- Thicken the underflow solids concentration from the primary clarifiers from 4 to 6 or 7 percent by allowing the sludge to accumulate to greater depths in the cross-collection troughs and sludge hoppers. The increase in solids content of sludge discharged from the primary clarifiers increases the single-phase storage capacity of the anaerobic digester.
- Allow the floating covers of the anaerobic digesters to rise to increase the displacement storage capacity.

Step 2. Solids can be stored within the digesters so that during peak solids loading periods, the dewatering capacity of the belt filter presses is not exceeded. The accumulated solids can be released from the digesters after the storm has passed without overwhelming the dewatering system.

Unlike anaerobic digesters, the principal storage mechanism for aerobic digesters is single-phase concentration storage [37]. While discharging a thicker sludge to the aerobic digester is one approach for using single-phase concentration storage, moisture evaporation is also effective in increasing this type of storage capacity. To achieve two-phase concentration storage through sedimentation, the highly agitated contents must be made quiescent to allow the solids to settle and facilitate decanting of the supernatant. As long as aerobic conditions are maintained throughout the digester, such storage is possible. However, if the settling characteristics of the solids are poor, the dissolved oxygen is depleted before effective liquid-solids separation occurs. Under these circumstances, the digester becomes anaerobic and may generate nuisance odors [37].

When displacement-type storage is attempted within an aerobic digester, the liquid level within the system must either rise or fall. Under these conditions, the aeration and mixing systems should automatically adjust to the volume changes to ensure that anaerobic conditions do not occur. Aerobic digesters equipped with floating mechanical aerators or fixed, bottom-mounted diffusers are capable of modifying their rate of oxygen transfer to accommodate the new liquid-volume requirements. Fixed mechanical aerators do not have this capability. Because of the energy requirement associated with maintaining aerobic conditions, long-term storage in aerobic digesters will have relatively low capital but high operating costs [28,34].

Drying beds are used extensively for solids storage by many smaller wastewater treatment plants that employ digestion to achieve sludge stabilization [45]. Drying beds are operated on a fill and draw basis and are often used to provide two-phase concentration and displacement-type storage. Drying beds that lend themselves to sludge storage are described in Sec. 5.7.

Composting is one of the wastewater solids treatment processes that has inherent long-term solids storage capacity. The principal mechanism for solids storage during composting treatment includes two-phase concentration and displacement. With temperatures in the compost piles reaching levels as high as 140°F (60°C), significant amounts of moisture are removed through evaporation (two-phase concentration storage) [37]. As the composting stabilization process proceeds, there is also a loss of volatile matter, resulting in an overall mass reduction. Finally, as the individual pile size decreases, stabilized compost can be combined into larger piles, allowing the systems to use displacement-type storage. It should be noted that once the composting process has stabilized the wastewater solids, the curing step can be extended as long as solids storage is necessary.

5.5.2 Dedicated systems for liquid biosolids/sludge storage

Dedicated liquid storage facilities consist primarily of industrial-type holding tanks, above-ground (i.e., diked) storage containment systems, or lagoons. Holding tanks are commonly provided as an integral part of most stabilization, conditioning, and dewatering operations (Fig. 5.36). Holding tanks are small relative to either diked containment or lagoon systems, with detention times measured in hours instead of days.

If the holding tank is located downstream from certain sludge treatment processes, special precautions may be required. For example, if the tank is located downstream from an anaerobic digester and

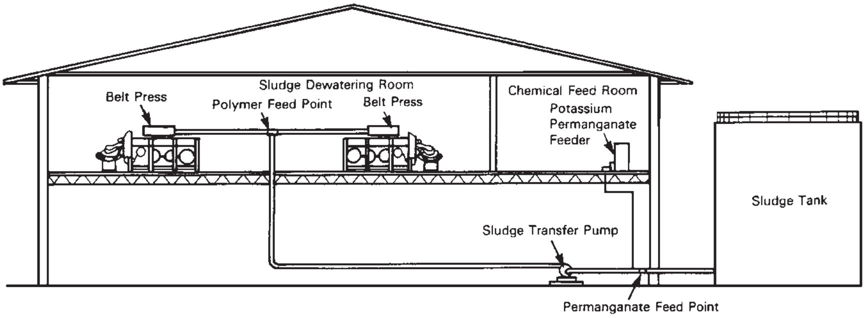


Figure 5.36 Schematic diagram of a sludge holding tank used for equalization storage prior to sludge dewatering. Note that potassium permanganate is being employed for odor control.

there are plans for more than a few hours of storage, the tank must be equipped to collect and remove combustible digester gas. Holding tanks may be used for blending different materials as well as for equalization storage. Holding tanks also provide storage for liquid sidestreams from various treatment processes such as thermal conditioning [37].

Holding tanks that are used for blending must be maintained in a homogeneous condition through employment of mechanical impellers, hydraulic recirculation, or gas agitation (Fig. 5.37). Such tanks can provide either single-phase concentration or displacement storage [37].

Whatever its function, holding tanks must be designed to manage or eliminate the production of malodorous gases. This operational requirement is especially difficult when the holding tank must both provide equalizing storage and operate on a batch basis (Fig. 5.38). Unless the solids supplied to the holding tank are completely stabilized, the tanks used for extended periods of storage (e.g., beyond a few days) will result in the creation of nuisance odors. Providing even short periods of storage (e.g., few hours) for unstabilized primary or secondary sludges in a holding tank can result in the production of

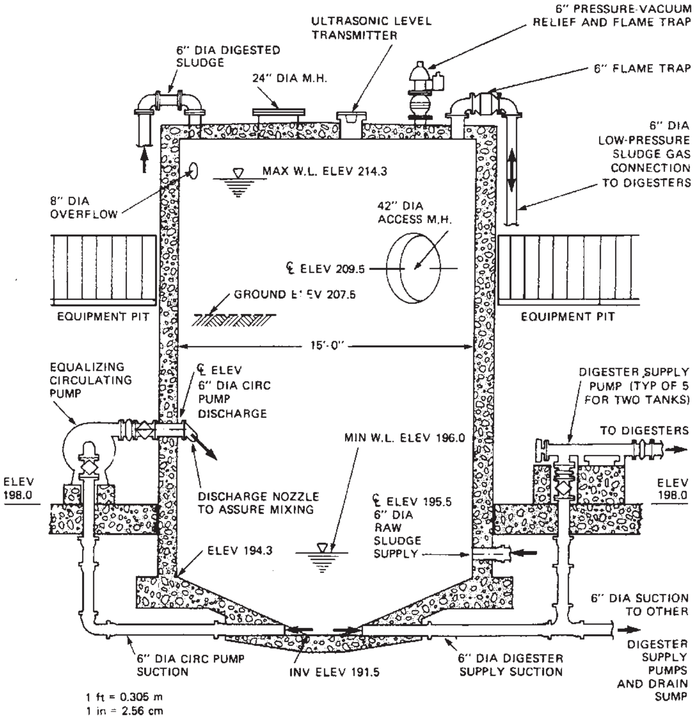


Figure 5.37 Schematic diagram of a sludge holding tank.

Figure 5.38 Photograph of a sludge holding tank.

nuisances odors if no provisions are made to either inhibit odor generation or capture and treat the malodorous gases. There are various approaches to managing the odorous gases from sludge holding tanks including exhausting the gases through (1) the wastewater aeration tank, (2) an activated-carbon filter, (3) a chemical scrubber, or (4) an incinerator.

To protect downstream sludge treatment processes, holding tanks must be designed with sufficient capacity to accommodate the varying sludge production rates. Therefore, a rational approach for holding tank design must account for the sustained peak volumetric sludge flow rates that occur at a particular wastewater treatment plant. Unfortunately, in most cases, only the daily average sludge rate at the wastewater treatment plant is known and not the peaking factors. The procedure for developing the peaking factors is described in Table 5.9. Example 5.6 illustrates the design approach for estimating the minimum volume necessary for sludge storage tanks, assuming that appropriate peaking factors have been estimated.

TABLE 5.9 Necessary Steps to Estimate Sludge Volume Peaking Factors*

Step 1. Estimate the average daily sludge flow rate (gal/day) from the annual sludge production rate (may be obtained from records of sludge pump activity).
Step 2. Review the daily sludge (or pumping) records to find the highest 1-day sludge flow rate (gal/day).
Step 3. Divide the highest 1-day sludge flow rate by the average daily sludge flow rate to obtain the peaking factor for the 1-day sustained flow.
Step 4. Search the daily sludge (or pumping) records to find the largest cumulative 2-day sludge flow (gal). Average the flows to obtain the 2-day sustained sludge flow rate (gal/day).
Step 5. Divide the 2-day sustained sludge flow rate by the average daily sludge flow rate to obtain the peaking factor for the 2-day sustained flow.
Step 6. Repeat the procedure until all peaking factors are found for the period of interest (for sludge holding tanks this is normally 10 days or less).

*Adapted from ref. [37].

Example 5.6 Carver County Water Reclamation Plant is using belt presses to dewater its thickened sludge. Based on the annual total thickened sludge production, the average daily thickened sludge production rate is 18,000 gal/day. If the belt presses are to be operated only 2 days a week (Thursday and Friday), estimate the minimum volume (in gallons) of the sludge storage tank and the required sludge-processing rate (gallons per day) over the 2-day period. Assume that the following peaking factors have been determined from analysis of the daily sludge volumetric flow rates (gallons per day). Note that the peaking factor is the ratio of the sustained peak loading divided by the average loading.

Length of sustained peak, days	Peaking factor
1	2.0
2	1.6
3	1.4
7	1.3
10	1.2

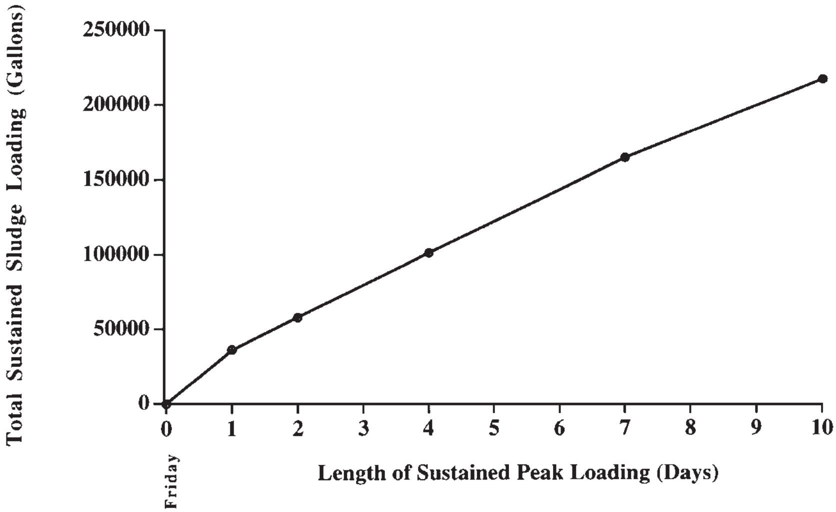
solution

Step 1. Compute the peak volumetric sludge loadings based on the annual average daily flow rate.

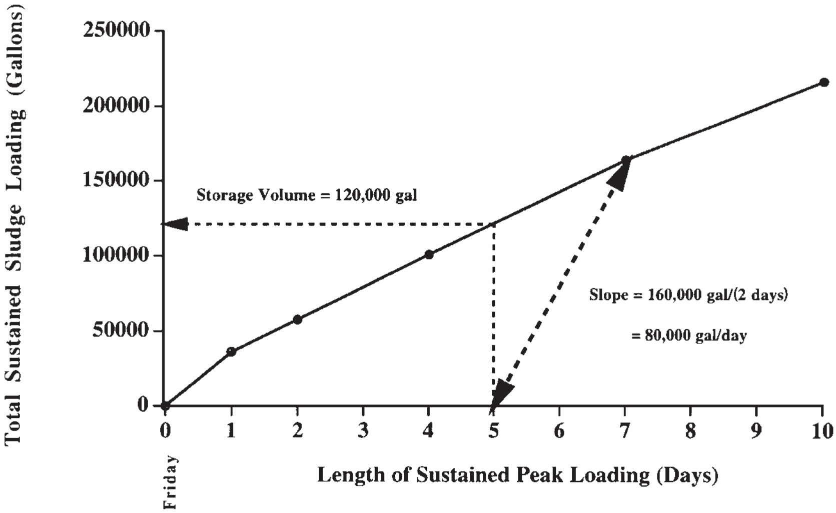
Length of sustained peak, days	Peaking factor	Peak volumetric loading, gal/day*	Total sustained loading, gal†
1	2.0	36,000	36,000
2	1.6	28,800	57,600
4	1.4	25,200	100,800
7	1.3	23,400	163,800
10	1.2	21,600	216,000

*Values are 18,000 gal times the peaking factor.
†Values are peak volumetric loading (gal/day) times the length of sustained peak (days).

Step 2. Plot the sustained volumetric loading versus length of sustained peak assuming that, at the end of the workweek (Friday), the sludge storage tank is empty.



Step 3. Estimate the storage volume by noting that 5 days of storage must be available (Saturday through Wednesday) prior to operation of the belt presses beginning on Thursday.



From the preceding figure, the minimum storage volume is approximately 120,000 gal. The sludge processing rate (i.e., throughput for the belt presses) is the total volume accumulated over the 7-day period divided by the 2-day processing time, i.e., 160,000 gal divided by 2 days, or 80,000 gal/day.

In addition to volume requirements, material management and odor control are critical holding-tank design concerns. No materials should be allowed in the tank that could incapacitate the mixing mechanism. Moreover, the pumps used to mix and/or discharge the contents of the sludge holding tanks should be low-efficiency, nonclog types [37]. Often decant and sludge-conditioning tanks operate at detention times sufficient to generate significant amounts of odorous gases. In such cases, provisions must be made to confine and treat the odorous gases to a level at which their discharge ceases to create a nuisance [43].

5.5.2.1 Earthen structures for liquid biosolids/sludge storage. Both above-ground (i.e., diked containment) and below-ground (i.e., lagoons) storage structures may be used for storage of liquid biosolids/sludge. Moreover, the general design approach for earthen containment structures is similar to the approach for structures used for surface disposal. The critical difference between these two types of earthen structures is that the systems that serve as surface disposal sites are subject to the surface disposal provisions of the 40 CFR Part 503 rules [66]. For example, lagoons used for long-term temporary storage of sludge are not subject to the regulatory requirements of 40 CFR Part 503 if the storage is considered part of the treatment process (e.g., dewatering) and if the facility's owner/operator has a plan for final use or disposal of the sludge. If, however, the facility's owner/operator has no intention of ever removing the sludge from the lagoon, the facility is considered a surface-disposal site and is subject to the 40 CFR Part 503 surface-disposal requirements that include pollutant limits, closure, management practices, pathogen and vector attraction reduction, monitoring, recordkeeping, and reporting [65,66].

5.6 Lagoon Systems

Lagoons are used extensively for the storage of biosolids/sludge of various qualities and moisture contents. Basically, three types of lagoon systems are used for liquid sludge storage: (1) facultative sludge lagoons, (2) anaerobic liquid sludge lagoons, and (3) sludge-drying lagoons. For the design of all types of lagoon systems, groundwater protection is a principal concern. A minimum soil buffer of 4 ft is recommended between the bottom of a lagoon and the seasonal annual high groundwater table [2,6,37]. Liners and leachate collection systems should be considered, depending on sludge quality, distance to drinking water wells, depth to groundwater, groundwater flow direction and velocity, aquifer classification, and underlying soil type and permeability [16]. Each of the principal lagoon sludge storage systems is described in the following sections.

5.6.1 Facultative sludge lagoons

Facultative sludge lagoons are sludge storage facilities that operate in conjunction with anaerobic digesters. These systems do not function properly (without major environmental impacts) when supplied with either unstabilized or aerobically digested sludge [28]. Facultative sludge lagoons are designed to maintain an aerobic surface layer free of scum or membrane-type film accumulation. The aerobic layer is achieved and maintained by limiting the organic loading to the lagoon together with providing daily agitation and mixing of the aerobic surface layer [37].

The aerobic surface layer is usually from 1 to 3 ft (0.3–0.9 m) deep and supports a dense population of algae (5×10^4 to 6×10^6 organisms/ml, usually *Chorella*). Dissolved oxygen is supplied to this layer by (1) algal photosynthesis, (2) atmospheric diffusion, and (3) atmospheric oxygen entrainment mediated by surface mixers [68,69]. The dissolved oxygen is consumed by aerobic microorganisms during the metabolism of dissolved and colloidal organic matter from the digested sludge liquor. Aerobic microorganisms eventually settle to the bottom of the lagoon and undergo anaerobic decomposition.

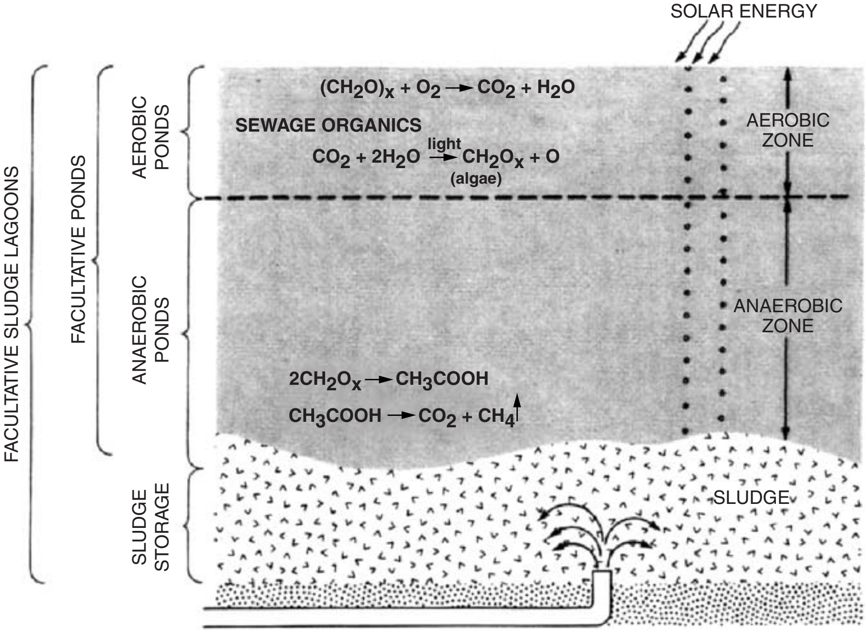


Figure 5.39 Schematic diagram of a facultative sludge lagoon.

The nutrient and carbon dioxide released during both the aerobic and anaerobic metabolism of organic matter are consumed by the algae in the cyclic-symbiotic relationship (Fig. 5.39). This relationship maintains the pH of the facultative sludge lagoon surface layer in a range between 7.5 and 8.5, which effectively minimizes the emission of any hydrogen sulfide (H_2S).

5.6.1.1 Design criteria. Design considerations for facultative sludge lagoons include (1) area organic loading rate, (2) surface agitation requirements, (3) operational considerations, (4) dimensional and layout limitations, and (5) physical factors. Each of these empirical design factors is discussed in the following sections.

Area loading rate. In general, to maintain an aerobic surface layer, the organic loading rate to facultative sludge lagoons should be no more than 20 lb volatile solids (VS) per 1000 ft^2 per day (1 metric ton VS/hectare·day). However, some lagoons have been found to be capable of receiving up to four times the normal daily organic loading as long as they have had 3 days of rest between loadings. Experiments on small basins loaded to failure indicated that peak loadings as large as 90 lb VS per 1000 ft^2 per day (4.4 mt VS/ha·day) could be tolerated during the summer and fall as long as the rate was not sustained for more than 1 week [37].

Surface agitation requirements. Experiments on facultative sludge lagoons continuously fed at 20 lb volatile solids (VS) per 1000 ft^2 per day have indicated that these systems cannot function properly without effective surface agitation [37]. The major operational problem resulting from the absence of surface agitation is the formation of a surface film. If the surface film is not dissipated, an important mechanism for oxygen transfer to the lagoon is eliminated. Field testing of mixing equipment has indicated that a brush-type surface mixer is effective in eliminating the surface film without causing excessive mixing of the aerobic and anaerobic zones (Fig. 5.40).

To maintain adequate oxygen transfer, facultative sludge lagoons with surface areas of from 4 to 7 acres (1.6–2.8 ha) require the use of at least two surface mixers operating from 6 to 12 hours per day [28,37]. Brush-type mixers should be designed to include an 8-ft-long (2.4-m) rotor turning at approximately 70 rpm 12 hours a day. Lagoons with surface areas of much less than 4 acres should be able to achieve the same operational results with two mixers equipped with 6-ft-long (1.8-m) rotors driven by 5-hp (3.7-kW) motors operating 12 hours per day. The benefit of having two mixers per facultative sludge lagoon is that it ensures maximum surface film and scum breakup. Operation of

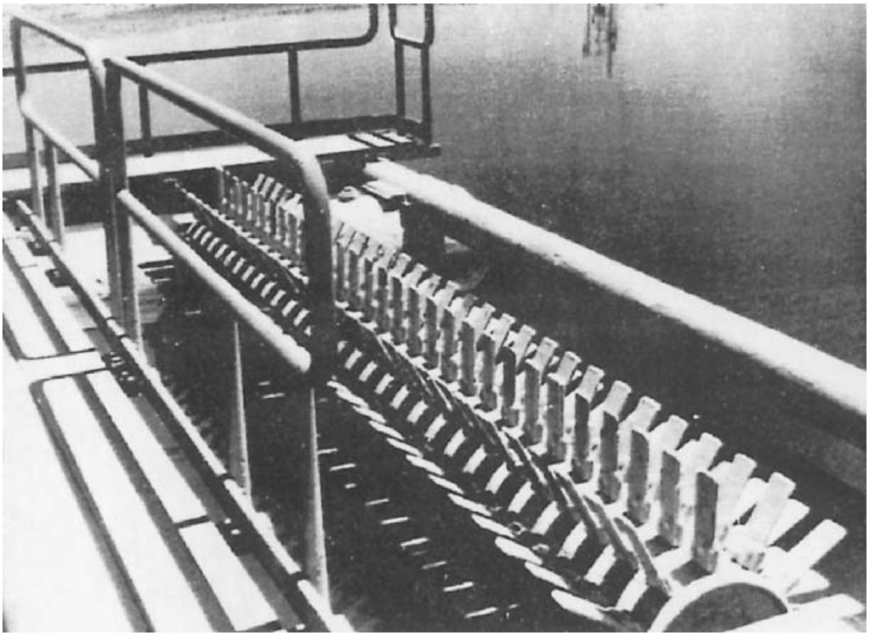


Figure 5.40 Photograph of a brush-type surface mixer. (*Adapted by permission from ref. [28].*)

surface mixers should not coincide with facultative sludge lagoon loading and always should occur during the hours of minimum human exposure (e.g., 6 P.M. to 6 A.M.) and favorable atmospheric conditions. The agitation and mixing effects of the two mixers located at opposite ends or sides of a lagoon also act to maintain equal distribution of the discharged anaerobic solids.

Although brush-type floating-surface mixers provide the best performance, submerged pump-type floating aerators may be required if sludge application under freezing conditions is required.

Operational considerations. Operational considerations for effective employment of facultative sludge lagoons can be divided into three categories: sludge loading, routine operations, and sludge removal. Facultative sludge lagoons should be filled initially with wastewater effluent. Ideally, the effluent should have from 3 to 6 weeks for development of an aerobic surface layer prior to introduction of the anaerobically digested sludge. All facultative sludge lagoons should be loaded daily with the discharged sludge distributed equally between all individual systems. Moreover, facultative sludge lagoons should be loaded only during periods of favorable atmospheric conditions to maximize odor dispersion.

Shock loadings of sludge (such as that which occurs during periodic digester cleaning) should be distributed to all operating facultative sludge lagoons in proportion to the current quantity of their sludge inventory (Fig. 5.41). The sludge blanket in a lagoon should not be allowed to rise higher than 2 ft below the operating water surface. Care must be taken in returning facultative sludge lagoon supernatant to the wastewater treatment plant headworks to avoid shock loadings of ammonia.

Facultative sludge lagoons that are scheduled to be emptied of accumulated solids should be removed from routine operation at least 30

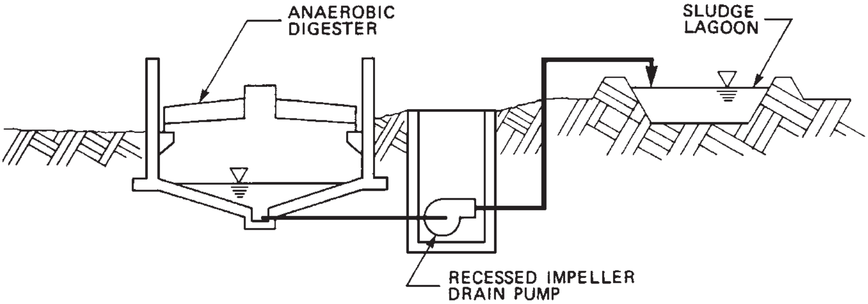


Figure 5.41 Schematic diagram of a digester discharging to a lagoon during tank cleaning operations.

days prior to removal of any solids. Sludge removal should be limited to those facultative sludge lagoons which have concentrated the solids to at least 6 percent [37]. During facultative sludge lagoon solids-removal operations, the water surface level should not be allowed to drop by more than 12 to 18 in (30–46 cm) below its normal operating level [28,37].

Dimensional layout. The maximum area for a single lagoon is based on the practical size for loading, surface agitation, mixing, and sludge removal (Fig. 5.42). The design of large 4- to 7-acre (1.6–2.8 ha) individual lagoons is appropriate only for wastewater treatment plants that operate over 70 acres (28 ha) of facultative sludge lagoon area. Facultative sludge lagoons as small as 150 ft (45.7 m) on a side have been operated successfully [37].

Lagoon depths typically range from 11.5 to 15 ft (3.5–4.7 m). If surface agitation is maintained by submerged-pump-type aerators, it may be necessary to use the deepest lagoon possible to ensure adequate separation between aerobic and anaerobic settling zones. Facultative sludge lagoons are best designed to have a long and short dimension, with the shortest dimension oriented parallel to the direction of the maximum prevailing wind. The longer side is made conducive to efficient dredge operation, whereas having the short sides parallel to the prevailing wind directly helps to minimize wave erosion on the surrounding levees.

When the total cumulative area of facultative sludge lagoons exceeds 40 acres (16.2 ha), the potential for significant odor emission

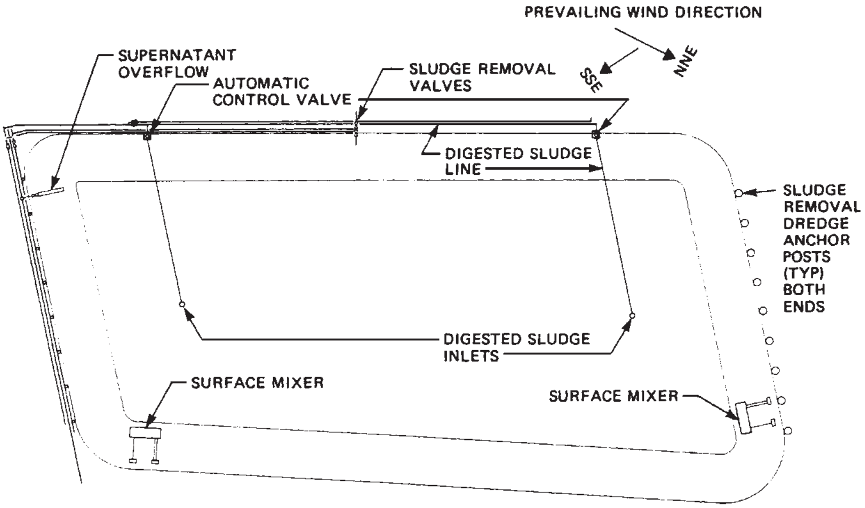


Figure 5.42 Plan view of a facultative sludge lagoon.

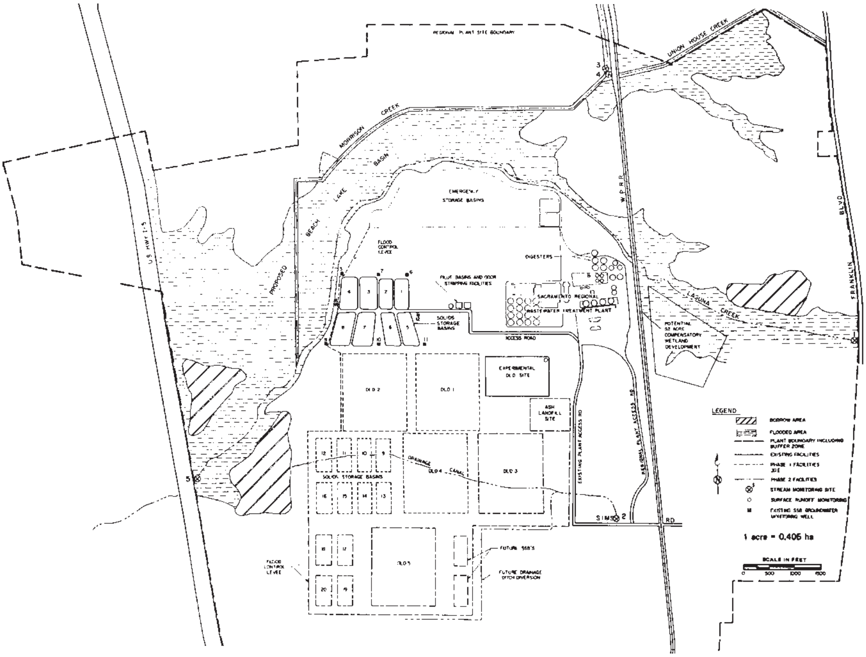


Figure 5.43 A 124-acre facultative sludge lagoon facility, Sacramento, California.

must be considered. Figure 5.43 depicts the layout for the 124 acres (50.2 ha) of facultative sludge lagoons in Sacramento, California. The physical orientation of this particular facility was based on the need to minimize the odor risk to the surrounding area. Table 5.10 provides the design criteria for these lagoon systems.

Physical considerations. Facultative sludge lagoon dikes should have slopes of 1:3 (vertical to horizontal), with adequate riprap provided in the working zone of the surface level (Fig. 5.44). Sufficient free board also must be provided to protect against any conceivable overtopping of the dikes.

To facilitate loading of the facultative sludge lagoon, two anaerobic digester sludge feed lines should be provided, each with its own automatic valve. This design feature ensures adequate distribution of solids over the whole lagoon basin. Digester sludge feed pipelines should be located directly below the bottom of the lagoons, with the inlet surrounded by a protective concrete surface. Surface mixers should be located downstream of the prevailing winds, while the harvested sludge dredge hookup should be located centrally (Fig. 5.45). All piping within the basin should be out of the way of any future dredging operations.

TABLE 5.10 Design Criteria for Sludge Storage Basins, Sacramento, California*

Total number of storage basins	20
Surface area, hectares (acres)	50.6 (125)
Depth at normal operation, m (ft)	4.57 (15)
Solids loading rate, kg/m^2·d (lb/1000 ft^2·d)	0.0975 (20)
Stored solids concentration (%)	>6%
Surface mixers for aeration	40
Barrier wall height, m (ft)	3.64 (12)

*Adapted from ref. [37].

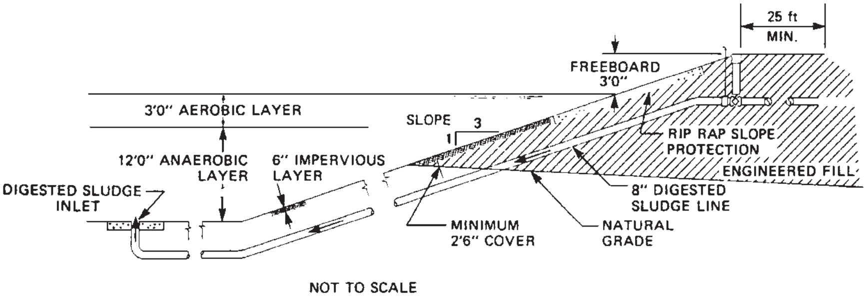


Figure 5.44 Cross-sectional view of facultative sludge lagoon basin design.

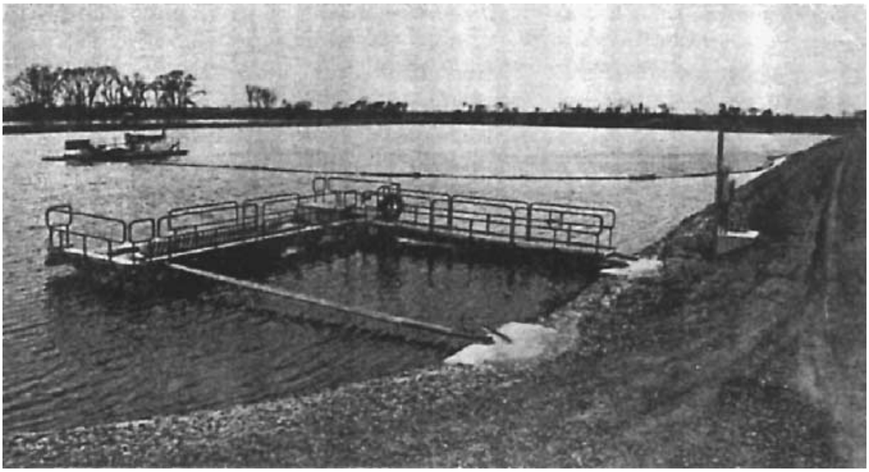


Figure 5.45 Photograph of a facultative sludge lagoon. Note the surface mixer in the foreground. (*Adapted by permission from ref. [28].*)

Supernatant withdrawal from a facultative sludge lagoon normally is located upstream from the prevailing winds to minimize scum accumulation. Because of its aqueous chemistry, facultative sludge lagoon supernatant typically precipitates magnesium ammonia phosphate (struvite) on any rough surface that is not completely submerged [37]. This crystalline material can completely clog cast iron fittings and pump valves when their surfaces undergo a wet-dry cycle. The most practical approach to minimizing operational problems associated with struvite formation has been the use of polyvinyl chloride (PVC) piping throughout the facultative sludge lagoon supernatant transport system, as well as to design the gravity return of supernatant to the wastewater treatment plant headworks with a minimum of critical depth conditions [37]. If pumping of supernatant is required, submerged slow-speed, nonclog centrifugal pumps with low suction and discharge velocities will be the most trouble-free [19]. All pipeline and pipeline appurtenances that are not made from PVC or another smooth, nonmetallic material should receive a smooth, impervious internal surface coating [19,37].

5.6.2 Anaerobic liquid sludge lagoons

An anaerobic liquid sludge lagoon is an open structure similar to a facultative sludge lagoon but with a greater depth in relation to surface area. These lagoons precipitate solids with a higher specific gravity than water and provide long-term sludge storage. Unlike the facultative sludge lagoon, an aerobic surface layer is not maintained in these systems, and floatable material is not settled or removed. The elimination of an aerobic surface layer results in the development of a thick scum layer on the lagoon surface (Fig. 5.46).

Sludge loading rates to anaerobic liquid sludge lagoons are significantly higher than the loading rates of facultative lagoons. Figure 5.47 depicts the layout of four anaerobic liquid sludge lagoons at the Metropolitan Sanitary District of Greater Chicago [37].

5.6.3 Sludge drying lagoons

Until recently, sludge drying lagoons often were located in soils with at least moderate permeability to take advantage of subsurface drainage and percolation. This practice is now the exception rather than the rule in the United States because of the more stringent environmental and groundwater protection regulations. If a groundwater aquifer with drinking water potential exists beneath the site, the lagoon may have to be lined to restrict any significant percolation of leachate. Unless a sand bottom and underdrains are installed, the

Figure 5.46 Photograph of the scum layer covering an anaerobic liquid sludge lagoon.

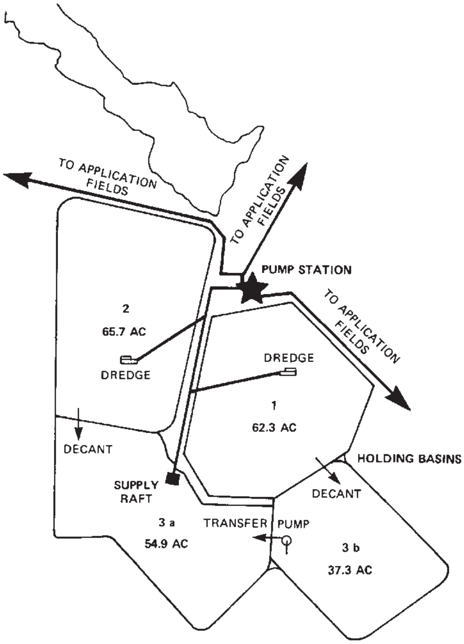


Figure 5.47 Layout of anaerobic liquid sludge lagoons for the Metropolitan Sanitary District of Greater Chicago.

TABLE 5.11 Advantages and Disadvantages of Sludge Drying Lagoons for Storage*

Advantages	Disadvantages
Low energy costs	May be source of periodic odors
Low chemical costs	Potential to contaminate groundwater
Not susceptible to sludge variability	May create a vector problem
Lagoons act as a buffer in case of shock loading to plant	More visible to the public
Organic matter is further stabilized	Land intensive
Require least amount of operator attention and skill	
Low capital cost if land is available	

*Adapted from ref. [37].

principal sludge dewatering mechanisms for these systems include decanting supernatant and evaporation.

Sludge drying lagoons are similar in operation to sludge drying beds in that decantation and evaporation are the principal mechanisms for water removal [45]. However, in sludge drying lagoons, the sludge is placed at depths three to four times greater than it would be in a drying bed [28]. Generally, sludge placed in a dry lagoon is allowed to dewater to some predetermined solids concentration before removal.

To minimize the production of obnoxious odors, sludge should be stabilized prior to addition to drying lagoons. Occasional odors, flies, and mosquitoes may still be a problem, so choosing a remote construction site is essential. Finally, it is important to note that lagoons can produce nuisance odors as they go through a series of wet and dry conditions. Advantages and disadvantages of using sludge drying lagoons are summarized in Table 5.11.

5.6.3.1 Design criteria. In effect, the sludge drying lagoon is similar in concept to a deep sand drying bed with restricted drainage. The type of sludge to be placed in the sludge drying lagoon can significantly affect the amount and type of odor and vector problems that may be produced. To minimize adverse environmental impacts, it is recommended that only anaerobically digested sludge be loaded into the drying lagoons. A minimum of two lagoon cells is essential, even for small wastewater treatment facilities, to ensure availability of storage space during cleaning, maintenance, or emergency conditions.

Evaporation and decanting are the dominant pathways for moisture removal even if a drainage network exists. The amount of moisture to be removed for the sludge lagoons to achieve a desired sludge moisture content includes the required portion of the sludge moisture content plus that portion of the local precipitation that infiltrates the sludge rather than being removed as supernatant. The dependence on evaporation tends to favor the construction of these systems in arid and semiarid areas. However, even in areas with appreciable rainfall, it is possible to accelerate the drying process by employing a tractor-mounted auger to remove the surface crust and expose the wet material below.

The required lagoon surface area depends on local climatic conditions including the temperature and the precipitation and evaporation rates. The design equations applicable for sludge drying lagoons are identical to those used for paved drying beds (see Sec. 5.7.4).

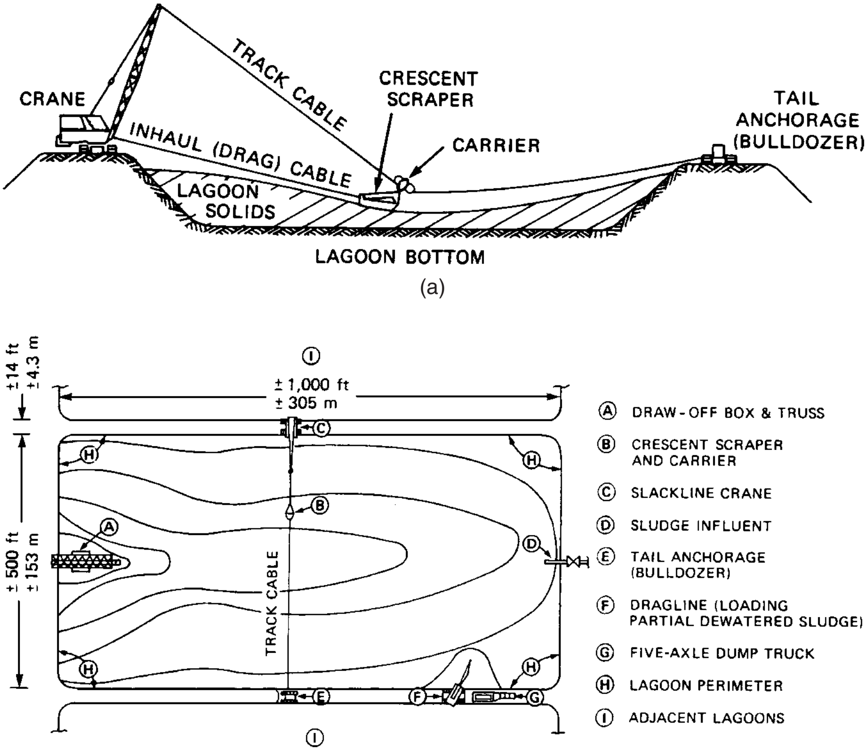
Structural elements. Sludge drying lagoons consist of retaining walls that are normally earthen dikes 2 to 4 ft (0.7–1.4 m) high. The earthen dikes usually enclose a rectangular space with a permeable subsurface. Appurtenant equipment includes (1) sludge feed lines and metering pumps, (2) supernatant decant lines, and (3) some type of mechanical sludge-removal equipment. Mechanical sludge-removal equipment can include trucks, front-end loaders, bulldozers, or drag lines depending on the size of the operation.

The actual depth and area requirements for sludge drying lagoons depend on several factors such as precipitation, evaporation, type of sludge, volume, and solids concentration. Solids loading criteria have been reported to be in the range of 2.2 to 2.4 lb of solids per year per cubic foot (36–39 $kg/m^3 \cdot yr$) of capacity.

Lagoons may be of any shape, but a rectangular shape facilitates sludge removal. Lagoon dikes should have a slope of 1:3 (vertical to horizontal) and should be of a shape and size to facilitate maintenance, mowing, passage of maintenance vehicles atop the dike, and access for the entry of trucks and front-end loaders into the lagoon. Provisions also must be made for limiting public access to the facility.

Performance expectations. Solids concentrations in the range of 15 to 40 percent are expected in the sludge removed from the lagoons (with higher solids contents possible in arid climates). Sludge drying lagoons share a common problem with other air-drying dewatering processes, and this is that a surface crust forms early in the evaporation stage that restricts further moisture removal. This problem is minimized with paved drying beds that use mechanical equipment to break up the surface crust (Sec. 5.7.4). Similar mechanical equipment and procedures can be used in sludge drying lagoons if sludge depth permits. Floating devices also can be used. Larger facilities sometimes employ a cable and scraper to remove the surface crust (Fig. 5.48).

Dike integrity. Earthen dikes and slopes are vulnerable to stability failures, releasing leachate to surrounding soils and groundwater. For this reason, earthen dikes must be designed carefully, and excavated slopes must be evaluated carefully to ensure that they are sufficiently



(a)

(b)

Figure 5.48 Sludge drying lagoon for West-Southwest Sewage Treatment Works, Chicago. (*a*) Cross-sectional view. (*b*) Plan view. Note the drag line for removal of surface crust and dewatered sludge.

stable to withstand the loading and hydraulic conditions to which they will be subjected during the unit's construction and operation. For more information on slope stability and dike integrity for sludge storage facilities, including information on material specifications and embankment construction, the reader is directed to refs. [2,6,16,70].

Operating and maintenance procedures. Regardless of the size of the sludge drying lagoon, the system will have similar operating procedures. The routine operational activities consist of sequential sludge applications and decanting until the lagoon contains the design volume of sludge. Once the evaporation stage begins, operational steps include the periodic breakup or removal of surface crust to ensure continued moisture evaporation. Operating procedures common to all sizes of sludge drying lagoons are summarized in Table 5.12.

Maintenance activities include preventive maintenance care for equipment and dikes, as well as the control of dike vegetation. Some

TABLE 5.12 Operational Procedures for Sludge Drying Lagoons*

1. Stabilized liquid sludge is pumped into the lagoon until a depth of 24 to 48 in (0.7–1.4 m) is achieved.
2. The discharged sludge is allowed to settle, and the supernatant formed is decanted either continuously or intermittently from the lagoon surface and returned to the headworks of the wastewater treatment plant.
3. Additional liquid sludge is added until the lagoon has achieved a desired sludge depth, after which the solids are permitted to dewater through moisture evaporation. The surface crust that forms must be removed periodically to ensure that moisture evaporation is not limited. Depending on the climate and the depth of applied sludge, the time necessary to reach a final solids content of between 20 to 40 percent may be from 3 to 12 months.
4. After the desired solids content is reached, the dewatered sludge is removed using mechanical equipment.
5. After the lagoon is emptied, the cycle may be repeated.

*Adapted from ref. [37].

sludge drying lagoons may require insect and odor control. Other possible maintenance activities include the repair of dikes, sealing of the lagoon bottom (if required), and underdrain maintenance (if used).

5.6.4 Aerated storage basins

Aerated storage basins are earthen containment structures operated similarly to a semibatch aerobic digester system. Stabilized solids from anaerobic or aerobic digesters are pumped directly into the storage basin. During the filling operation, the contents of the basin should be aerated continuously. The time required to fill the basin depends on (1) basin volume, (2) volume of discharged sludge, (3) precipitation rate, and (4) evaporation rate. To use aerated storage basins successfully, the design must meet the following criteria:

1. Basin contents must be mixed sufficiently to ensure uniformity of solids concentration and complete dissemination of oxygen.
2. Sufficient oxygen must be available to maintain aerobic conditions throughout the basin at maximum attainable solids concentration.
3. Liquid level variations must be sufficient to accommodate maximum storage needs under anticipated rainfall.
4. Like all other earthen storage systems, the basin subsurface must be impermeable or equipped with a suitable underdrainage to provide groundwater protection.

Equipment required to aerate storage basins is similar to that used in aerobic digestion. The final oxygen-transfer system design must account for inherent equipment limitations. For example, fixed or floating turbine or propeller-type aerators are often affected by limited side boundaries, whereas the brush-type aerators and aspirating pumps have restricted vertical mixing capabilities. Submerged static aerator devices have excellent vertical mixing but are limited by confined side boundaries. To design the appropriate systems, the equipment supplier should be given information about the configuration of the basins, its liquid-level operating range, maximum solids concentration expected, and the level of dissolved oxygen to be maintained.

Under normal operating conditions, oxygen requirements for aerated storage basins will be considerably less than those of aerobic digesters because the material being stored is typically stabilized prior to its introduction into the basin. A minimum dissolved oxygen level of 0.5 mg/liter is adequate to maintain aerated storage basins free from anaerobic activity. If the sludge discharged to the aerated storage basin is not sufficiently stabilized, the oxygen requirements for the system will be similar to aerobic digester operation [37].

As with aerobic digesters, it is often difficult to achieve two-phase concentration storage within aerated storage basins. Because of poor solids settling characteristics, the time necessary to achieve effective liquid-solids separation is normally too long to avoid the problems associated with oxygen depletion and the onset of anaerobic conditions. Separate continuous decanting systems that achieve liquid-solid separation by sedimentation or dissolved air flotation are often employed in these situations.

Aerobic storage basins that do not have separate decanting facilities must be designed to achieve storage based on either the single-phase concentration or displacement storage mechanism. The single-phase concentration mechanism will function as described for aerobic digesters, with moisture evaporation used primarily to enhance storage capacity. Displacement-type solids storage involves liquid-level variability that requires modification of the oxygen-transfer system to ensure aerobic conditions. To avoid production of nuisance odors, the aeration equipment must be capable of maintaining adequate mixing and oxygen transfer over the complete range of liquid-level variations.

5.7 Storage in a Drying Bed

In addition to lagoon systems, long-term storage of solids can be accomplished within several of the drying processes used to dewater sludge. Drying-bed processes that lend themselves to long-term solids storage include (1) sand drying beds, (2) reed-enhanced sand drying beds, (3) freeze-assisted sand drying beds, and (4) paved drying beds. Other types of drying-bed processes, e.g., vacuum-assisted drying beds and wedge-wire drying beds, are used exclusively for dewatering operations and are not capable of providing long-term solids storage.

Drying beds are suitable for long-term storage because they employ relatively large land areas for dewatering. Drying beds accomplish storage through two-phase concentration (via moisture evaporation and drainage) and displacement mechanisms. With several months of processing time, drying beds can produce a sludge having a solids content that exceeds 60 percent [45]. Since the drying-bed designs are based on generating a final sludge moisture content, the time to achieve the designated moisture content represents the minimum solids storage period. Each of the principal drying-bed dewatering processes is described in the following sections. If, in addition to dewatering, it is desired to employ these systems for sludge storage, the design engineer should design the systems to provide at least 50 percent excess drying capacity [37].

5.7.1 Sand drying beds

Sand drying beds have been used successfully for sludge dewatering for many years and remain the principal method of dewatering used in the United States [45]. Dewatering on the sand drying bed occurs through gravity drainage of free water followed by evaporation (Fig. 5.49). In areas of high precipitation, covered-sand drying beds have been used [37].

5.7.1.1 Design considerations. The critical design parameter for construction of sand drying beds is the surface area required to achieve the final sludge solids content. Water is removed from the sludge through gravity drainage and evaporation. The amount of water that

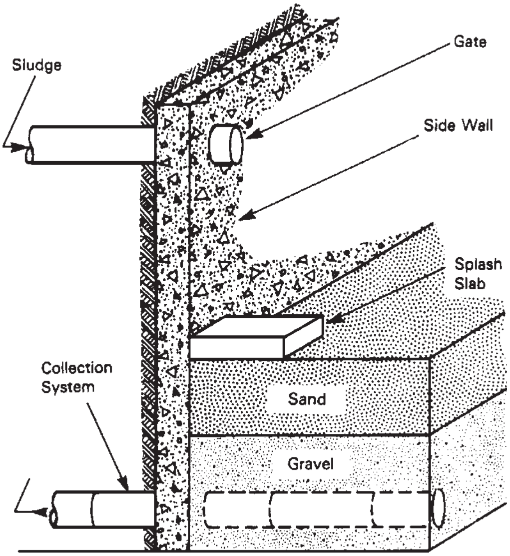


Figure 5.49 Schematic diagram of sand drying-bed construction.

can be removed by drainage is strongly influenced by the type of sludge applied. Drainage may account for 25 percent of the moisture removed during the dewatering of anaerobically digested plus waste-activated sludge and 75 percent for a well-conditioned sludge. The drainage process is typically complete within 3 to 5 days of sludge loading to the drying bed.

The rate of evaporation is a function of local climatic conditions and the sludge surface characteristics. Seasonal evaporation rates can be obtained from local pan or lake evaporation values. Since the crust that forms on the sludge surface inhibits evaporation, the pan evaporation values must be adjusted when designing sand drying beds. An adjustment factor of 0.6 was derived experimentally and is often used as a default value in sand drying-bed design [45]. Once cracking of the sludge surface occurs, the evaporation rate will approach the pan value due to the additional sludge surfaces exposed. Equation (5.17) may be used to estimate the drying time for a single sludge application to a sand drying bed.

$$t_d = \frac{y_0\,[1-(s_0/s_f)]\,(1-D)}{k_e E_v} \tag{5.17}$$

where t_d = dewatering time for a single sludge application, months
y_0 = initial depth of applied sludge layer, in (cm)
s_0 = initial dry solids content, % as a decimal
s_f = final dry solids content, % as a decimal
D = fraction of water removed by drainage, decimal
E_v = average pan evaporation rate during time t_d, in/mo (cm/mo)
k_e = reduction factor (% as a decimal) for sludge evaporation relative to free water surface (typically 0.6, or measured value)

The annual number of sludge applications that can be made to the sand drying bed may be estimated by using Eq. (5.18):

$$N = \frac{n_v}{t_d} = \frac{n_v k_e E_{vn}}{y_0\,[1-(s_0/s_f)]\,(1-D)} \tag{5.18}$$

where N = number of sludge applications
n_v = length of operating season or an increment if evaporation is significantly different, months
t_d = dewatering time for a single sludge application, months
y_0 = initial depth of applied sludge layer, in (cm)
s_0 = initial dry solids content, % as a decimal
s_f = final dry solids content, % as a decimal
D = fraction of water removed by drainage, decimal

E_{vn} = average pan evaporation rate during period n_v, in/mo (cm/mo)

k_e = reduction factor (% as a decimal) for sludge evaporation relative to free water surface (typically 0.6, or measured value)

The design solids loading rate to the sand drying bed may be estimated by combining Eqs. (5.17) and (5.18) to produced Eq. (5.19). Example 5.7 illustrates the use of these equations in estimating the maximum sludge loading rate to a sand drying bed.

$$L = 10.4 s_0 \frac{n_v k_e E_{vn}}{[1 - (s_0/s_f)]\,(1 - D)} \quad \text{(SI units)} \tag{5.19a}$$

$$L = 5.41 s_0 \frac{n_v k_e E_{vn}}{[1 - (s_0/s_f)]\,(1 - D)} \quad \text{(U.S. customary units)} \tag{5.19b}$$

where L = sludge loading during period n_v, kg/m^2 (lb/ft^2)

10.4 = metric conversion factor assuming a sludge with a specific gravity of 1.04

5.41 = standard U.S. conversion factor assuming a sludge with a specific gravity of 1.04

n_v = length of operating season or an increment if evaporation is significantly different, months

s_0 = initial dry solids content, % as a decimal

s_f = final dry solids content, % as a decimal

D = fraction of water removed by drainage, decimal

E_{vn} = average pan evaporation rate during period n_v, in/month (cm/month)

k_e = reduction factor (% as a decimal) for sludge evaporation relative to free water surface (typically 0.6, or measured value)

Example 5.7 Walker County Wastewater Treatment Plant desires to use a series of sand drying beds to dewater an anaerobically digested sludge from an initial solids content of 2 percent to a final solids concentration of 25 percent. If 30 percent of the initial moisture in the loaded sludge is removed through drainage and the average monthly evaporation rate if 3.2 in, estimate the following:

1. Dewatering time for a single 12-in sludge layer
2. Number of applications that can be made per year
3. The annual solids loading rate

solution

Step 1. Estimate the dewatering time for a 12-in sludge layer using Eq. (5.17):

$$t_d = \frac{y_0\,[1-(s_0/s_f)]\,(1-D)}{k_e E_v}$$

$$= \frac{12\,[1-(0.02/0.25)]\,(1-0.30)}{0.6 \cdot 3.2 \text{ in/month}} = 4.0 \text{ months}$$

Step 2. Estimate the number of applications that can be made per year using Eq. (5.18):

$$N = \frac{n_v}{t_d} = \frac{12 \text{ months}}{4 \text{ months/application}} = 3 \text{ applications/year}$$

Step 3. Estimate the annual solids loading rate using Eq. (5.19*b*):

$$L = 5.41 s_0 \frac{n_v k_e E_{vn}}{[1-(s_0/s_f)]\,(1-D)}$$

$$= 5.41 \cdot 0.02 \quad \frac{12 \text{ months} \cdot 0.6 \cdot 3.2 \text{ in/month}}{[1-(0.02/0.25)]\,(1-0.3)} \qquad 3.87 \text{ lb/ft}^2 \cdot \text{yr}$$

The annual solids loading rate is determined by summation of the results of Eq. (5.19) for the operational periods selected over a full 12-month cycle. As a first approximation, assume that the number of operational months in the year n_v is 12, and use the average annual pan evaporation rate E_v if the beds are to operated on a year-round basis. The final depth of the dewatered sludge cake is given by Eq. (5.20):

$$y_f = y_0\,(s_0/s_f) \tag{5.20}$$

where y_f = final depth of applied sludge cake, in (cm)
y_0 = initial depth of applied sludge layer, in (cm)
s_0 = initial dry solids content, % as a decimal
s_f = final dry solids content, % as a decimal

Since thin layers of sludge will dry faster than a thick layer, it is sometimes tempting to use excessively low loading rates despite the fact that the annual solids loading rate is independent of the depth of the individual layers applied. Moreover, applying thin layers of sludge has several disadvantages, including (1) more frequent operation and maintenance, (2) greater sand loss from the bed, and (3) increased costs.

To reduce operation and maintenance costs, an operational goal should be to achieve the maximum possible solids loading with the minimum number of applications and removal cycles. Repeated calculations of Eqs. (5.18) through (5.20) will converge on the most effective combination of initial solids concentration and layer depth for a particular project. Final optimization of the layer depth is only

TABLE 5.13 Loading Criteria for Anaerobically Digested, Nonconditioned Sludge on an Uncovered Sand Drying Bed*

Digested sludge type	Mass loading rate ($kg/m^2 \cdot yr$)†
Primary	100–200
Primary plus low-rate trickling filter	100–160
Primary plus waste-activated sludge	60–100

*Adapted from ref. [45].
†$lb/ft^2 \cdot yr = 0.2048\ kg/m^2 \cdot yr$.

possible with operational experience. Currently accepted loading criteria for sand drying beds are summarized in Table 5.13.

The upper end of these ranges applies to warm, dry climates and to sludges that drain readily. The final sludge moisture content required before solids removal depends on the technical or regulatory requirements for final sludge disposal or use. If no special requirements apply, the sludge cake typically is “liftable” at about 25 percent solids and can be recovered without excessive sand loss.

The annual solids loading rate depends on the solids concentration in the applied sludge. For example, an increase in solids content from 2 to 4 percent would double the loading rate and reduce the required bed area by one-half (Fig. 5.50). This relationship demonstrates the potential advantage of thickening or preconditioning the sludge prior to application to the bed.

However, increasing the solids content beyond 8 percent is not recommended because the sludge flow will not distribute uniformly on the bed at this level. Example 5.8 illustrates the standard approach for estimating the minimum required drying bed area.

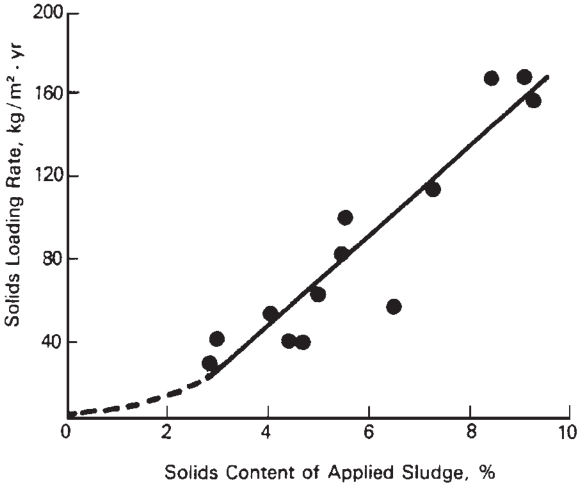


Figure 5.50 Influence of solids content on solids loading rate in sand drying beds.

Example 5.8 Drew County Wastewater Treatment Plant currently produces 1 million gal/year (3785 m^3/year) of anaerobically digested sludge having a solids content of approximately 2 percent. If the treatment works desires to use sand drying beds to dewater the sludge, estimate the minimum area necessary if the final solids content of the dewatered sludge should be at least 30 percent. Assume that the average annual pan evaporation rate for the area is 48 in (4 in/month) and that 25 percent of the sludge moisture is removed through gravity drainage. Assume also that the total required will be divided into six beds designed to operate year round.

solution

Step 1. Estimate the solids loading rate using Eq. (5.19*b*):

$$L = 5.41 s_0 \frac{n_v k_e E_{vn}}{[1 - (s_0/s_f)]\,(1 - D)}$$

$$= 5.41 \cdot 0.02 \quad \frac{12 \text{ months} \cdot 0.6 \cdot 4.0 \text{ in/month}}{[1 - (0.02/0.30)]\,(1 - 0.25)} \qquad 4.45 \text{ lb/ft}^2 \cdot \text{yr}$$

Step 2. Estimate the pounds of dry sludge generated at the wastewater treatment plant per year:

$$\text{lb/yr} = 20{,}000 \text{ mg/liter} \cdot \text{MG/yr} \cdot 8.34 \text{ lb/(mg/liter) MG}$$

$$= 166{,}800 \text{ lb/yr}$$

Step 3. Estimate the minimum total area required and the amount of area per bed:

$$\text{Minimum area required (ft}^2) = \frac{\text{solids production rate, lb/yr}}{\text{solids loading rate, lb/ft}^2 \cdot \text{yr}}$$

$$= \frac{166{,}800 \text{ lb/yr}}{4.45 \text{ lb/ft}^2 \cdot \text{yr}} = 37{,}482 \text{ ft}^2$$

For six beds, $\quad \text{Area per bed} = \dfrac{37{,}483 \text{ ft}^2}{6 \text{ beds}} = 6248 \text{ ft}^2\text{/bed}$

NOTE: Beds could be constructed as 80 ft by 80 ft basins (i.e., 6400 ft^2 per bed).

Typically, the total bed area is subdivided into multiple cells. It is convenient to size the cells so that one or two cells are adequate to contain the total volume of sludge from a scheduled digester withdrawal.

The design width of the bed depends primarily on the removal method. Small to moderate-sized facilities with hand or semimechanical dewatered sludge removal systems are about 6 m (20 ft) wide. Greater widths are used with mechanical removal methods, and sand beds in excess of 60 m (200 ft) have been constructed. Uniform sludge distribution on the bed can be difficult, particularly when polymers are used for conditioning. In these cases, the bed length should not exceed 15 to 25 m (50–75 ft), and/or multiple distribution points should be incorporated into the design [21,45].

5.7.1.2 Structural elements. Sludge can be applied to each cell with a valved pipe (plug valves) or from an open channel with gate controls along the perimeter of the bed. The open channel is easier to clean but presents more operational problems in cold weather. The valves in a pipe network should be protected from freezing in cold climates because the adjacent pipe will not always drain completely. Covered sand drying beds have been used in colder climates to extend seasonal dewatering operations.

A minimum sand depth of 12 in (30 cm) is recommended for proper drying-bed operation. In some cases, depths of up to 18 in (46 cm) can be used to extend the life of the bed. A splash block on the bed at every sludge entry point minimizes erosion of the sand. Since sand is unavoidably removed every time dewatered sludge is removed from the bed, new sand eventually must be installed. Preferred characteristics of the sand are (1) clean, hard particles (no clay, silt, or organic matter), (2) effective size in the range 0.01–0.3 in, or 0.3–0.75 mm, and (3) uniformity coefficient of less than 3.5 [45].

The gravel layer is usually 20 to 46 cm (8–18 in) thick, with gravel sizes ranging from 3 to 25 mm (0.1–1.0 in). With mechanical sludge removal, greater gravel depth is needed to structurally protect the underdrain network. A thinner layer of coarser stone, overlain by a permeable geotextile membrane, can be used when dewatered sludge is to be removed manually or mechanically with light equipment. Sidewalls normally are constructed of reinforced concrete, treated timber planks, or concrete planks.

Underdrains employed in sand drying beds usually are constructed from plastic pipe or clay tile laid with open joints. The main underdrain pipes should be at least 10 cm (4 in) in diameter and be laid with a slope of at least 1 percent to ensure drainage [45]. The water collected in the underdrainage network is returned to the headworks of the wastewater treatment plant. Characteristics of the drainage water or filtrate will vary depending on the type of sludge stabilization treatment process used. Table 5.14 summarizes the quality of filtrate from the dewatering of various sludge types.

5.7.1.3 Operation and maintenance. Any chemical conditioners added to sludge to enhance bed dewatering performance should be introduced continuously during the pumping operation at points in the system that will ensure proper mixing [37]. If possible, multiple dosage points for polymers should be constructed. Polymer dosage points, at a minimum, should be located ahead of the pump suction, at the pump

TABLE 5.14 Sludge Filtrate Characteristics*

Parameter	Raw waste-activated sludge	Digested sludge	
		Aerobic	Anaerobic
Soluble BOD, mg/liter	706	722	1012
COD, mg/liter	1585	1815	3325
Suspended solids, mg/liter	14	17	18
Total phosphorus, mg/liter	28	46	80

*Adapted from ref. [45].

discharge, and ahead of the discharge point to the bed. It may not be necessary to use all polymer dosage points, but the multiple array will allow optimization after operation commences.

Bed maintenance involves the periodic replacement of sand lost during dewatered sludge removal, leveling and scarification of the sand surface prior to discharging sludge, and removal of vegetation. Odors should not be a concern if well-stabilized sludges are being discharged to the bed. However, the application of unstabilized or partially stabilized sludge to a sand drying bed may result in the generation of objectionable odors. To control odors, calcium hypochlorite, potassium permanganate, or ferrous chloride can be added to the sludge prior to discharge to the bed.

5.7.2 Reed-enhanced sand drying beds

The reed-enhanced sand drying bed uses the common reed *Phragmites* to improve dewatering, stabilization, and nutrient removal during the dewatering of anaerobically or aerobically digested sludge [21]. The plants facilitate oxygen transport to their root systems that harbor a rich consortium of aerobic bacteria (Fig. 5.51). These bacteria metabolize the organic matter in the sludge while simultaneously mobilizing nutrients that promote vigorous plant growth [25]. In addition to enhancing organic matter and nutrient removal, the plants' dense root systems open moisture drainage channels, while their leaf systems are responsible for transpiring significant amounts of water [4].

During dewatering operations, sludge is allowed to accumulate within the reed-enhanced sand drying beds until it reaches a depth of approximately 3 to 4 ft (1 m). When the bed is full, it is taken out of service and allowed to stand for another 3 to 4 months to permit continued solids concentration. The sludge is then removed from the bed using mechanical methods.

The solids content of the digested sludge discharged to a reed-assisted sand drying bed can vary but normally will be no more than 4 percent solids, with a range of 1.5 to 2.0 percent solids preferred [21]. Discharging sludge with a solids content of greater than 4 percent may create difficulties in achieving a uniform distribution.

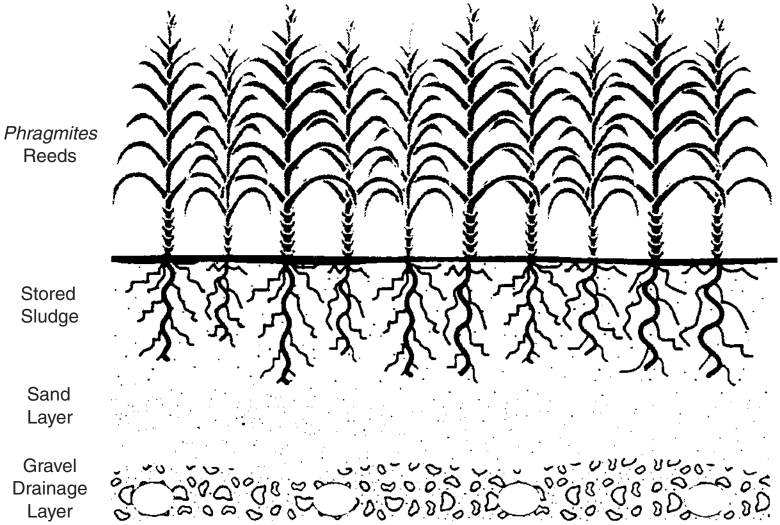


Figure 5.51 Conceptual design of a reed-enhanced sand drying bed. (*Adapted by permission from ref. [4].*)

The annual volumetric loading rate generally will range from 25 to 50 gal/ft^2·yr. However, the specific loading rate for a particular application will be a function of the sludge solids content and whether the sludge has been aerobically or anaerobically digested. Aerobically digested sludges impose less stress on the plants and normally can be applied at higher loading rates than anaerobically digested sludges.

At 2 percent solids, anaerobically digested sludge can be applied at loading rates of about 25 gal/ft^2·yr (1 m^3/m^2·yr), whereas aerobically digested sludges can be applied at approximately 50 gal/ft^2·yr (2 m^3/m^2·yr). The corresponding solids loading rates are 4.2 lb/ft^2·yr (20 kg/m^2·yr) for anaerobic sludges and 8.4 lb/ft^2·yr (40 kg/m^2·yr) for aerobic sludges. For each 1 percent increase in sludge solids content (up to 4 percent), the volumetric loading should be reduced by 10 percent to prevent the beds from becoming overloaded [21]. It should be noted that in the first growing season, sludge application rates should be reduced so that the plant root system can become established. Once the plants mature, the design sludge loading rate can be applied (Fig. 5.52).

The typical operational cycle for reed-enhanced sand drying bed consists of sludge application every 10 days during warm months and every 20 to 24 days during the winter. In temperate climates, this schedule allows approximately 28 sludge applications per year. For a 2 percent aerobic sludge, each sludge layer will be approximately 4 in (10.7 cm) thick. The number of separate reed beds required at a facility will depend on the frequency of discharging sludge to the system and the volume discharged during each event.

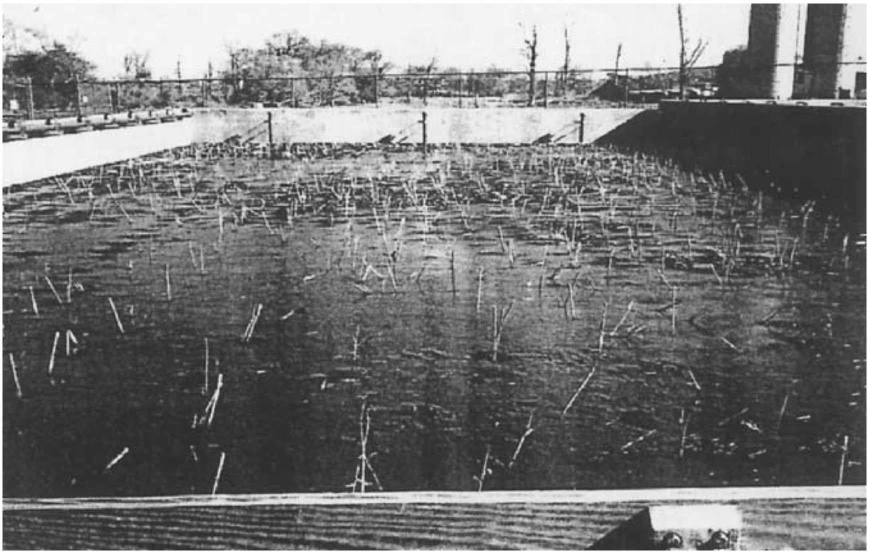


Figure 5.52 Photograph of a reed-enhanced sand drying bed. (*Adapted by permission from ref. [25].*)

The total area required for reed-enhanced sludge drying beds will be a function of the design solids loading rate. Although the solids loading rate for reed-enhanced sand drying beds rarely exceeds 8.5 $\text{lb/ft}^2 \cdot \text{yr}$, solids loading rates as high as 16.4 $\text{lb/ft}^2 \cdot \text{yr}$ (80 $\text{kg/m}^2 \cdot \text{yr}$) have been applied successfully to conventional sand drying beds [21]. Therefore, under normal circumstances, the area required for reed-enhanced sand drying beds will be significantly larger than for conventional sand drying beds. Example 5.9 illustrates the approach for estimating the size of a reed-enhanced sand drying bed.

Example 5.9 Drew County Wastewater Treatment Plant (see Example 5.8) generates 1 million gal/year (3785 m^3/year) of anaerobically digested sludge having a solids content of approximately 2 percent. The wastewater treatment plant desires to compare the area required for reed-enhanced sand drying beds with that for conventional sand drying beds. Estimate the minimum area required for a reed-enhanced sand drying bed if the volumetric loading rate is to be limited to 20 $\text{gal/ft}^2 \cdot \text{yr}$.

solution Estimate the minimum area required for the reed-assisted sand drying bed:

$$\text{Area (ft}^2\text{)} = \frac{\text{volumetric production rate, gal/yr}}{\text{volumetric loading rate, gal/ft}^2 \cdot \text{yr}}$$

$$= \frac{10^6 \text{ gal/yr}}{20 \text{ gal/ft}^2 \cdot \text{yr}} = 50{,}000 \text{ ft}^2$$

$$\text{For six beds:} \qquad \text{Area per bed} = \frac{50{,}000\ \text{ft}^2}{6\ \text{beds}} = 8333\ \text{ft}^2/\text{bed}$$

NOTE: At the given volumetric loading rate, reed-assisted drying beds will have an area requirement approximately 33 percent greater than conventional sand beds. To determine whether expending extra resources for constructing and operating the reed-assisted sand drying beds is warranted, the impact of the dewatering operation on sludge quality must be assessed.

Reeds normally are planted in the sand layer at approximately 12-in spacing and emerge from the root system as small shoots, growing to a height of 8 to 10 ft (2.4–3.0 m) within 3 months. At the end of the growing season, reeds are harvested approximately 8 in (20 cm) above the sludge surface so that new shoots can emerge from the root system in the next season. The reed harvest produces approximately 56 metric tons (wet weight) per hectare (25 tons/acre) or 2.5 tons (dry weight) per acre. The harvested material can be composted, landfilled, or where allowed, burned [21].

5.7.3 Freeze-assisted sand drying beds

Freezing and thawing sludge will transform a material with a jelly-like consistency to a granular-type material that drains readily. The freeze-thaw process is effective with any type of sludge and at any solids concentration. However, since the energy costs associated with an artificial freeze-thawing process are prohibitive, sludge treatment must be accomplished using the natural freezing and thawing to be cost-effective [20,21].

Freezing sludge changes both the structure of the sludge-water mixture and the characteristics of the solids particles. In effect, the solid matter tends to be compressed into large, discrete conglomerates surrounded by frozen water. When thawing commences, drainage occurs instantaneously through the large pores and channels created by the frozen water. Cracks in the frozen mass also act as conduits to convey melt water. Figure 5.53 illustrates the effect of freeze-thaw cycles on the sludge drainage rate.

5.7.3.1 Design considerations. The proper design of a freeze-assisted sand drying bed must be based on worst-case environmental conditions to ensure successful performance at all times. If sludge freezing is to be a reliable expectation every year, the design should assume the warmest winter during the period of concern (usually 20 years), from which a design sludge layer thickness that will freeze completely during the winter period is chosen. In most cases, a 3-in (8-cm) sludge layer is chosen as the design sludge thickness because

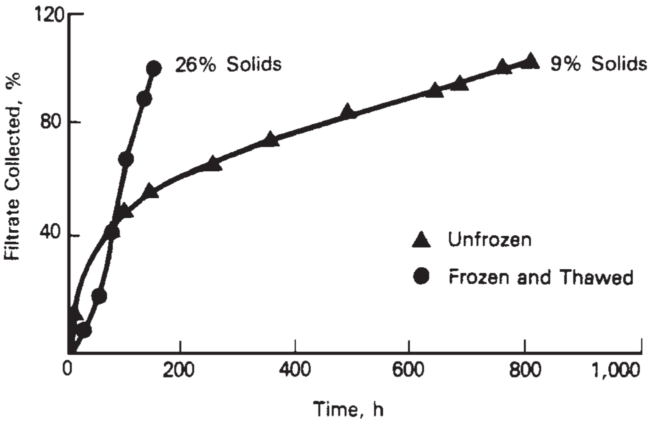


Figure 5.53 Effect of freeze-thawing cycles on the drainage rate for anaerobically digested sludge.

this depth typically can be frozen in moderately cold climates. For colder climates, a much thicker design thickness can be employed. For example, an 18-in (46-cm) layer has been used as the design thickness for construction of freeze-assisted sand drying beds in Fairbanks, Alaska [21]. The depth of freezing or thawing of a sludge layer depends on environmental conditions and may be estimated using Eq. (5.21):

$$Y = m\,(\Delta Tt)^{1/2} \tag{5.21}$$

where Y = depth of freezing or thawing, in (cm)
m = proportionality coefficient, 0.6 in (°F · day) $^{-1/2}$ (U.S. units), 2.04 cm (°C · day) $^{-1/2}$ (SI units)
ΔTt = freezing or thawing index, °F·day (°C·day)
ΔT = difference between average ambient temperature and freezing temperature, °F (°C)
t = time period of concern, days

The proportionality coefficient m is related to the thermal conductivity, density, and latent heat of fusion for the material being frozen or thawed. A median value of 0.6 in (°F·day)$^{-1/2}$ [or 2.04 cm (°C·day)$^{-1/2}$] was determined experimentally for wastewater sludges having a solids content in the range of 0 to 7 percent [21].

The freezing or thawing index (ΔTt) is an environmental parameter that is characteristic for a particular location. The freezing or thawing indices are sometimes published, but if they are unavailable from the literature, they can be estimated from temperature records (Example 5.10).

Example 5.10 The average daily air temperature data for the following 7-day winter period at the Washington County Wastewater Treatment facility are given as follows:

Day	Mean temperature
1	−2°C
2	−6°C
3	−3°C
4	−9°C
5	0°C
6	−1°C
7	−5°C

Estimate the freezing index for the 7-day period.

solution

Step 1. Estimate the average air temperature for the 7-day period:

Average air temperature (°C)

$$= \frac{(-2°\text{C}) + (-6°\text{C}) + (-3°\text{C}) + (-9°\text{C}) + (0°\text{C})\,(-1°\text{C}) + (-5°\text{C})}{7}$$

$$= -3.7°\text{C}$$

Step 2. Estimate the freezing index:

$$\Delta T \cdot \text{day} = [0°\text{C} - (-3.7°\text{C})\,] \cdot 7 \text{ days} = 25.9°\text{C} \cdot \text{day}$$

To determine the minimum time necessary to freeze a sludge layer of a particular thickness Y, Eq. (5.21) can be rearranged to generate Eqs. (5.22*a*), (5.22*b*), and (5.22*c*).

$$t = \frac{(Y/m)^2}{\Delta T} \qquad (5.22a)$$

where Y = thickness of sludge layer, in (cm)
ΔT = difference between average ambient temperature and freezing temperature, °F (°C)

For example, the minimum time (days) required for freezing or thawing a 3-in-thick (8-cm) sludge layer becomes

$$t = \frac{15.38}{\Delta T} \qquad \text{(SI units)} \qquad (5.22b)$$

NOTE: Y = 8 cm and m = 2.04 cm $(°\text{C}\cdot\text{day})^{-1/2}$.

$$t = \frac{25.0}{\Delta T} \qquad \text{(U.S. customary units)} \qquad (5.22c)$$

NOTE: $Y = 3$ in and $m = 0.6$ in $(°\text{F}\cdot\text{day})^{-1/2}$.

Equation (5.22*a*) or (5.22*b*) can be used with local temperature data to determine how many 3-in (8-cm) layers can be frozen during each winter of the study period. The year with the smallest number of frozen layers is then designated as the control year for design. In the calculations, thaw periods should be taken into account, since a new sludge layer will not be applied until the previous layer has frozen. The procedure is repeated through the end of the winter season. A tabular format is recommended for summarizing data and calculation results (see Example 5.11).

Example 5.11 Temperature data over a 4-year study period is being used to identify the design year for developing the construction specifications for a freeze-assisted sludge drying bed dewatering system. It is assumed that freezing weather will only occur over a 4-month period (December through March). The following are the average monthly temperatures.

	December (31 days)	January (31 days)	February (28 days)	March (31 days)
Year 1	−2°C	−6°C	−5°C	−1°C
Year 2	−4°C	−8°C	−2°C	−3°C
Year 3	−1°C	−4°C	−4°C	−2°C
Year 4	−3°C	−2°C	−3°C	−1°C

What year should be used as the design year for freeze-assisted sludge drying bed design?

solution

Step 1. Estimate the number of days required to freeze an 8-cm sludge layer during the average monthly temperature conditions using Eq. (5.22). For example, the average days required to freeze an 8-cm sludge layer in the month of December of year 1 is estimated using Eq. (5.22*b*) as follows:

$$t = \frac{15.38}{\Delta T} = \frac{15.38}{0°\text{C} - (-2°\text{C})} = 7.7 \text{ days}$$

Step 2. Estimate the number of 8-cm layers that can be frozen in the month of December of year 1:

$$\text{No. of 8-cm layers that can be frozen} = \frac{\text{31 days in month}}{\text{7.7 days required/frozen layer}}$$

$$= 4.0 \text{ (8-cm layers can be frozen)}$$

Step 3. Calculate the number of frozen 8-cm layers that can be frozen in each month for each year and total:

Year 1	Number of frozen 8-cm layers
December	4
January	12
February	9
March	2
Total	27

Year 2	Number of frozen 8-cm layers
December	8
January	16
February	4
March	6
Total	34

Year 3	Number of frozen 8-cm layers
December	2
January	8
February	7
March	4
Total	21

Year 4	Number of frozen 8-cm layers
December	6
January	4
February	5
March	2
Total	18

Since year 4 produces the least number of frozen 8-cm sludge layers, year 4 should be used as the design year.

It should be noted that the rate of freezing decreases with time under steady-state temperatures because the frozen material acts as an insulating barrier between the cold ambient air and the remaining unfrozen sludge. As a result, it is possible to freeze a greater total depth of sludge in a given time if the sludge is applied in thin layers.

5.7.3.2 Preliminary feasibility assessment. A rapid method for performing a preliminary assessment of the freeze-assisted dewatering process involves comparing the potential depth of frozen sludge in the "design" year to the maximum depth of frost penetration at a particular location using Eqs. (5.23*a*) and (5.23*b*). The maximum depth of frost penetration for an area can be found in local weather and soil records (Table 5.15). Example 5.12 illustrates use of the preliminary assessment approach.

$$\Sigma Y = 1.76F_p - 101 \qquad \text{(SI units)} \qquad (5.23a)$$

$$\Sigma Y = 1.76F_p - 38 \qquad \text{(U.S. customary units)} \qquad (5.23b)$$

TABLE 5.15 Maximum Depth of Frost Penetration (F_p) and Potential Depth of Frozen Sludge ($\sum Y$)*

Location	Maximum frost penetration (cm)	Potential depth of frozen sludge (cm)
Bangor, Maine	183	221
Concord, N.H.	152	166
Hartford, Conn.	124	117
Pittsburgh, Pa.	97	70
Chicago, Ill.	122	113
Duluth, Minn.	206	261
Minneapolis, Minn.	190	233
Montreal, Que.	203	256

*Adapted from ref. [21].

where $\sum Y$ = total depth of sludge that can be frozen in 3-in (8-cm) layers during design year

F_p = maximum depth of frost penetration into the soil for the location, cm (in)

Example 5.12 Garvey County Wastewater Treatment Facility is considering freeze-assisted sand drying for dewatering its anaerobically digested sludge. If the sludge production rate is 2500 m^3/year (at 3 percent solids), estimate the potential depth of the frozen sludge and the minimum bed area if the average frost penetration for the area has been found to be 110 cm.

solution

Step 1. Estimate the potential depth of frozen sludge using Eq. (5.23*a*):

$$\sum Y = 1.76F_p - 101$$

$$= 1.76 \cdot 110\text{ cm} - 101 = 92.6\text{ cm}$$

Step 2. Estimate the minimum bed area required to completely freeze the generated sludge:

$$\text{Minimum bed area (m}^2) = \frac{\text{annual volumetric production (m}^3)}{\text{depth of frozen sludge (m)}}$$

$$= \frac{2500\text{ m}^3}{0.926\text{ m}} = 2700\text{ m}^2\ (29{,}060\text{ ft}^2)$$

From Eqs. (5.22) and (5.23), it can be shown that sludge freezing will not be feasible unless the maximum depth of frost penetration is at least 22 in (57 cm) for a particular location. In general, such environmental conditions will occur above the thirty-eighth parallel, which includes most of the northern half of the United States, with the exception of the West Coast [21].

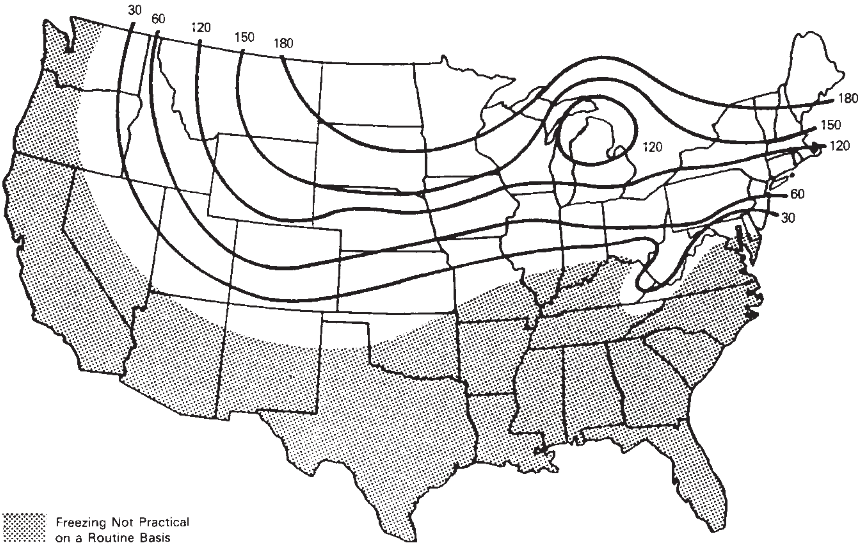


Figure 5.54 Potential depth of sludge freezing in the continental United States. (*Adapted by permission from ref. [21].*)

A minimum frost penetration of 110 cm (39 in) would result in a frozen sludge depth of approximztely 75 cm (30 in). The freeze-assisted dewatering process should be cost-effective under these circumstances depending on land and construction costs. Using Eqs. (5.23*a*) and (5.23*b*), the potential depth of frozen sludge over the continental United States is depicted in Fig. 5.54. This figure can be used for preliminary estimates, but the final design should be based on actual weather records for the site.

The time required for sludge thawing can be estimated using Eq. (5.23*a*) or (5.23*b*) and the appropriate thawing index. The coolest expected spring/summer temperatures should be used for design purposes, and the depth to be used in the equation is the total depth of frozen sludge, not the individual layers (see Example 5.13). Data illustrating the effects of the freeze-assisted dewatering process are provided in Table 5.16.

Example 5.13 Preliminary evaluation of the freeze-assisted dewatering process at the Johnson County Wastewater Treatment Plant indicated that a 90-cm (35-in) depth of sludge could be frozen in the design year. Assuming that the average monthly temperatures are below freezing from December through February, estimate the number of days required to completely thaw the frozen sludge. Assume that the average monthly temperatures during March (31 days), April (30 days), May (31 days), and June (30 days) are 9, 18, 25, and 32°C, respectively.

TABLE 5.16 Effects of Sludge Freezing on Sludge Solids Content*

Location and sludge type	Percent solids content Before freezing	 After freeze-thaw
Cincinnati, Ohio		
Primary sludge (with alum)	0.7	18.0
Ontario, Canada		
Waste-activated sludge	0.6	17.0
Anaerobically digested sludge	5.1	26.0
Aerobically digested sludge	2.2	21.0
Hanover, N.H.		
Digested primary sludge (with alum)	2–7	25–35
Digested primary sludge	3–8	30–35

*Adapted from ref. [21].

solution

Step 1. Determine the thawing index for the 90-cm depth of frozen sludge [Eq. (5.21)]:

$$\Delta Tt = (Y/m)^2 = \left[\frac{90\text{ cm}}{2.04\text{ cm }(°\text{C}\cdot\text{day})^{-1/2}}\right]^2$$

$$= 1946°\text{C}\cdot\text{day}$$

Step 2. Determine the number of days with an average temperature above freezing that is required to completely thaw the frozen sludge:

$$\Delta Tt = \text{March} + \text{April} + \text{May} + \text{June}$$

$$\Delta Tt = 9°\text{C}\cdot 31\text{ days} + 18°\text{C}\cdot 30\text{ days} + 25°\text{C}\cdot 31\text{ days} + 32°\text{C}\cdot 11\text{ days}$$

$$= 279°\text{C}\cdot\text{day} + 540°\text{C}\cdot\text{day} + 775°\text{C}\cdot\text{day} + 352°\text{ C}\cdot\text{day}$$

$$= 1946°\text{C}\cdot\text{day}$$

The frozen sludge will be completely thawed in 103 days (or by June 12).

The same basic structure used for conventional sand drying beds can be used for freeze-assisted drying beds. The area can be designed either as a series of underdrained beds, similar in detail to the conventional sand drying beds, or as a deep, lined, or underdrained trench. Neither beds nor trenches require a roof or cover. However, application of sludge under a deep snow cover must be avoided because the snow will act as an insulator and retard sludge freezing [21]. Any deep snow layer (i.e., beyond 1 ft in depth) should be removed prior to a new sludge application.

Although effective, freeze dewatering is a seasonal process. Except in cold climates, it is not economical to store sludge in the warm months and depend only on winter freezing for dewatering. Moreover, if freezing is the only method used to dewater, sludge storage is required during warm periods. In most parts of the United States, it will be more cost-effective to combine winter freezing with polymer-assisted summer dewatering on the same beds. This combination of techniques eliminates the need for large-scale sludge storage and reduces the total number of beds required. Finally, it should be noted that the freeze-assisted dewatering process will not improve pathogen destruction, and therefore, the process offers little additional stabilization [21,37].

5.7.4 Paved sludge drying beds

Until recently, paved sludge drying beds were constructed with an asphalt or concrete pavement placed over a porous gravel subbase. Unpaved areas (typically constructed sand drains) were positioned around the perimeter or along the center of the bed to collect and convey drainage water (Fig. 5.55). The main advantage of this approach was the ability to use relatively heavy equipment (e.g., front-end loaders) for dewatered sludge removal [45]. Reported field data have indicated that the pavement inhibits drainage so that the total bed area has to be significantly greater than that of conventional sand drying beds to achieve the same level of dewatering within a comparable time period [37].

Figure 5.55 Photograph of a standard paved drying bed.

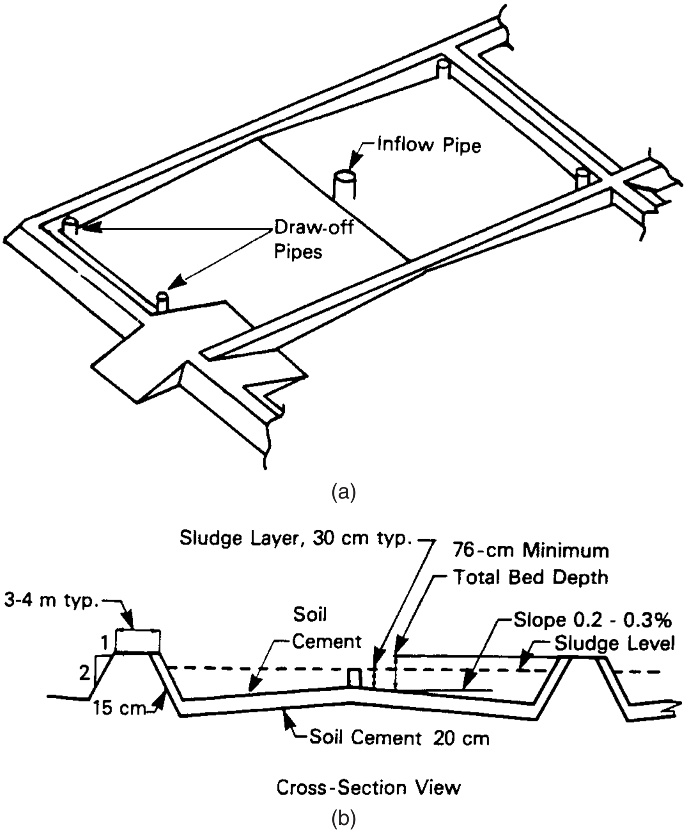
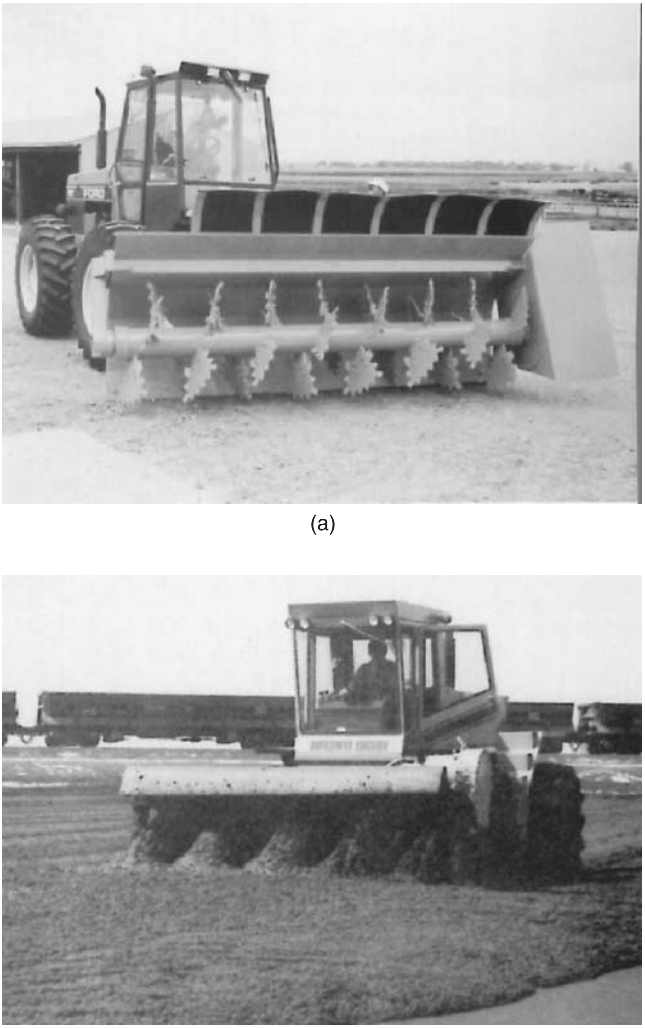


Figure 5.56 Schematic diagram of a paved drying bed designed for decanting and enhanced evaporation: (*a*) plan view; (*b*) side view.

Recent improvements to the paved sludge drying bed process include (1) use of decanting structures to remove supernatant and (2) use of a tractor-mounted horizontal auger/aerator (or other device) to regularly break up the surface crust that inhibits moisture evaporation (Fig. 5.56).

5.7.4.1 Design considerations. Paved beds can be used in any location, but since evaporation provides the major mechanism for water removal, they work best in warm, arid and semiarid climates. Like sludge drying lagoons, the design solids loading for paved sludge drying beds is a function of the potential moisture evaporation and precipitation in the local area. In many paved drying bed operations, it is possible to decant sludge supernatant that can account for as much as 20 to 30 percent of the water removed from the sludge. Moreover, if the sludge has particularly good settling characteristics, it may be



(a)

(b)

Figure 5.57 (*a*) Photograph of a tractor-mounted auger/aerator. (*b*) Photograph of a tractor-mounted auger/aerator being employed to enhance dewatering of sludge applied to paved drying beds. (*Courtesy of Brown Bear Corp.*)

possible to use several fill and draw cycles to remove moisture prior to actively facilitating evaporative moisture removal by aerating [45].

The rate of evaporation may be determined through small-scale pilot studies, or it can be estimated to be a fraction of the pan evaporation rate in the local area. To estimate the mass of water that must be removed from the sludge to achieve a desired final moisture content, the initial water content and the amount of water removed through the decantation process must be determined. The total mass of water applied with the sludge and the amount of water removed through the decantation process may be estimated using Eqs. (5.24) and (5.25), respectively.

$$W_0 = 1.04S\,[(1 - s_0)/s_0)] \tag{5.24}$$

where W_0 = total water content in applied sludge, lb/yr (kg/yr)
1.04 = assumed specific gravity of the sludge
S = annual sludge production, dry solids, lb (kg)
s_0 = dry solids in applied sludge, % as a decimal

$$W_D = 1.04S\,[(1 - s_d)/s_d] \tag{5.25}$$

where W_D = total water remaining after decantation, lb/yr (kg/yr)
1.04 = assumed specific gravity of the sludge
S = annual sludge production, dry solids, lb (kg)
s_d = dry solids in sludge after decantation, % as a decimal

Since paved sludge drying beds typically are uncovered, precipitation must be taken into account when evaluating the required evaporation rate to achieve a final desired sludge moisture content. Equations (5.26*a*) and (5.26*b*) may be used to estimate the required evaporative moisture removal rates given an annual precipitation rate (ft/yr, m/yr):

$$W_E = W_D - 1.04S\,[(1 - s_e)/s_e] + PA \cdot 1000 \quad \text{(SI units)} \tag{5.26a}$$

$$W_E = W_D - 1.04S\,[(1 - s_e)/s_e] + PA \cdot 62.4 \quad \text{(U.S. customary units)} \tag{5.26b}$$

where W_E = total water to be evaporated after decantation, lb/yr (kg/yr)
1.04 = assumed specific gravity of the sludge
S = annual sludge production, dry solids, lb (kg)
s_e = dry solids required in sludge after evaporation, % as a decimal
P = annual precipitation, ft (m)
A = bed area, ft^2 (m^2)
1000 = conversion factor (1000 kg/m^3)
62.4 = conversion factor (62.4 lb/ft^3)

NOTE: If the paved drying bed does not allow for decantation, use W_0 rather than W_D in Eqs. (5.26*a*) and (5.26*b*).

The area-specific evaporation rate for sludge in a given location can be estimated using Eqs. (5.27*a*) and (5.27*b*). It should be noted that the area-specific evaporation rate for sludge assumes that the evaporation rate is a fraction of the pan evaporation rate.

$$R_e = 1000k_eE_v \qquad \text{(SI units)} \tag{5.27a}$$

$$R_e = 62.4k_eE_v \qquad \text{(U.S. customary units)} \tag{5.27b}$$

where R_e = evaporation potential for sludge on a mixed and aerated paved bed, kg/m² · yr (lb/ft² · yr)
k_e = reduction factor for sludge evaporation versus free water surface, % as a decimal (0.6 is typical)
E_v = free water pan evaporation rate, m/yr (ft/yr)

Dividing the required moisture removal rate by the area-specific evaporation rate [Eq. (5.28*a*) or (5.28*b*)] allows estimation of the minimum paved drying bed area to achieve a final sludge moisture content (see Example 5.14).

$$\text{Area (m}^2) = \frac{1.04S\,[(1 - s_d)/s_d - (1 - s_e)/s_e] + PA \cdot 1000}{R_e} \qquad \text{(SI units)} \tag{5.28a}$$

$$\text{Area (ft}^2) = \frac{1.04S\,[(1 - s_d)/s_d - (1 - s_e)/s_e] + PA \cdot 62.4}{R_e} \qquad \text{(U.S. customary units)} \tag{5.28b}$$

where A = bed area, ft² (m²)
R_e = evaporation potential for sludge on a mixed and aerated paved bed, lb/ft² · yr (kg/m² · yr)
1.04 = assumed specific gravity of the sludge
S = annual sludge production, dry solids, lb (kg)
s_d = dry solids in sludge after decantation, % as a decimal
s_e = dry solids required in sludge after evaporation, % as a decimal
P = annual precipitation, ft (m)

It should be noted that since sludge drying lagoons (Sec. 5.6) employ the same moisture-removal mechanisms as paved drying beds, the present design equations may be employed for estimating the minimum required area for those systems as well.

Example 5.14 The Lamu County Sewer Improvement District has decided to employ paved drying beds to increase the solids content of its anaerobically digested sludge from 2.6 to 25.0 percent. If the facility is generating

2.4 million gallons of anaerobically digested sludge per year, estimate the required bed area if the solids content of the sludge after decanting has been estimated at 10 percent, the annual precipitation and evaporation rates are 2.5 and 3.7 ft/year, respectively, and the reduction factor for sludge evaporation k_e has been measured to be 0.8.

solution

Step 1. Estimate the pounds of sludge solids generated each year:

Sludge production (lb/yr)

$$= 26{,}000 \text{ mg/liter} \cdot 2.4 \text{ MG/yr} \cdot 8.34 \text{ lb/[MG (mg/liter)]}$$

$$= 520{,}416 \text{ lb/yr}$$

Step 2. Estimate the sludge moisture evaporation rate R_e, using Eq. (5.27*b*):

$$R_e = 62.4 k_e E_v$$

$$= 62.4 \text{ lb/ft}^3 \cdot 0.8 \cdot 3.7 \text{ ft/yr} = 184.7 \text{ lb/ft}^2 \cdot \text{yr}$$

Step 3. Using Eq. (5.28*b*), estimate the minimum required bed area:

$$\text{Area (ft}^2) = \frac{1.04S\,[(1 - s_d)/s_d - (1 - s_e)\,/s_e] + PA \cdot 62.4}{R_e}$$

$$= \{1.04 \cdot 520{,}416 \text{ lb/yr} \cdot [\,(1 - 0.1)\,/0.1 - (1 - 0.25)\,/0.25]$$

$$+ 2.5 \text{ ft/yr} \cdot A \cdot 62.4\}/184.7 \text{ lb/ft}^2 \cdot \text{yr}$$

$$= 17{,}584 \text{ ft}^2 + 0.845 \cdot A \text{ (ft}^2)$$

$$= 113{,}150 \text{ ft}^2 \text{ (2.6 acres)}$$

Once the minimum bed area has been determined, sludge storage capacity can be incorporated into the dewatering system design by multiplying the necessary dewatering area by a safety factor (typically 1.5). The total design bed area should be divided into at least three beds for all but the smallest operation to provide operational flexibility [45]. A detailed month-to-month analysis of weather records will determine the optimum number of beds required.

Since moisture evaporation and decanting are the principal water-removal processes, paved beds have been constructed with and without drains [45]. A long rectangular configuration improves operational efficiency by reducing the time required for turning the auger/aerator vehicle. A number of inlet and decantation structures are also possible. The minimum total depth of the bed is about 2.6 ft (0.8 m), which is necessary to provide freeboard above the 12-in (30-cm) sludge layer. In some systems, up to 3 ft (1 m) of liquid sludge is applied in the initial layer, with the freeboard increased proportionately [45].

The major operational steps for paved sludge drying beds include (1) sludge application, (2) decantation, (3) mixing/aeration, and (4) sludge removal. Depending on the size of the operation and the time of year, the sludge should be augered several times a week to maintain optimal evaporating conditions. Maintenance requirements include routine care of the auger/aeration equipment, the sludge pumps and piping network, the decantation piping, and the bed and dikes. If the site experiences freezing weather in the winter months, the valves and pumps in the system need to be protected and checked periodically during the critical freezing periods [21].

5.7.4.2 Performance expectations. In general, to avoid odor complaints and to satisfy regulatory requirements only digested or otherwise stabilized sludge should be discharged into a paved drying bed. The initial decantation phase may require 2 to 3 days for sludge settling, with 1 to 2 additional days required to decant each additional increment of sludge applied [21]. If gravity drainage is included in the paved drying bed design, it should be essentially complete during the time allowed for sludge settling and decanting.

The final evaporation drying period will depend on the climatic conditions and the regular use of the auger/aerator equipment. Sludge solids contents in the range of 40 to 50 percent can be achieved within 30 to 40 days in arid climates.

5.8 Storage Facilities for Dewatered Biosolids/Sludge

Dedicated dewatered biosolids/sludge storage facilities can be designed to store dry solids (above 60 percent solids) or wet solids (15–60 percent solids). Dry solids typically are the product of heat-drying, high-temperature conversion, or air-drying processes and can be stored in any of the standard dry materials storage structures [21,45]. The storage of wet solids is more challenging, with the most commonly used structures being (1) sludge drying lagoons, (2) drying beds, (3) enclosed (i.e., confined) structures, or (4) unconfined stockpiles. The first two types of storage facilities have been discussed previously. This section focuses on confined and unconfined storage structures for dewatered biosolids/sludge.

5.8.1 Confined hoppers or bins

In the design of enclosed or confined storage systems for short- or long-term storage of dewatered sludge, it is often tempting to base the design on volume of the dewatered material. However, if the dewatered product is wet (i.e., less than 30 percent solids), several materials handling problems arise with this design approach. These materials handling problems include continuing decomposition, liquefaction, concentration, and consolidation. Although each may lead to its own effect, all three materials handling problems are interrelated and combine to limit the use of this type of storage to equalization storage only and then only if special precautions are taken [45].

5.8.1.1 Continuing decomposition. Unless sludge is stabilized to nonreactive levels (more than 50 percent solids by dry weight), the volatile organic material will continue to decompose. This decomposition will reduce organic material and generate gaseous by-products. Depending on the stage and type of stabilization/conditioning process employed prior to dewatering and the dewatering method itself, gaseous by-products may be odorous. For example, storage of aerobically stabilized dewatered sludge would be more likely to generate strong odors than storage of anaerobically stabilized dewatered sludge. Similarly, polymer-conditioned dewatered sludge would be more likely to generate strong odors relative to dewatered sludge conditioned with lime and ferric chloride.

Enclosed structures are often used for storage of dewatered sludge to ensure odor-free operation. Such structures are extremely hazardous if the designer fails to recognize the potentially explosive nature of some of the gaseous by-products [37]. To minimize the hazardous conditions and to ensure convenient access to the loading equipment, the storage structures must be ventilated. At a minimum, the area should be ventilated at 20 to 30 air changes per hour [3].

Exhausted air should be effectively treated through the use of an odor removal system (e.g., chemical scrubber, incinerator, etc.). To ensure a negative pressure within the area and to minimize gas leakage that might bypass the odor-removal system, the exhaust ventilation should be slightly greater than supply ventilation. The atmosphere of enclosed areas should be monitored with hydrocarbon detectors to provide ample warning if the gas released begins to develop dangerous mixtures of methane and air.

5.8.1.2 Liquefaction. When the reduction of putrescible organic material proceeds within a confined structure, liquefaction of dewatered solids may occur. The rate at which liquefaction occurs is affected by storage time, temperature, and sludge chemical characteristics [37]. For example, liquefaction is negligible when the storage is limited to equalization (3–4 days) but can become significant as the storage time increases much beyond this level. Moreover, if lime and ferric chemicals are used to condition the sludge for dewatering, liquefaction will be greatly reduced because of both the lower overall organic matter concentration and the inhibiting effects of these chemicals on the microbial metabolism of the organic matter [28,37]. Because of the

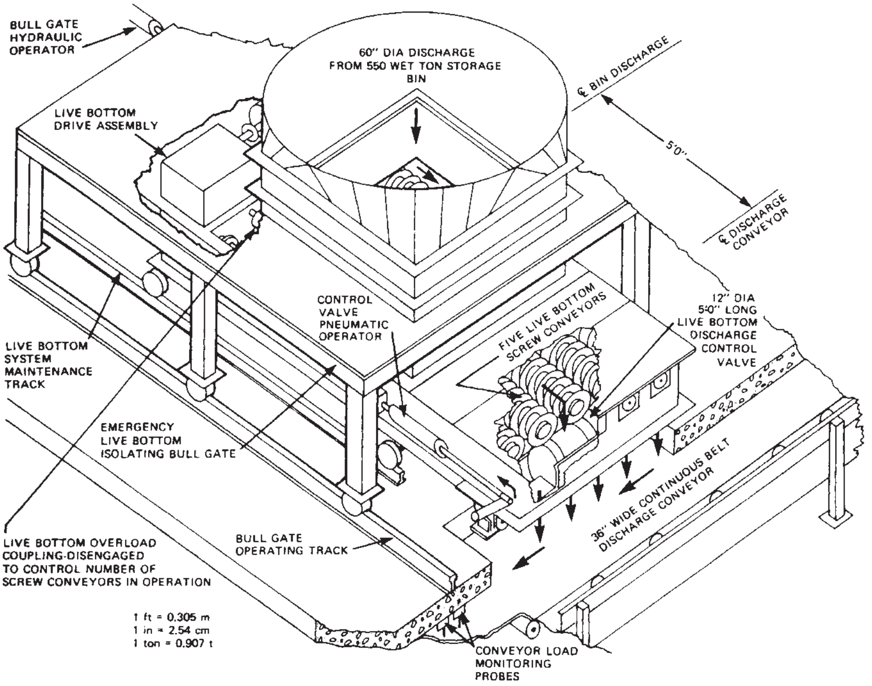


Figure 5.58 Dewatered sludge storage bin discharge control system.

impact of sludge liquefaction on solids management operations, the design engineer must account for it when designing the materials handling and transport systems.

5.8.1.3 Concentration and consolidation. The materials handling properties of the dewatered sludge entering an enclosed storage facility often do not resemble those of the materials discharged from the same facility. The method of controlling the discharge must have sufficient flexibility to adapt to the changes in material consistency.

As long as the storage containment volume does not exceed the capacity of the transport system receiving the discharge and the transportation system is of the bulk handling type (e.g., truck, rail car, or barge), the discharge control can be a simple open-close valve. Facilities whose storage volumes exceed the discharge transport system capacity or whose transportation system is a continuous-rate type (e.g, conveyor belt, screw conveyor, pipeline) must be provided with a backup discharge system capable of handling sludge of varying consistencies. Such backup systems must be provided with remote controls capable of detecting material overload prior to their overwhelming thetransport system. The controls must be capable of automatically closing the discharge control system isolating valves [37].

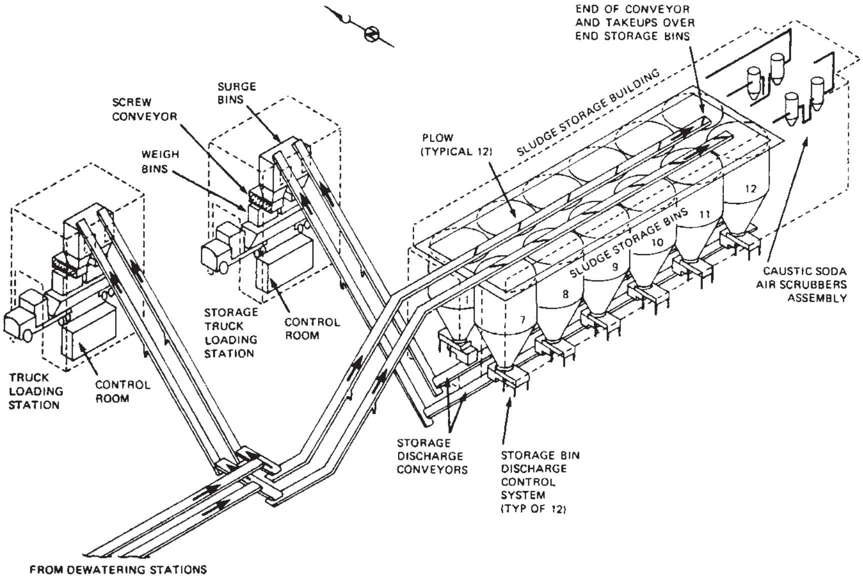


Figure 5.59 Schematic diagram of a dewatered sludge storage and truck loading station.

5.8.2 Earthen structures for dewatered biosolids/sludge storage

Use of diked storage facilities is normally limited for biosolids and sludge having solids contents of at least 20 percent. At above-ground earthen storage facilities, dikes are used to contain the biosolids or sludge, and haul vehicles discharge dewatered material directly into the containment area. The earthen dikes can be of various heights, but they require side slopes of at least 2:1 and possibly 3:1. A 15-ft-wide (4.6-m) road covered with gravel should be constructed atop the dikes.

If biosolids or sludge is removed periodically from the facility, it is considered a storage and treatment unit. If biosolids or sludge is never removed, it is considered a surface-disposal facility and is then subject to the surface-disposal restrictions of 40 CFR Part 503 [65,66].

5.8.3 Unconfined stockpiles

Unconfined stockpiles and mounds are a major method of providing equalization and short-term storage for dewatered sludge. In general, piles and mounds are suitable only for stabilized sludges with a high chemical content (i.e., greater than 40 percent lime plus ferric chloride) or a low organic matter content.

Unconfined stockpiles of dewatered biosolids or sludge having solids contents less than 25 percent solids lose all structural stability

Figure 5.60 Photograph of a dewatered biosolids/sludge stockpile being employed for short-term storage.

when exposed to extensive rainfall. Moreover, the rewetting event can lead to malodorous conditions. The adverse effect of rainfall on this system is the principal reason that this type of storage process is limited to equalizing or short-term storage. Covered stockpiles are often used for equalizing storage in those areas where rainfall is intense. However, such stockpiling is usually limited because of the expense of developing covered areas of sufficient size to provide adequate storage area and equipment accessibility.

It should be noted that if there is no intention of removing the sludge to a final disposal or land-application site, the stockpile or mound is considered a surface-disposal facility and is subject to the requirements of the 40 CFR Part 503 rule (e.g., requirements for pathogen control, vector attraction reduction, pollutant limits, siting restrictions of public access, runoff collection, and groundwater protection). To protect groundwater, it is recommended that stockpiles and mounds be located on an impervious surface. Figure 5.60 depicts a typical unconfined stockpile commonly used for short-term dewatered biosolids/sludge storage.

5.9 Treatment of Sidestreams from Biosolids/Sludge Processing

Sidestreams generated as a result of biosolids/sludge processing represent a significant source of organic matter and suspended solids loadings to wastewater treatment systems [37,68]. Failure to account for these sludge-processing liquors in the overall wastewater treatment

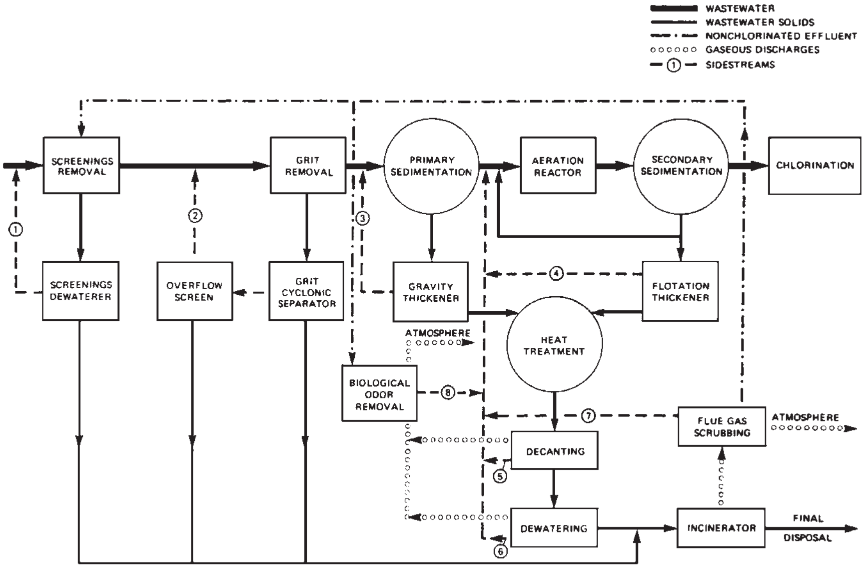


Figure 5.61 Schematic diagram illustrating sources of sidestreams.

design can result in the overloading of critical wastewater treatment operations [28,37]. Sidestreams are produced during sludge thickening, stabilization, conditioning, and dewatering, as well as air pollution control (e.g., scrubbers) and the washing and transporting of debris from structures and equipment. Figure 5.61 illustrates eight typical sidestreams generated within a wastewater treatment plant, including those produced from (1) screenings centrate, (2) grit separator overflow, (3) gravity thickener supernatant, (4) dissolved air-flotation subnatant, (5) decantate following heat treatment, (6) mechanical dewatering filtrate and wash water, (7) scrubber water from furnace flue gas cleanup, and (8) overflow from a biological odor-removal system. It should be noted that the choice of location for reintroducing sidestreams into the wastewater treatment system should consider the potential adverse impacts of this waste stream on process performance [28].

5.9.1 Sidestream quality

The relationship between a wastewater treatment plant effluent and sludge quality must be scrutinized carefully during the planning of sidestream treatment in order to avoid compliance violations in either wastewater effluent or biosolids quality. Sidestreams should be returned to points in the wastewater treatment process that will result in their effective treatment while protecting downstream water and sludge quality. In general, several options are available for minimizing the adverse impact of sidestreams on wastewater treatment operations. These options include the following:

1. Modification of sludge treatment and disposal systems to eliminate a particular sidestream
2. Modification of previous sludge-processing steps to improve sidestream water quality
3. Changing the timing, return rate, or return point for reintroducing sidestreams into the wastewater treatment system
4. Modification of wastewater treatment facilities to accommodate sidestream loadings
5. Providing separate sidestream treatment

Each of these options is discussed briefly in the following sections.

Finally, although sidestreams generated from sludge-processing operations within the wastewater treatment plant may be reintroduced directly into the wastewater treatment operations, treatment of (1) leachate generated from sludge composting, (2) sludge lagoon supernatant, or (3) runoff from biosolids land-application sites poses a unique challenge to sidestream management because these wastewater flows typically are generated in locations remote from the wastewater treatment plant. Options for treatment of these process streams may include (1) on-site treatment, (2) collection and pumping of sidestreams to the wastewater treatment plant, or (3) discharge into the municipal sewer system (if available).

5.9.1.1 Elimination of sidestreams. Specific situations may arise at the wastewater treatment plant in which it is possible to modify the sludge treatment and/or disposal process to eliminate a troublesome sidestream. For example, at wastewater treatment plants employing two-stage anaerobic digestion followed by mechanical dewatering, it will be possible to eliminate the variable-quality anaerobic digester supernatant by converting the secondary digester (i.e., unmixed digester) to a completely mixed system. Although this modification may increase the cost of digester mixing, by employing a completely mixed system, there would be no variable supernatant stream to treat. Moreover, the filtrate or centrate stream produced from the subsequent dewatering operations would be of low solids content and amenable to biological treatment.

5.9.1.2 Modification of upstream sludge processing steps. In many cases, modification of unit operations preceding sludge treatment can offer significant advantages in sidestream treatment and management.

For example, increased thickening of sludge prior to anaerobic digestion can improve the overall quality of sidestreams and reduce the quantity of digester supernatant. Liquor that otherwise would be produced in the secondary digester as supernatant is generated in the thickening step. The sidestream quality generated in the sludge thickener will be more amenable to biological treatment and will have a negligible impact on the wastewater treatment system [37,69]. Other factors that affect sidestream quality associated with anaerobic digestion include the organic loading and stabilization temperature. In general, an increase in digester organic loading will result in a poorer supernatant quality, whereas thermophilic digestion (140°F) produces poorer supernatant quality than does mesophilic digestion (95°F) [37].

Substitution of an equivalent sludge-treatment process for another also may reduce problems associated with sidestream management. For example, substitution of chemical conditioning for thermal conditioning will reduce the level of contaminants produced in the subsequent dewatering operations [37].

5.9.1.3 Changes in timing, return rate, or return point. Sidestreams normally are returned to the wastewater treatment facility at the plant headworks. In general, return of sidestreams at the headworks should be at a low, steady rate rather than in a slug load that can cause operational upsets downstream. Where high diurnal load fluctuations occur at the wastewater treatment plant, consideration should be given to returning sidestreams to the plant headworks during off-peak hours [28,37]. As illustrated in Fig. 5.61, minimizing adverse effects on the primary treatment system such as septicity, odors, and floating sludge can be achieved by returning certain sidestreams (e.g., filtrate or centrate from the dewatering of thermally conditioned sludge) to the secondary treatment process influent stream rather than the plant headworks.

5.9.1.4 Modification of wastewater treatment facilities. Wastewater treatment facilities should be designed with excess capacity sufficient to receive and treat recycled sidestreams whenever the sidestreams contain significant concentrations of pollutants or have a large hydraulic impact [7,18]. The sidestreams generated from sludge thickening, digestion, conditioning, and dewatering operations will recycle significant quantities of total suspended solids (TSS) and organic matter (BOD_5) to the wastewater treatment system. If the primary sedimentation systems are not hydraulically overloaded, the majority of suspended solids can be removed within the primary clarifier with no significant increase the in primary clarifier effluent suspended solids concentration. However, if the primary sedimentation tanks are

TABLE 5.17 Effect of Supernatant Return on Suspended Solids Loading*

Point of measurement for suspended solids	With supernatant return† (lb/day)	Without supernatant return (lb/day)
Raw wastewater	10,520	16,035
Primary clarifiers	36,801	15,969
Secondary clarifiers	15,306	9,501
Final effluent	3,467	2,836
Primary sludge	19,626	13,249
Waste-activated sludge	14,645	9,593

*Adapted from ref. [37].
†Supernatant returned ahead of primary clarifiers.

hydraulically overloaded and/or they are allowed to go septic, high levels of recycled BOD_5 and suspended solids could become a serious operational problem. Table 5.17 compares the effect of anaerobic digester supernatant return on the suspended solids loading at various locations within an activated-sludge wastewater treatment plant.

Although the bulk of suspended solids returned in sludge-processing sidestreams can be removed in primary sedimentation systems, this is not true for the soluble BOD_5 fraction of the recycled sidestream. The soluble organic recycled loading must be added to the anticipated primary effluent BOD_5 loading when determining the minimum capacity for secondary wastewater treatment systems.

In addition to suspended solids and BOD_5, sidestreams may contain compounds that are difficult to remove in wastewater treatment systems. For example, the nonbiodegradable chemical oxygen demand (COD) in the filtrate or centrate from the dewatering of thermally conditioned sludge will pass through normal secondary treatment systems. Digester supernatant also may contain high concentrations of nutrients (particularly phosphorus and ammonia) that could significantly impair wastewater effluent quality [34]. Potential options for treating these pollutants include (1) addition of advanced wastewater treatment systems (e.g., activated carbon, nitrification/denitrification, etc.) to the current wastewater treatment unit operations or (2) providing separate sidestream pretreatment.

5.9.1.5 Separate treatment of sidestreams. Anaerobic digester supernatant typically will contain significant quantities of suspended solids, BOD_5, nitrogen, and phosphorus [34]. In most cases, recycling and treatment of this sidestream to a preaeration tank results in its having a minimum impact on the overall wastewater treatment system. However, if the supernatant is reintroduced without any pretreatment to the primary sedimentation system, it could result in significant operational problems. For example, in addition to causing nuisance odors and septicity, when lime is being used as a chemical coagulant in the primary sedimentation system, introduction of anaerobic digester supernatant will cause significant increases in both chemical demand and sludge handling costs. The increase in chemical use stems from the reaction of lime with the carbon dioxide (CO_2) contained in the digester supernatant. This reaction not only requires increases in lime addition to achieve adequate coagulation but also results in the production of calcium carbonate ($CaCO_3$) precipitate, whose generation leads to a significant increase in primary sludge volume and management costs [34,37].

Pretreatment of digester supernatant is a potential option for mitigating the adverse effects of treating this sidestream. One possible pretreatment system may consist of air stripping of the CO_2 from the supernatant followed by lime–ferric chloride precipitation to reduce the suspended solids concentration. The addition of lime to the digester supernatant has the added benefit of raising the pH of the solution, facilitating nitrogen removal through enhanced ammonia volatilization [37]. Figure 5.62 illustrates the conceptual digester supernatant pretreatment system and the potential effects on digester supernatant quality.

Thermal conditioning of sludge produces liquor that, depending on the type of subsequent mechanical dewatering process used, may be referred as *decantate, filtrate,* or *centrate.* The thermal conditioning process enhances the dewatering characteristics of the sludge as well

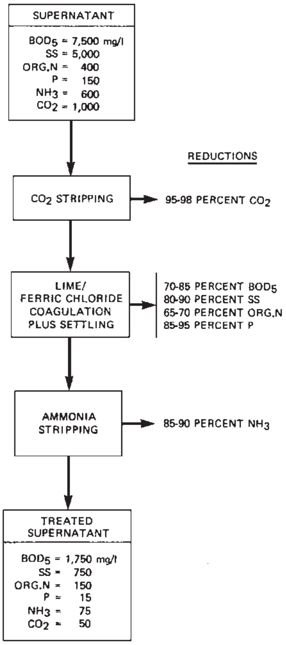


Figure 5.62 Pretreatment process for anaerobic digester supernatant.

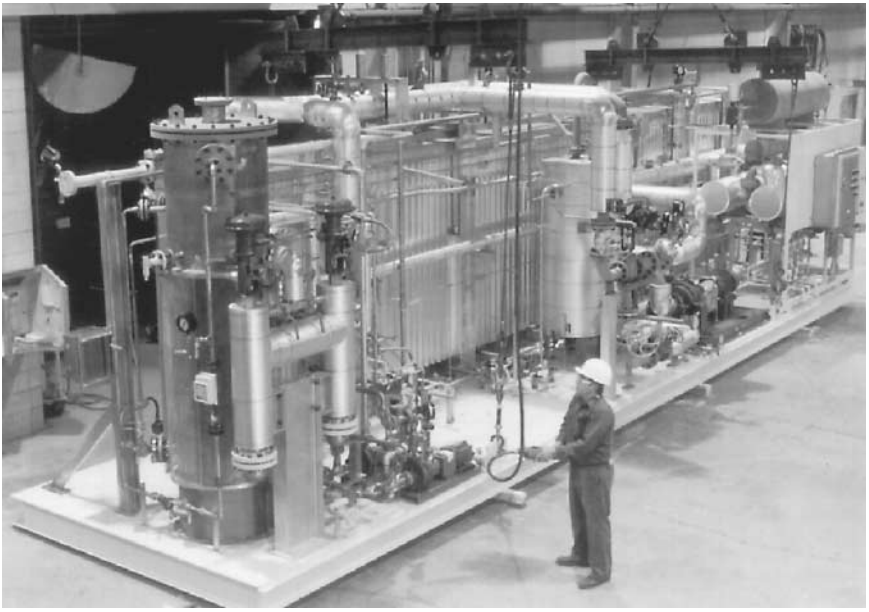


Figure 5.63 Photograph of skid-mounted wet-air oxidation system. (*Courtesy of Zimpro/Passavant Inc.*)

as increases pathogen destruction through the simultaneous application of heat and pressure [37]. Two basic thermal conditioning operations currently are employed in sludge treatment: (1) low-pressure oxidation (LPO process) and (2) heat treatment (HT process). Both operations are continuous-flow processes in which the sludge is heated to temperatures ranging from 177 to 204°C (250–400°F) in a reactor under pressures that range from 1720 to 2750 kPa (250–400 psig) for 15 to 40 minutes (Fig. 5.63). The principal difference in the two operations is that the LPO process introduces air into the conditioning system and the HT process does not (Chap. 3).

During the thermal conditioning process, a portion of the volatile suspended solids is solubilized as a result of the breakdown in the sludge structure [28,37]. Although this solubilization process does not change the total organic carbon content of the sludge, it does increase the soluble BOD_5 concentration, making treatment of this sidestream a major concern for secondary wastewater treatment operations. The solubilizing of volatile suspended solids and the resulting soluble BOD_5 generated from the subsequent mechanical dewatering systems may be estimated using Eqs. (5.29) and (5.30), respectively. Example 5.15 illustrates the use of Eqs. (5.29) and (5.30) in estimating the destruction of volatile suspended solids (VSS) and the BOD_5 generated as a result of thermal conditioning of sludge.

TABLE 5.18 General Characteristics of Liquor from Mechanical Dewatering of Thermally Conditioned Sludge*

Parameter	LPO process	HT process
Suspended solids, mg/liter	100–20,000	300–12,000
COD, mg/liter	10,000–30,000	2,500–22,000
BOD_5, mg/liter	5,000–15,000	1,600–12,000
Ammonia, mg/liter	400–1,700	30–700
pH	5.0–6.5	5.0–6.4

*Adapted from ref. [37].

$$\text{VSS} = 0.1 \cdot \text{PS} + 0.4 \cdot \text{WAS} \tag{5.29}$$

$$\text{BOD}_5 = 0.07 \cdot \text{PS} + 0.3 \cdot \text{WAS} \tag{5.30}$$

where VSS = volatile suspended solids solubilized during thermal treatment, kg (lb)
PS = primary sludge added, kg (lb)
WAS = waste-activated sludge added, kg (lb)
BOD_5 = 5-day biochemical oxygen demand produced by VSS solubilized, kg (lb)

In general, the composition of the filtrate, centrate, or decantate generated from the mechanical dewatering of thermally conditioned sludge is difficult to predict. The composition will be a function of the type of sludge, feed volatile solids concentration, reaction time, and temperature [37]. It is recommended that pilot testing be conducted in order to determine the sidestream characteristics and the optimal treatment strategy. Table 5.18 provides ranges of some of the constituents of concern contained in this sidestream.

Example 5.15 Woodson County Water Reclamation Facility is currently employing thermal treatment for conditioning its daily sludge flow of 16,000 lb (dry basis). If the thickened sludge flow consists of 60 percent primary sludge (PS) and 40 percent waste-activated sludge (WAS), estimate the pounds of volatile suspended solids (VSS) solubilized during the conditioning process and the increase in BOD_5 as a result of VSS solubilization.

solution

Step 1. Estimate the pounds of primary sludge and waste-activated sludge discharged to the thermal conditioning process:

$$\text{Primary sludge (PS)} = 0.6 \cdot 16{,}000 \text{ lb} = 9600 \text{ lb}$$

$$\text{Waste-activated sludge (WAS)} = 0.4 \cdot 16{,}000 \text{ lb} = 6400 \text{ lb}$$

Step 2. Estimate the pounds of VSS solubilized and the BOD_5 produced as a result of this solubilization using Eqs. (5.29) and (5.30):

$$\begin{aligned}\text{VSS} &= 0.1 \cdot \text{PS} + 0.4 \cdot \text{WAS} \\ &= 0.1 \cdot 9600 \text{ lb} + 0.4 \cdot 6400 \text{ lb} = 3520 \text{ lb VSS} \\ \text{BOD}_5 &= 0.07 \cdot \text{PS} + 0.3 \cdot \text{WAS} \\ &= 0.07 \cdot 9600 \text{ lb} + 0.3 \cdot 6400 \text{ lb} = 2592 \text{ lb BOD}_5\end{aligned}$$

Although trickling filters, activated-sludge, and aerobic digesters have been used to treat this liquor, to effectively reduce the nonbiodegradable organic matter, chemical treatment methods must be employed (e.g., activated carbon).

5.10 Odor Control

Most unit operations at wastewater treatment plants are potential sources of odors. If odorous gases such as hydrogen sulfide (H_2S) or ammonia (NH_3) are dissolved in wastewater entering the headworks of a wastewater treatment plant, turbulence induced by drops, flumes, aerated grit chambers, or similar structures will cause the odorous gases to be released from solution. Moreover, accumulation of organic debris in influent channels, on bar screens, in comminutors, and on fine screening devices can result in odor generation if regular cleaning and flushing are not practiced.

To minimize odors, bar screens and other preliminary treatment processes should be cleaned daily to remove any accumulated organic debris that can putrefy and cause odors. Grit- and screening-conveyance systems should be flushed with water to remove all solid materials. Removed solids should be transferred immediately to closed containers to minimize the escape of odors.

In addition to wastewater influent, high-organic-strength sidestreams from sludge-processing operations, such as wet oxidation decant liquors, filtrates, centrates, digester supernatants, etc., will release malodorous gases when exposed to the atmosphere. Most liquid sidesteams from sludge-processing operations contain large concentrations of BOD and ammonia and are thus major sources of odors. Sidestreams are often returned directly to the headworks of the wastewater treatment plant, where they may release odorous gases and/or cause odor generation through depletion of dissolved oxygen. In many instances, high-strength sidestreams require pretreatment before return to the wastewater treatment headworks [37].

5.10.1 Odors from primary and secondary wastewater treatment operations

Primary treatment operations can be a source of odors if unit operations (e.g., clarifiers) are improperly designed and/or maintained. If scum-removal mechanisms are inadequate, resulting scum accumulation and subsequent putrefaction will result in odor generation. Therefore, scum scrapers, pits, and grease-collection wells should be cleaned frequently and chemically treated, if necessary, to remove accumulated grease and scum. Figure 5.64 depicts a typical scum-removal system employed in primary clarifiers.

In addition to scum removal, infrequent or incomplete withdrawal of settled solids will cause septicity and the generation of odorous gases. Settled solids should not be allowed to accumulate in the bottom of clarifiers for periods exceeding 2 hours because septic conditions can develop resulting in odor generation [8,37,43].

With regard to secondary wastewater treatment systems, fixed-film reactors, such as trickling filters and rotating biological contactors, can be sources of odors when oxygen transfer to the biomass is inadequate. This situation occurs often during organic and/or hydraulic overload conditions. Plugging or improper sizing of underdrains in the

Figure 5.64 Scum-removal arm used in clarifier design.

trickling filter also can lead to odor generation by reducing oxygen availability. Other operational problems that may lead to odor generation in trickling filters include poor distribution of the wastewater over the media and discontinuous wetting that may lead to excessive slime accumulation and odor generation.

To minimize the generation of odors, the trickling filter media must be wetted continuously and kept free from plugging. During periods of low flow, this can be accomplished by increasing the effluent recirculation rate. Distribution nozzles should be kept clear at all times to allow uniform application of wastewater. Finally, filter underdrains and drain lines should be checked frequently to ensure that wastewater flow is not impeded.

Although suspended-growth secondary treatment systems typically are not significant sources of odor, the existence of poor mixing characteristics can result in deposition of organic solids in corners or along the edges of aeration basins. Accumulation of organic deposits will rapidly deplete dissolved oxygen and lead to the generation of odorous gases. In addition to poor mixing characteristics, periodic clogging of diffusers will lead to oxygen-limiting conditions and the generation of odors. In most cases, systems experiencing clogged diffusers will be characterized by the accumulation of foam, bubbles, or scum at the wastewater surface. In activated-sludge basins, sufficient and complete mixing is essential to prevent deposition of solids. Moreover, routine maintenance activities that will help minimize the deposition of solids in suspended growth treatment systems include inspection of air piping and diffusers.

Secondary clarifiers normally are not sources of odors if the upstream aerobic stabilization processes are properly designed and maintained. However, in wastewater treatment plants that experience significant rates of nitrification, algae accumulation on effluent weirs and/or ladders may lead to periodic odor production. Figure 5.65 is a photograph of a motorized brush-cleaning system used to remove algae and other attached growth on a secondary clarifier. Another major consideration in preventing odor generation in secondary clarifiers is maintenance of adequate sludge withdrawal rates. Sludge solids should not be allowed to accumulate within the secondary clarifier for periods of more than 4 hours to avoid septic conditions [37].

5.10.2 Odors in sludge processing

Sludge-processing systems are normally the most significant sources of odors in wastewater treatment plants. Although all sludges emit odors, fresher sludges generate less intense, less offensive odors. Septic sludges emit highly offensive and persistent odors. To minimize odor generation during sludge processing, care should be taken

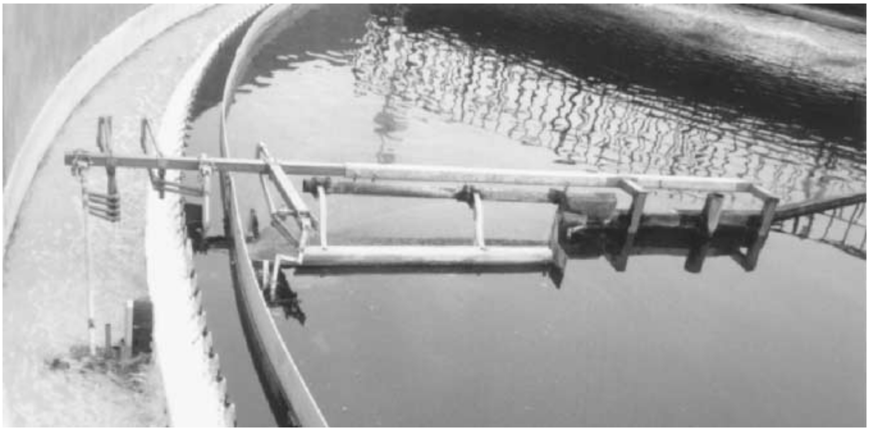


Figure 5.65 Cleaning brushes attached to a clarifier scum-removal arm.

to ensure that sludge transfer systems such as conveyors, screw pumps, and conduits are kept as clean as possible. Moreover, spillage should be cleaned and flushed immediately to prevent unnecessary odor generation.

Uncovered sludge thickeners and sludge holding tanks may generate intense odors and often are the cause of odor complaints from neighborhoods surrounding wastewater treatment plants. Wet oxidation processes can be a major source of odors unless special precautions have been taken to contain and treat the odorous emissions from the decant tanks.

Other sources of odors in sludge processing include sludge stabilization, storage, and dewatering systems. In most cases, odors generated during sludge stabilization are not highly offensive if the system is designed and operated properly. Overloaded aerobic digesters, however, will generate offensive odors. Similarly, lime stabilization processes (including lime pasteurization) may generate highly offensive odors due to the emission of large quantities of ammonia gas [37]. Although properly designed aerated static pile compost systems generate little odor if the gas is passed through piles of finished compost, windrow-composting systems may generate significant odors during pile turning. Sludge storage tanks, basins, and lagoons also may be significant sources of odors at wastewater treatment plants. Odor problems associated with storage systems are sometimes difficult to control because storage facilities are often uncovered and provide a large surface area that facilitates the production of odor emissions. Sludge dewatering processes also may be a source of odors. However, the extent of odor generation will vary depending on the type and characteristics of the sludge, the method used for dewatering, and the chemicals used for conditioning [37].

5.10.3 Septage handling

Septage receiving and handling facilities are major sources of odor at municipal wastewater treatment plants. Odors are often generated during transfer from septage hauling trucks to holding tanks at the wastewater treatment plant as well as during septage discharge to manholes, lift stations, headworks, or sludge-processing facilities. Uncontrolled addition of septage to the main wastewater influent stream also may result in rapid depletion of dissolved oxygen and subsequent odor generation.

To minimize odor generation, septage received at wastewater treatment plants should be transferred from the hauler truck into a closed tank or subsurface receiving basin by using quick-disconnect, watertight fittings. These types of fittings will reduce splashing, turbulence, and release of odors. Finally, provisions should be made for control of the rate of addition of the septage into the wastewater stream to avoid excessive dissolved oxygen depletion. Allowable loadings to the influent wastewater stream will depend on the aeration and solids-handling capacity of the plant as well as the characteristics of the septage.

5.10.4 Approaches to odor control

There are three general approaches to odor control: (1) prevention of odorous emissions, (2) collection and treatment of odorous air, and (3) odor modification, counteraction, and masking. In many cases, odor emissions can be reduced or eliminated through improved operation and maintenance practices. Regular and frequent cleaning of preliminary treatment devices such as comminutors, bar screens, and grit chambers, flushing of tank walls, removal of sludge deposits from influent and effluent process channels, and increased rate of withdrawal of settled solids are examples of routine operation and maintenance necessary to control odors [27,28,37]. Where high sulfide concentrations are present in influent wastewater, air or oxygen injection can be employed at the wastewater treatment plant headworks. In other cases, addition of hydrogen peroxide, chlorine, or metal salts (e.g., ferrous sulfate) at the headworks is effective in odor control. Where odors are generated in enclosed spaces such as covered wastewater treatment systems, sludge-processing buildings, covered sludge holding tanks, and/or wet wells, the odorous air can be treated effectively prior to release to the atmosphere by a variety of techniques, including wet scrubbers, activated carbon, chemical adsorbers, soil/compost filters, and bioscrubbers (Fig. 5.66).

Figure 5.66 Photograph of a wastewater treatment plant with covered primary, secondary, and sludge-treatment operations. (*Courtesy of CONSERVATEK, Inc.*)

5.10.4.1 Prevention of odor emissions. Good housekeeping is always effective in minimizing odor generation. Many odors associated with wastewater treatment operations can be controlled or eliminated by ensuring that process components are kept clean and free of accumulated grease, solids, and debris. In addition to good housekeeping, direct chemical addition to wastewater, sludge, or process sidestreams can be a simple and effective technique for odor control. Chemicals used for this purpose include (1) hydrogen peroxide, (2) potassium permanganate, (3) sodium nitrate, and (4) chlorine.

Under conditions typically found at wastewater treatment plants, hydrogen peroxide will oxidize odor-causing compounds such as H_2S by reactions described in Eqs. (5.31) and (5.32):

$$\text{pH} < 8.5: \qquad H_2O_2 + H_2S \rightarrow S^0 + 2H_2O \qquad (5.31)$$

$$\text{pH} > 8.5: \qquad 4H_2O_2 + S^{2-} \rightarrow SO_4^{2-} + 2H_2O \qquad (5.32)$$

In addition to oxidizing H_2S, excess hydrogen peroxide decomposes to yield molecular oxygen and water. Therefore, addition of hydrogen peroxide for odor control has the added benefit of increasing the dissolved oxygen level of the solution. Hydrogen peroxide is used frequently in sludge-handling systems, particularly sludge holding tanks and dewatering systems, to control odors [43].

Like hydrogen peroxide, potassium permanganate ($KMnO_4$) is a strong oxidizing agent. Potassium permanganate reacts with many odor-producing compounds, including (1) aliphatic compounds, (2) aromatic compounds, (3) nitrogen-containing compounds, (4) sulfur-containing compounds, and (5) inorganic compounds. Potassium

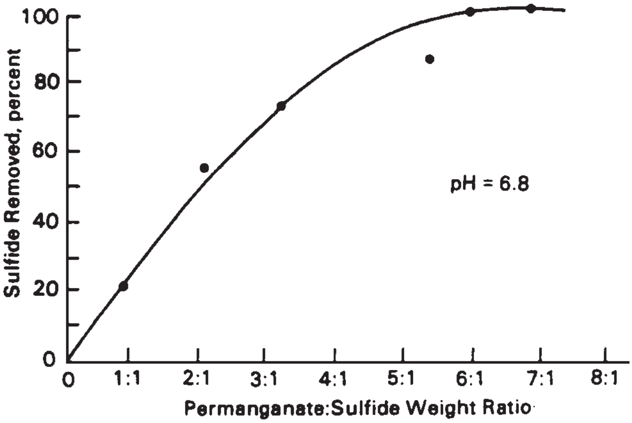


Figure 5.67 H_2S removal efficiency as a function of $KMnO_4$-to-H_2S weight ratio.

permanganate reacts with H_2S in either of two ways, as described by Eqs. (5.33) and (5.34):

Acidic conditions:

$$3H_2S + 2KMNO_4 \rightarrow 3S + 2H_2O + 2KOH + 2MnO_2 \qquad (5.33)$$

Alkaline conditions:

$$3H_2S + 8KMNO_4 \rightarrow 3K_2SO_4 + 2H_2O + 2KOH + 8MnO_2 \qquad (5.34)$$

$KMnO_4$ may be applied at various points in the liquid streams of the wastewater treatment plant. The required $KMnO_4$-to-H_2S weight ratio to achieve sulfide control in wastewater generally ranges from approximately 2.5:1.0 to 6.0:1.0 [43]. Figure 5.67 depicts the percentage sulfide removed as a function of the $KMnO_4$-to-H_2S weight ratio for a wastewater having a pH of 6.8.

Addition of potassium permanganate has been used successfully for odor control in sludge-handling applications, particularly dewatering, where it is added to the suction side of sludge pumps feeding the dewatering units. Figure 5.68 illustrates the relationship between dosage of $KMnO_4$ and the fraction of sulfide removed from sludge. A schematic diagram of potassium permanganate used upstream of a sludge dewatering process is shown in Fig. 5.69.

Like potassium permanganate, chlorine is a strong chemical oxidant whose application can be an effective means of odor control for wastewater unit processes. Chlorine application in wastewater treatment plants has been aimed primarily at preventing odors from the liquid streams. Addition of chlorine at the headworks is a common

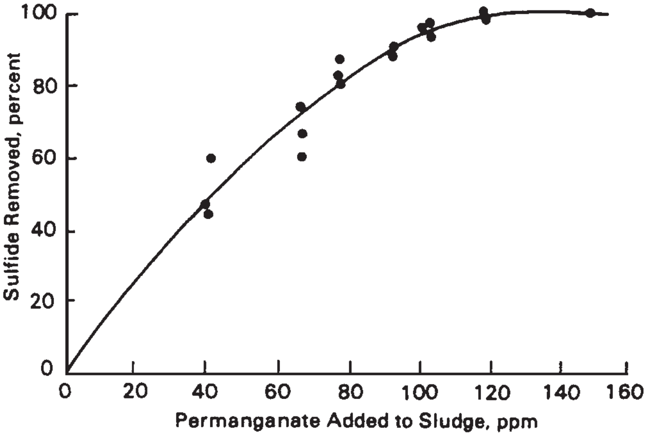


Figure 5.68 H_2S removal efficiency as a function of $KMnO_4$ dosage to sludge.

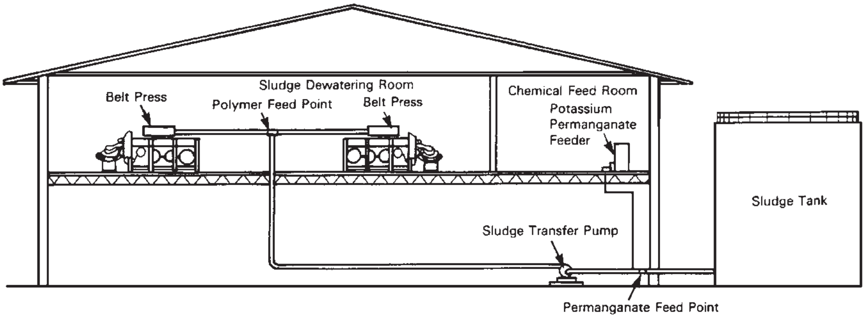


Figure 5.69 Schematic diagram of KMnO4 addition to control odors during sludge dewatering.

odor-control technique for many wastewater treatment plants. Chlorine will react with available sulfide according to the reaction described by Eq. (5.35):

$$HS^- + 4Cl_2 + 4H_2O \rightarrow SO_4^{2-} + 9H^+ + 8Cl^- \qquad (5.35)$$

The amount of chlorine required for odor control is typically less than 80 percent of the wastewater chlorine demand. Because of the high chlorine demand of wastewater sludges, chlorine typically is not used for controlling odors from sludge-handling operations.

Sodium nitrate ($NaNO_3$) has been used successfully for odor control in anaerobic lagoons, carbon columns, trickling filters, and sludge storage lagoons. The basis for adding $NaNO_3$ is the fact that bacteria will use electron acceptors preferentially in the order of (1) oxygen (O_2), (2) nitrate (NO_3), and (3) sulfate (SO_4). Therefore, in the absence of dissolved oxygen, no sulfide will be generated if nitrate is available in nonlimiting concentrations [43,52].

Finally, hydrogen sulfide also may be controlled by the addition of ferrous salts, particularly ferrous sulfate. The addition of ferrous sulfate to sulfide-containing wastewater results in the precipitation of ferrous sulfide, as described by Eq. (5.36):

$$H_2S + FeSO_4 \rightarrow FeS\,(s) + H_2SO_4 \tag{5.36}$$

Some of the benefits claimed in using ferrous sulfate over other chemical species for odor control include the fact that the material (1) specifically targets hydrogen sulfide for oxidation and (2) is much safer to store than hydrogen peroxide, potassium permanganate, or chlorine. Another benefit of using ferrous sulfate to control odors within the wastewater collection system or at the plant headworks is that it may indirectly facilitate the precipitation of phosphorous. The use of ferrous sulfate to control hydrogen sulfide results in the production of ferrous sulfide (FeS), which, when introduced to an aerobic (e.g., aerated grit chamber, activated-sludge system, etc.) environment, is rapidly converted to ferric sulfate [$Fe_2(SO_4)_3$]. Ferric sulfate will react to precipitate soluble phosphorous by the chemical reaction described in Eq. (5.37):

$$Fe_2(SO_4)_3 + 2Na_3PO_4 \rightarrow 2FePO_4(s) + 3Na_2SO_4 \tag{5.37}$$

5.10.5 Collection and treatment of odorous air

In cases where odors are generated from wastewater unit processes, such as primary clarifiers, sludge thickeners, and septage holding tanks, it is possible to construct covers or domes over the odor-generating units for the purpose of controlling odor emissions (Fig. 5.70). The contained air is then passed through an air pollution control device for odor removal prior to release to the atmosphere.

Domes generally are constructed of fiberglass, aluminum, or Styrofoam. Inflatable domes also have been employed. Aluminum domes are available with clear spans in excess of 400 ft (122 m; Fig. 5.71), whereas fiberglass domes can be used for covering tanks having diameters in excess of 90 ft (27 m). Flat, low-profile covers can be used for covering tanks that do not require frequent access for maintenance (e.g., sludge holding tanks with little or no mechanical equipment).

Covers should be designed so as to minimize condensation problems within the dome. In addition, covers must be designed to withstand wind loadings as well as static loadings resulting from snow and ice

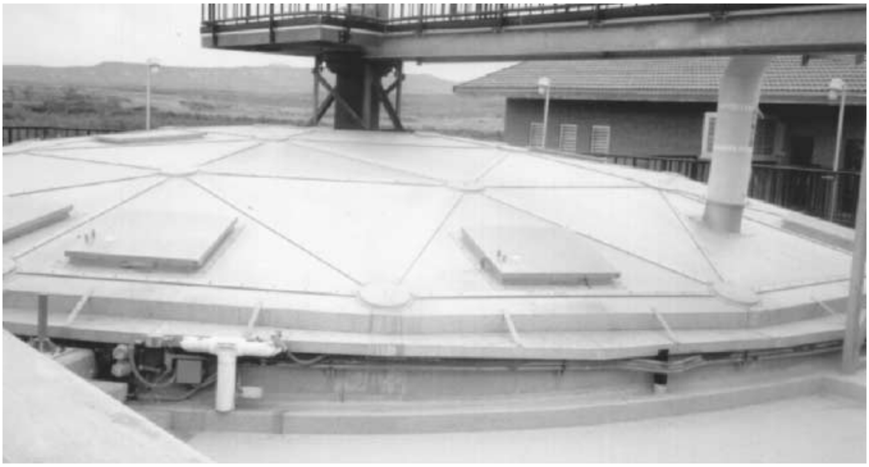


Figure 5.70 Photograph of an aluminum dome covering a sludge thickening tank.

Figure 5.71 Photograph of an aluminum dome covering a sludge storage tank. (*Courtesy of AQUASTORE Tanks.*)

accumulation. Negative pressures are maintained under the dome to prevent escape of odors though openings and cracks and to allow continuous exhausting of odorous air to odor treatment systems. Wet scrubbers, activated carbon, iron oxide filters, ozone contactors, combustion systems, soil/compost filters, and bioscrubbers all have been employed successfully to treat the odorous air.

Wet scrubbing (or absorption) involves contact of odorous gas with a scrubber solution. The scrubber facilitates transfer of the odor-causing compounds from the gas stream to the scrubber solution by one or more of the following mechanisms: (1) condensation of the odorous vapors, (2) removal of odorous particulates, (3) odor absorption into the scrubbing solution, (4) odor reaction with an oxidizing scrubbing solution, or (5) emulsification of odorous gases in a chemical reagent [8,43,55]. Full characterization of the contaminated air, including the identity of the major odorant compounds, their concentration, and

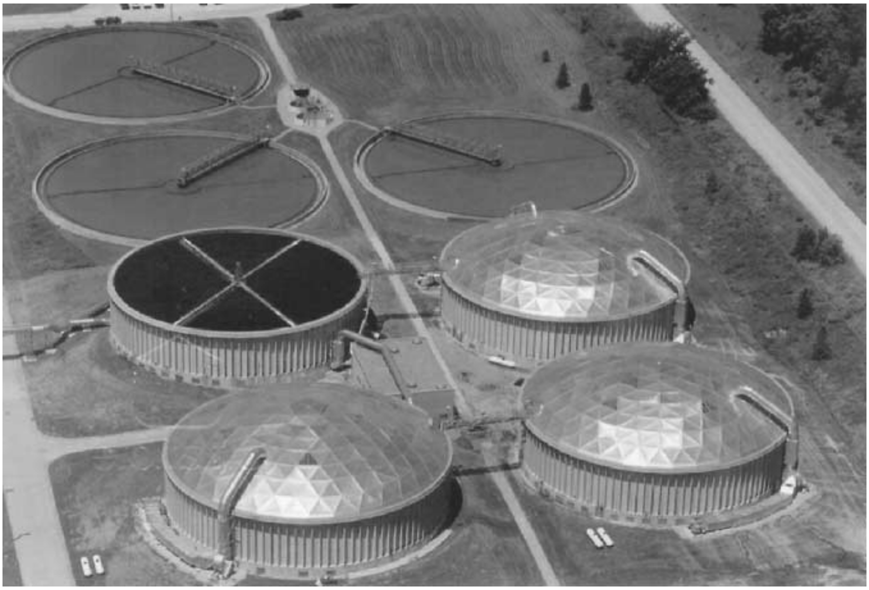


Figure 5.72 Photograph of an aluminum dome covering a trickling filter system. (*Courtesy of ULTRAFLOTE Corporation.*)

emission rate, is a critical first step in scrubber design. Diurnal fluctuations in odor intensity should be recorded to ensure that design parameters are based on peak odor emission levels [37].

Wet scrubber systems may be constructed as the vertically oriented countercurrent type or the horizontally oriented cross-flow or coflow (Venturi) type [8,55]. A typical countercurrent system employs spray nozzles for injection of the scrubbing solution and an inert packing material to provide a gas-liquid contact surface (Fig. 5.73).

In the countercurrent-type scrubber system, the gas stream enters the bottom of the scrubber unit and passes through the packing material that is irrigated from the top with the scrubber liquid. The purpose of the packing material is to increase the liquid-gas surface area, which effectively enhances the mass transfer of the odorant compound from the air to the liquid phase. Once the treated air leaves the packing material, it passes through a mist eliminator to remove any liquid droplets and is eventually exhausted to the atmosphere by a fan (Fig. 5.74).

Wet scrubber systems are also available that generate fine fogs or mists of scrubber liquid to obtain large surface areas for gas-liquid contact, thereby precluding the need for packing material. These systems are often referred to as *spray chambers*. Several variations of wet scrubber designs are shown in Fig. 5.75.

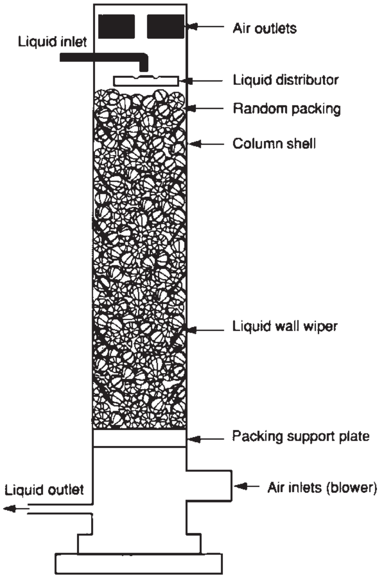
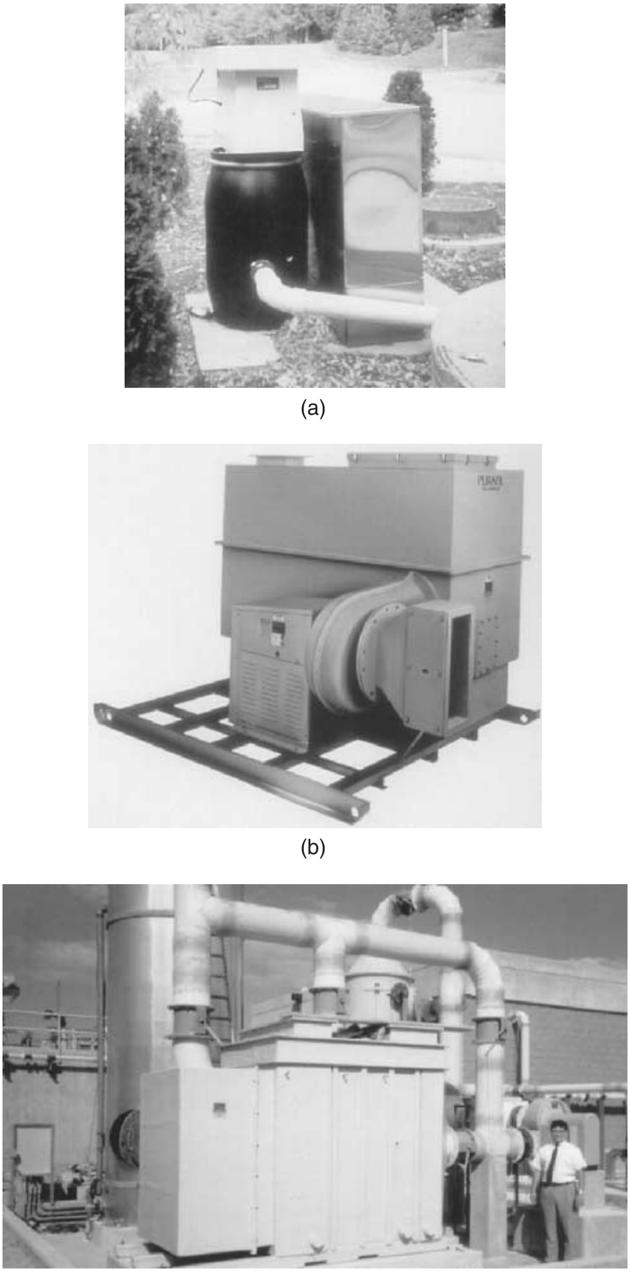


Figure 5.73 Schematic diagram of a typical countercurrent wet scrubber system. (*Adapted by permission from LaGrega et al., Hazardous Waste Management, McGraw-Hill, 1994.*)

Selection of a scrubbing liquid depends largely on the specific odor-causing compound to be removed. Water-soluble odorant compounds such as H_2S, ammonia, and organic acids may be removed using water as the scrubbing liquid. To enhance the removal of these compounds, it is common practice to add a chemical oxidant such as chlorine (typically in the form of sodium hypochlorite or chlorine dioxide), potassium permanganate, hydrogen peroxide, or ozone to the scrubbing liquid. Figure 5.76 depicts typical sodium hypochlorite and chlorine dioxide generation systems used on site in conjunction with wet scrubbers.

It should be noted that sodium hypochlorite (NaOCl) is generated from the electrolytic separation of a brine (e.g., sodium chloride) solution, whereas chloride dioxide is produced from the reaction of sodium chlorite and chlorine gas under acidic conditions [43]. Although chlorine gas is also effective in controlling odors and biological growth within wet scrubbers, sodium hypochlorite and chlorine dioxide are preferred sources of chlorine given the significant health and safety hazards associated with handling chlorine gas. Table 5.19 provides data on the expected performance of sodium hypochlorite scrubbers for removal of various odorant compounds.

In addition to single-stage wet scrubbers, multistage scrubber systems are often employed for odor control. The number of stages and choice of scrubber liquid depend on the characteristics and intensity of the odor and the effectiveness of the particular chemical additives in the scrubber water [19,43]. A typical multistage wet scrubber system for the removal of H_2S is depicted in Fig. 5.77.



(a)

(b)

(c)

Figure 5.74 Photographs of scrubber systems: (*a*) drum scrubber; (*b*) tub scrubber; (*c*) multistage scrubber. (*Courtesy of PURAFIL, Inc.*)

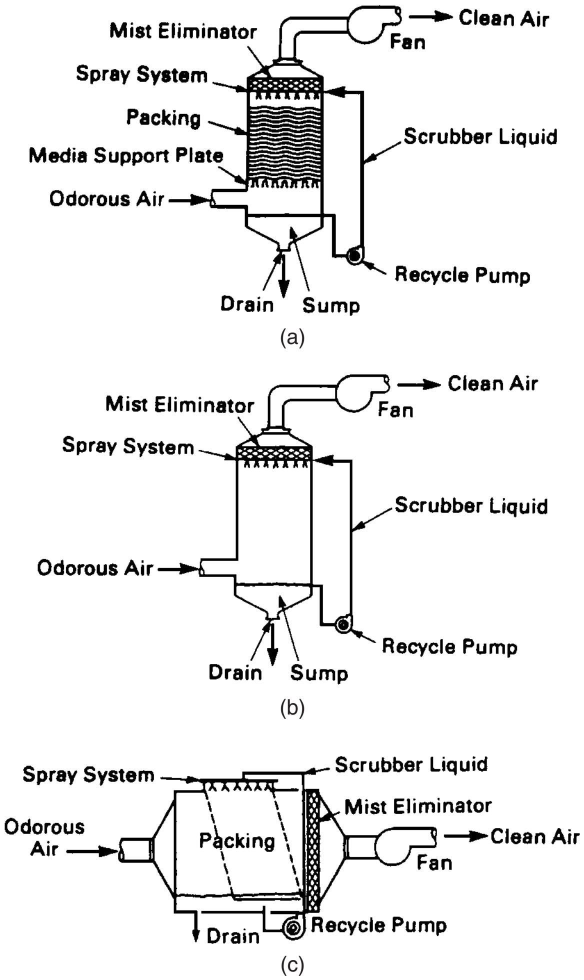


Figure 5.75 Schematic diagrams of various wet scrubber designs: (*a*) countercurrent packed tower; (*b*) spray chamber absorber; (*c*) cross-flow scrubber.

Equipment requirements for a wet scrubber system depend on the type of scrubber used and the scrubber liquid employed, volume and concentration of odorous air, and desired removal efficiency. A typical single countercurrent scrubber uses an enclosed tower containing packing material, a medium support plenum, a scrubber solution distribution system, a mist eliminator, and an exhaust fan. Various types of packing materials used in countercurrent scrubber systems are depicted in Fig. 5.78.

Although the theoretical design of a wet scrubber system is beyond the scope of this text, several important considerations

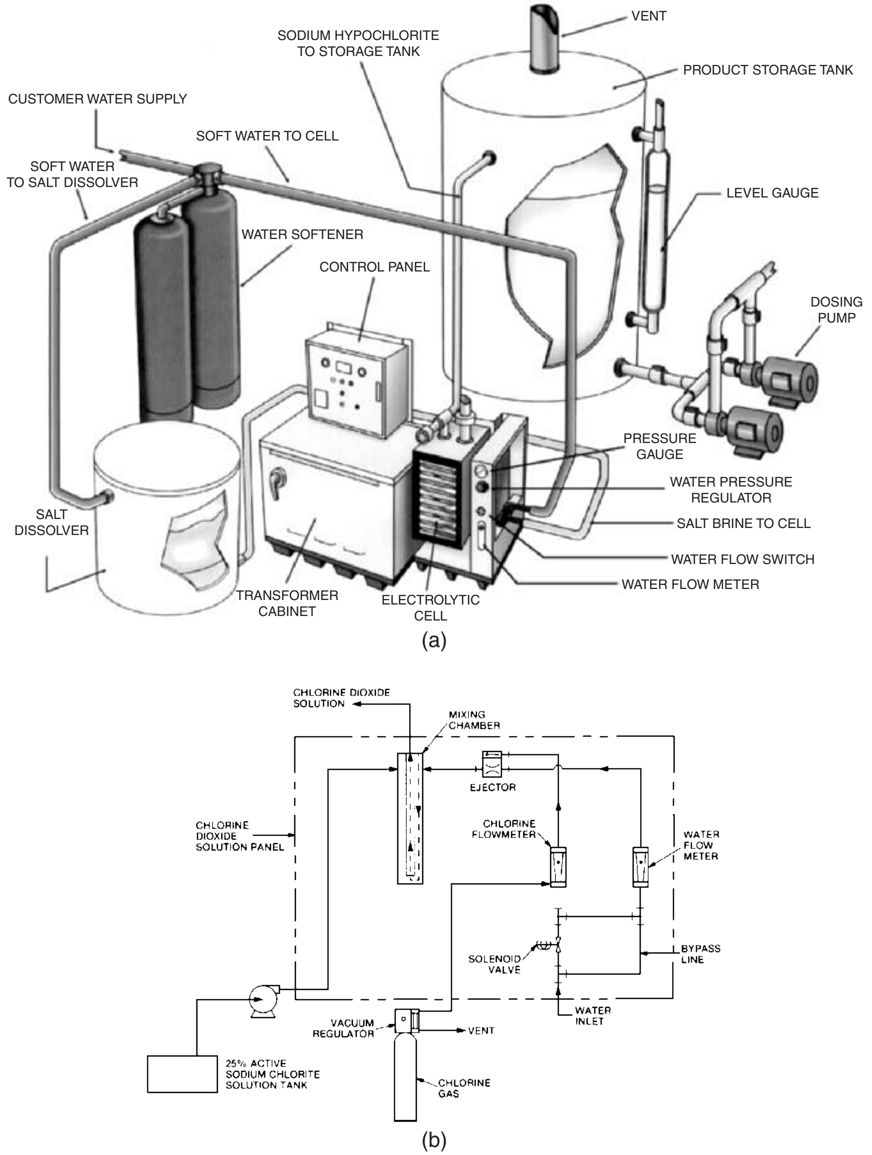


Figure 5.76 (*a*) Diagram of a sodium hypochlorite generation system used in conjunction with wet scrubbers. (*Courtesy of Exceltec International Corporation.*) (*b*) Schematic diagram of a chlorine dioxide generation system. (*Courtesy of Waterford Systems, Inc.*)

TABLE 5.19 Effectiveness of Hypochlorite Wet Scrubbers for Removal of Odorous Gases*

Gas	Removal efficiency
H_2S	98
Ammonia	98
SO_2	95
Mercaptans	90

*Adapted from ref. [43].

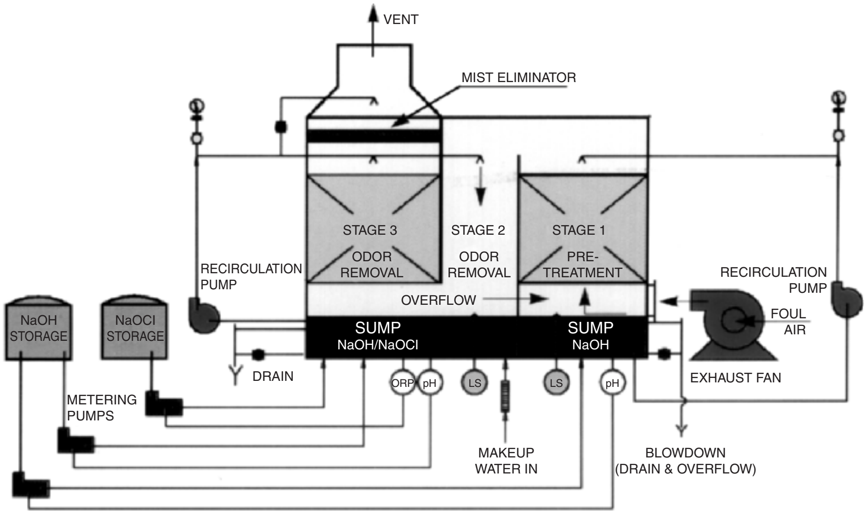
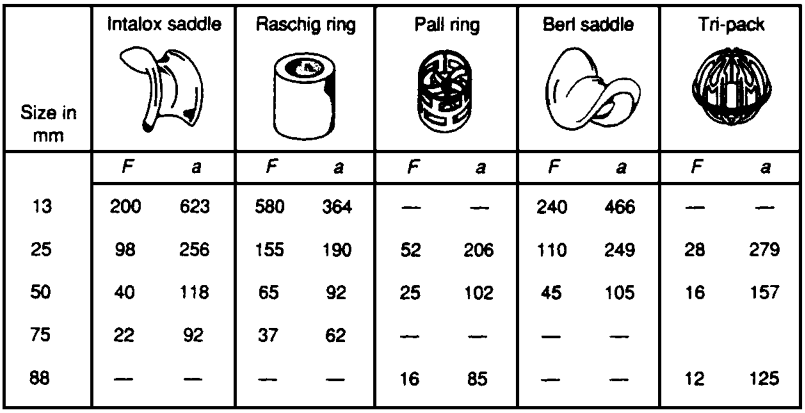


Figure 5.77 Diagram of a multistage scrubber system for H_2S removal. (*Courtesy of US Filter/RJ Environmental Products.*)

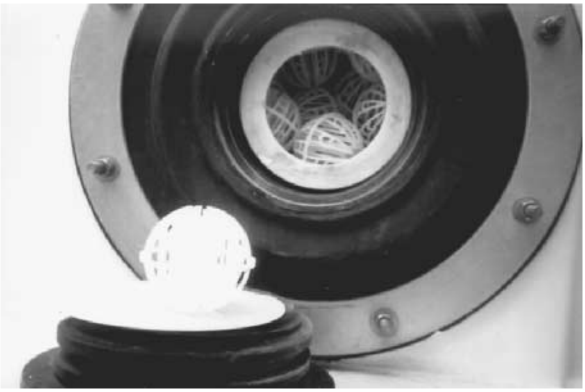
should be highlighted in approaching the overall design. First, the minimum and design liquid-to-gas-flow ratio must be established for effective odor control by evaluating a laboratory- or pilot-scale system. Once the range of permissible liquid-to-gas-flow ratios is determined, the minimum height of the scrubber can be estimated by consideration of the overall resistance to mass transfer between the gas and liquid phases, the average mass-transfer driving force, and the total interfacial area available for mass transfer. Finally, after the minimum height of the scrubber has been calculated, the diameter of the scrubber may be estimated by evaluating the impact of the packing configuration on the unit pressure drop (i.e., pressure drop per foot of packing height). The results of laboratory and or pilot scale studies are then used to develop the full-scale scrubber design (Fig. 5.79).



Size in mm	Intalox saddle		Raschig ring		Pall ring		Berl saddle		Tri-pack	
	F	a	F	a	F	a	F	a	F	a
13	200	623	580	364	—	—	240	466	—	—
25	98	256	155	190	52	206	110	249	28	279
50	40	118	65	92	25	102	45	105	16	157
75	22	92	37	62	—	—	—	—		
88	—	—	—	—	16	85	—	—	12	125

F = Packing factor (dimensionless); a = area/volume ratio (m^2/m^3). Data for ceramic shapes except Pall Ring and Tri-pak® data for plastic shape.

(a)



(b)

Figure 5.78 (*a*) Types of packing materials used in wet scrubber systems. (*b*) Photograph of packing material placed in a wet scrubber.

5.10.5.1 Activated-carbon adsorption. Activated-carbon adsorption (sometimes referred to as *dry scrubbing*) is a commonly used method for treatment of malodorous air. Activated carbon has been used in wastewater treatment plants as a primary odor-control system and as a polishing step following other odor-control systems such as wet scrubbers. Activated carbon has a high surface-area-to-volume ratio and therefore is an ideal material for odor adsorption. The physical characteristics of activated carbon used for odor control are provided in Table 5.20.

Owing to the nonpolar characteristics of its surface, activated carbon adsorbs organic and some inorganic compounds in preference to water. The quantity of odorant materials adsorbed depends on the

Figure 5.79 Photograph of a full-scale wet scrubber system used for odor control.

TABLE 5.20 Physical Characteristics of Activated Carbon Used for Odor Control*

Parameter	Value
Surface area, m^2/g	950
Surface area, m^2/cm^3	380–600
Pore volume, cm^3/g	0.6–1.0
Pore volume, cm^3/cm^3	0.24–0.5
Mean pore diameter, angstroms†	15–20

*Adapted from refs. [8,43].
†Angstrom $= 1 \cdot 10^{-10}$ m.

following factors: (1) concentration of odor-causing compound(s), (2) total surface area of the activated carbon, (3) total pore volume, (4) temperature, (5) presence of competing contaminants, (6) characteristics of odor-causing compounds (e.g., molecular weight, boiling point, polarity, etc.), (7) polarity of activated carbon, (8) relative humidity of the air stream, and (9) contact time [8,19,43].

The nonselectivity of activated carbon is a desirable characteristic in that activated-carbon systems have the ability to remove complex mixtures of odorous compounds. However, nonselectivity can result in higher operational costs, since the capacity of the carbon can be exhausted prematurely by the adsorption of nonodorous hydrocarbons. In some cases, a special activated carbon impregnated with caustic (NaOH or KOH) is specified for odor-control applications. When compounds such as H_2S are adsorbed on the carbon surface, the caustic chemical reacts with the adsorbed odorant compounds to form elemental sulfur and/or sulfate.

Relatively little mechanical equipment is required with an activated-carbon odor-control system. A typical activated-carbon odor-control system would consist of air pretreatment units for removing grease and/or water vapor (optional), activated-carbon adsorber unit(s), exhaust fans(s), and associated piping and ductwork (Figs. 5.80 and 5.81).

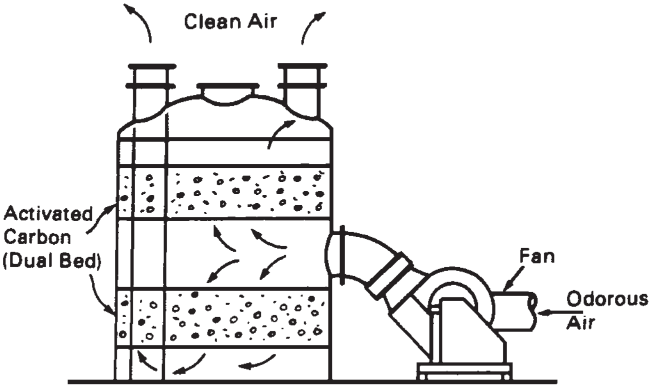


Figure 5.80 Schematic diagram of an activated-carbon adsorption system.

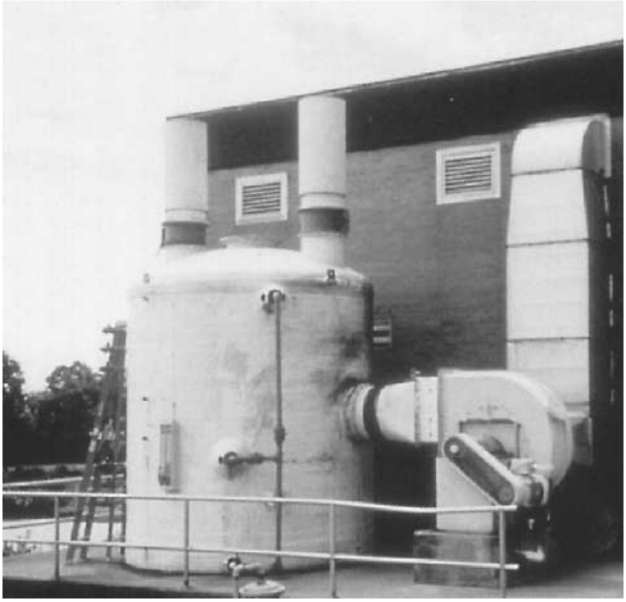


Figure 5.81 Photograph of an activated-carbon adsorption system. (*Courtesy of Environmental Systems Division PURAFIL, Inc.*)

An operational benefit of carbon adsorption is that once exhausted, the carbon can be regenerated and reused. Activated carbon typically is regenerated thermally, although special activated carbon impregnated with KOH or NaOH can be regenerated chemically using a 50 percent solution of KOH or NaOH. An advantage of chemical regeneration is that it can be performed in situ, whereas thermal regeneration requires removal and transfer of the carbon from the adsorption system to a furnace.

As with wet scrubber systems, full characterization of the odorous air is critical in the design of activated-carbon odor control systems. Of particular importance is the diurnal variation in air volumes and odor intensity. Failure to accurately estimate these parameters may result in poor performance or higher than anticipated carbon replacement/regeneration frequency.

In addition to quantification of specific odorous compounds (e.g., methyl mercaptan), an olfactometer should be employed as a measure of total odor removal by activated-carbon systems. Odor breakthrough curves should be developed to assist in determining the frequency of carbon regeneration or replacement. An initial approximation of carbon life before odor breakthrough occurs may be estimated using Eq. (5.38). Example 5.16 illustrates the use of Eq. (5.38) for the preliminary design of a carbon adsorption system.

$$t = \frac{XW_a}{QC} \tag{5.38}$$

where t = breakthrough time, days
X = carbon adsorption capacity (mass of odorant/mass of carbon), typically 0.16 to 0.5
W_a = mass of carbon in bed, lb
Q = air flow rate, ft^3/day
C = concentration of odorant in gas, lb/ft^3

Since exhaust air from sludge-handling buildings, wet wells, and covered process tanks often contains a complex mixture of odorous compounds, laboratory/pilot testing of the activated-carbon system is essential prior to full-scale design. At a minimum, the following objectives should be addressed as part of laboratory/pilot-scale system testing: (1) define expected performance (i.e., odorant removal efficiency), (2) estimate the useful life of the carbon, (3) determine the effectiveness of carbon regeneration, and (4) develop design criteria for the full-scale system.

Example 5.16 The Turkana County Water Reclamation Plant is currently directing 9.6×10^6 ft^3/day of air from its sludge conditioning/dewatering building to a series of activated-carbon beds for treatment prior to discharge to the atmosphere. If the air contains an average of 20 parts per million of H_2S (volume basis), estimate the minimum number of pounds of carbon needed if it is desired that the carbon beds last at least 60 days before they are exhausted (i.e., breakthrough of the last bed). Assume that the following conditions apply:

1. Carbon adsorption efficiency: 0.25
2. Molecular weight of H_2S: 34 lb/lb·mol
3. Molar volume of air at operating conditions: 386 ft^3/lb·mol

solution

Step 1. Convert H_2S concentration from ppmv to pounds per cubic foot (lb/ft^3):

$$\text{H}_2\text{S concentration (lb/ft}^3) = \frac{20 \text{ lb} \cdot \text{mol H}_2\text{S}}{1 \cdot 10^6 \text{ lb} \cdot \text{mol air}} \cdot \frac{\text{lb} \cdot \text{mol air}}{386 \text{ ft}^3} \cdot \frac{34 \text{ lb H}_2\text{S}}{\text{lb} \cdot \text{mol H}_2\text{S}}$$

$$= \frac{1.76 \cdot 10^{-6} \text{ lb H}_2\text{S}}{\text{ft}^3}$$

Step 2. Use Eq. (5.38) to estimate the minimum number of pounds of carbon needed:

$$t = \frac{XW_a}{QC}$$

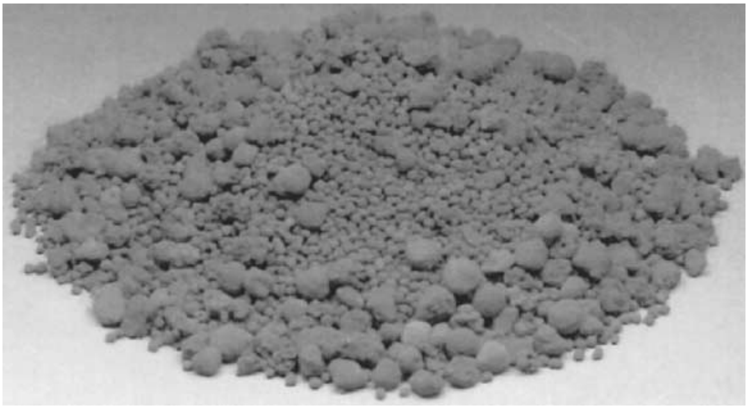
$$60 \text{ days} = \frac{0.25W_a}{9.6 \cdot 10^6 \text{ft}^3/\text{day} \cdot 1.76 \cdot 10^{-6} \text{ lb H}_2\text{S/ft}^3}$$

or,

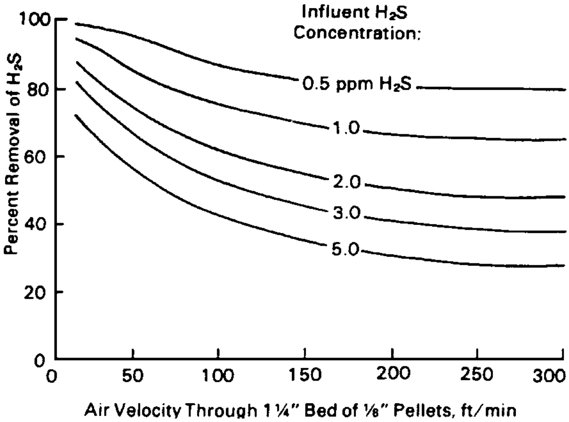
$$W_a = 4055.04 \text{ lb of carbon}$$

5.10.5.2 Dry scrubbing systems. In addition to activated carbon, other adsorptive or dry scrubbing media used for odor control at wastewater treatment plants include activated alumina and woodchips mixed with iron oxide. With regard to activated alumina, several commercial products are available that consist of activated alumina pellets impregnated with potassium permanganate (Fig. 5.82*a*). Contaminated air is passed through a series of shallow beds containing the media, after which it is exhausted to the atmosphere. Odorous compounds are adsorbed onto the surface of the pellets and are subsequently oxidized by the potassium permanganate. Packaged systems typically are horizontal- or vertical-flow units employing a prefilter and/or mist eliminator for removal of particulates and moisture.

The potassium permanganate–impregnated activated-alumina pellets have a finite capacity for removal of odorous compounds. When the capacity is exhausted, the media are discarded. The useful life of the



(a)



(b)

Figure 5.82 (*a*) Photograph of activated-alumina pellets impregnated with $KMnO_4$. (*Courtesy of PURAFIL, Inc.*) (*b*) Removal of H_2S by $KMnO_4$-impregnated activated alumina.

media depends on the total mass throughput of the odorous contaminant. Pellet diameters are typically 1/8 to 3/8 in (3–9 mm) and contain approximately 5 percent potassium permanganate by weight. The efficiency of $KMnO_4$-impregnated activated alumina to remove H_2S is depicted in Fig. 5.82*b*.

Iron oxide filters (often referred to as iron sponges) have been employed successfully to control odors. Such systems typically incorporate a bed of woodchips mixed with iron oxide (Fe_2O_3). In iron oxide filter systems, contaminated air is passed upward through the bed, where odorous contaminants are adsorbed by the media. A schematic diagram of an iron oxide filter bed is provided in Fig. 5.83.

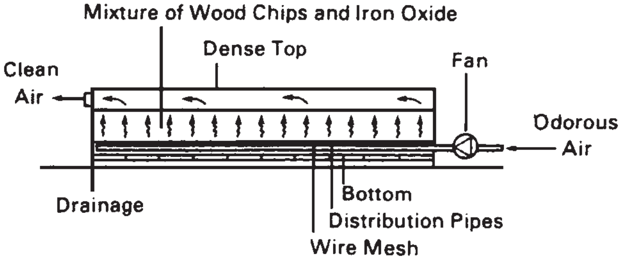


Figure 5.83 Schematic diagram of an iron oxide filter bed.

For H_2S-contaminated air, Eq. (5.39) describes the primary odor-removal mechanism in an iron oxide filter bed. In addition to H_2S removal, iron oxide filters have been found to be effective in controlling ammonia typically generated from sludge-stabilization processes.

$$Fe_2O_3 + 3H_2S \rightarrow Fe_2S_3 + 3H_2O \tag{5.39}$$

5.10.5.3 Ozone contactors. Ozone (O_3) is a strong oxidant that has been employed successfully for odor control. Examples of ozone oxidation reactions in odor-control applications are described by Eqs. (5.40) through (5.42).

Hydrogen sulfide:

$$H_2S + O_3 \rightarrow S + H_2O + O_2 \tag{5.40}$$

Amine:

$$R_3N + O_3 \rightarrow \underset{\text{Amine oxide}}{R_3N\text{-}O} + O_2 \tag{5.41}$$

Methyl mercaptan:

$$CH_3SH + O_3 \rightarrow \underset{\text{Methyl sulfonic acid}}{CH_3\text{-}SO_3H} + O_2 \tag{5.42}$$

Since ozone is an unstable gas, its use requires generation on site. Most ozone-generating systems employ the corona discharge method to produce ozone from feed gases. In this method, oxygen or preheated air is passed through a discharge gap across which a high voltage is maintained (Fig. 5.84*a*). As a result of the high voltage, high-energy electrons bombard oxygen molecules, resulting in the formation of radical species that combine to form ozone [43].

For economical operation of an ozone generator, heat must be removed continuously from the system, and the feed gas must be clean, cool, and dry. Water and impurities in the influent gas can cause generation of fouling agents that may coat the dielectrics, thus lowering ozone production and increasing power consumption. Moisture in

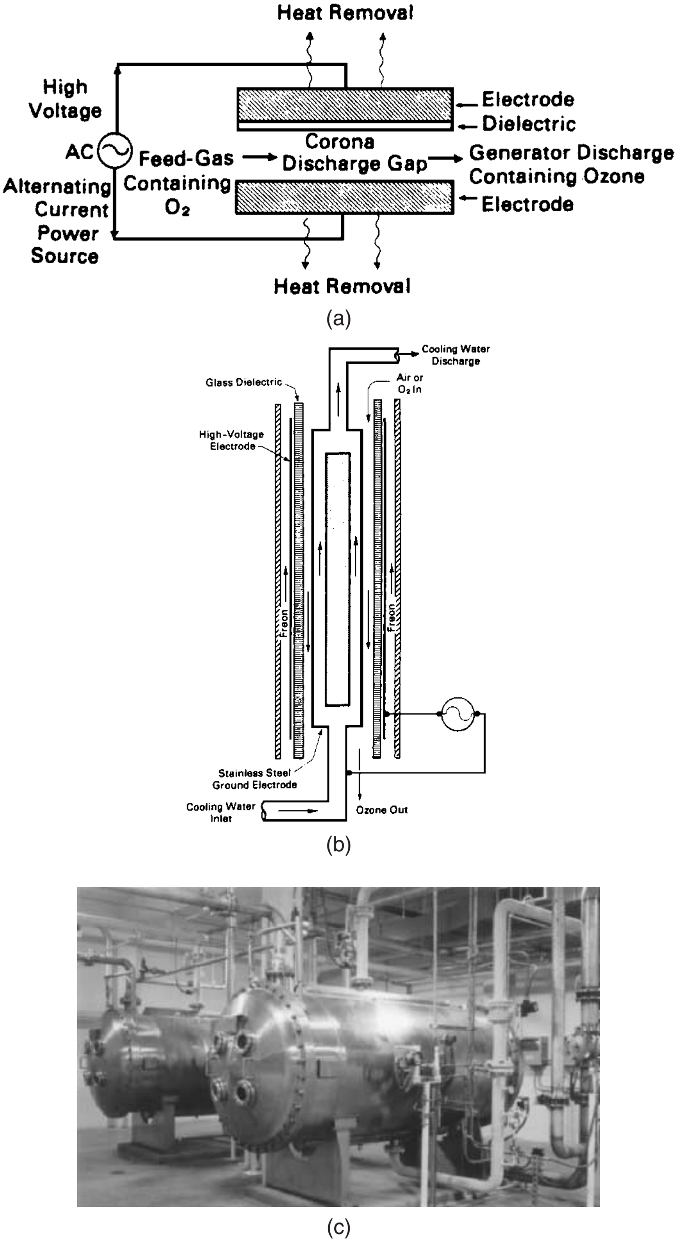


Figure 5.84 (*a*) Corona discharge ozone generation. (*b*) Vertical-tube ozone generator system. (*c*) Photograph of ozone generator. (*Courtesy of Ozonia North America.*)

the feed gas also may cause formation of nitric acid in the corona, which is corrosive and may result in reduced efficiency and increased maintenance costs (see Fig. 5.84*b*).

To use air as the oxygen source for ozone processing requires the following equipment: (1) compressors, (2) heat exchangers, and (3) various sized filter units (Fig. 5.84*c*). A typical air filtration system may include an air-conditioning filter, a 50-μm filter for the compressor intake, a 5-μm filter, and a 4-Å molecular sieve. The molecular sieve is used for removal of water vapor, carbon monoxide, carbon dioxide, nitrogen, methane, and H_2S. Use of pure oxygen as the feed gas precludes the need for gas pretreatment.

The effluent gas from an ozone generator contains ozone at relatively low concentrations (typically 1 to 2 percent by weight). To control odors, the effluent gas stream from the ozone generator is introduced into a baffled contact chamber, where it is allowed to mix with the malodorous gases (Fig. 5.85).

Reaction times in the contact chamber (which is typically a fiberglass-reinforced plastic baffled chamber) may vary considerably. Although contact times as short as 7 seconds have been used, contact times of 30 to 40 seconds are recommended [43]. The required detention time depends on the type of odor and its concentration.

Normally, 3 to 4 parts per million (volume basis, ppmv) of ozone fed into the odorous gas stream is sufficient to control odors. However, this concentration may need to be modified based on the specific application. For example, exhaust air from sludge storage tanks and dewatering rooms may require 10 ppmv of ozone or more. A general guideline for H_2S oxidation is that 1 ppmv ozone will oxidize 10 ppmv of H_2S [43].

Given the necessary ozone dosage for oxidation of odorous compounds and the airflow rate, Fig. 5.86 can be used to estimate the ozone requirement. For example, for an airflow rate of 5000 acfm (140

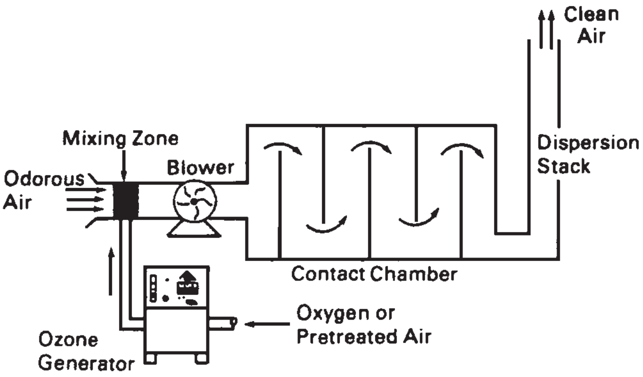


Figure 5.85 Schematic diagram of an ozone contact chamber.

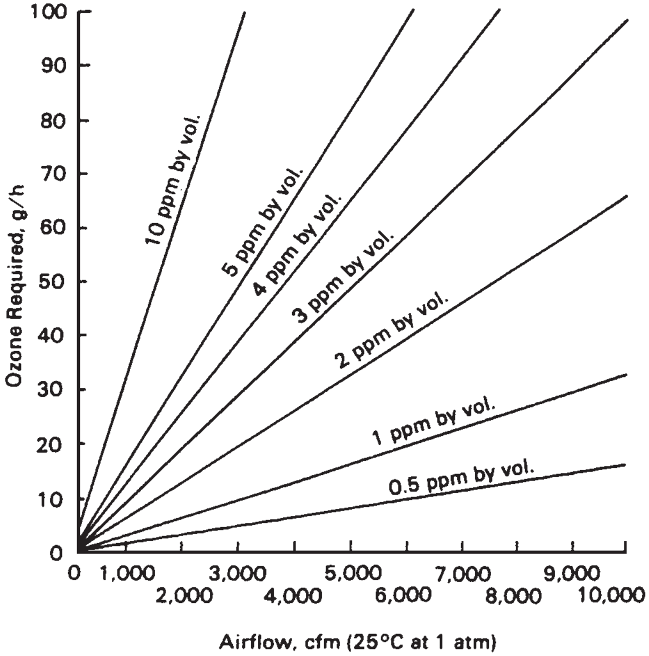


Figure 5.86 Preliminary design estimates of ozone requirements for odor control.

m^3/min) and required ozone dosage of 3 ppmv, the ozone requirement would be approximately 50 g/h, or 2.6 lb/day (1.2 kg/day).

Design criteria for an ozone odor-control system include (1) type of odor, (2) odor concentration, (3) temperature of exhaust gases, (4) humidity of exhausts gases, (5) retention time within the contact chamber, and (6) ozone distribution within the contact chamber. In many cases, the feasibility of ozone oxidation for odor control can be established in the laboratory. However, on-site pilot testing is often necessary to accurately determine the value of design parameters.

Another important design consideration for ozone odor control includes the health and safety impacts associated with ozone generation. The Occupational Safety and Health Administration (OSHA) has set a maximum 8-hour continuous exposure level of 0.1 ppmv for ozone. Using ozone monitors, ozone dosages to the odorous air stream should be adjusted to minimize the discharge of unreacted ozone.

5.10.5.4 Combustion processes. Combustion is an effective means of odor control. During this high-temperature oxidation process, hydrocarbons are oxidized to carbon dioxide and water, while nitrogen and sulfur-bearing compounds are oxidized to oxides of nitrogen and sulfur, respectively. To be effective in odor control, combustion systems should result in the complete oxidation of odorous compounds; otherwise, intermediate products may be formed that are also malodorous.

Two types of combustion processes are used in odor-control applications: (1) direct-flame oxidation and (2) catalytic oxidation. In the direct-flame process, odorous gas is mixed with combustion air and heated to temperatures of 900 to 1500°F (480–815°C) in the presence of a combustion flame. Combustion chamber residence times of 0.25 to 0.6 second normally result in effective odor control [43,53]. To ensure oxidation of the incoming odorous compounds, incoming odorous air should have a heating value of at least 100 Btu/ft^3 (3700 kJ/m^3). The heating value of the incoming gas may be estimated using Eq. (5.43).

$$\text{Btu/ft}^3 = 0.075\,[(145.4)\,\text{C} + (620)\,\text{H} - (0.125)\,\text{O}_2 + (41)\,\text{S}] \qquad (5.43)$$

where Btu = British thermal unit (i.e., energy required to raise 1 lb of water 1°F)

0.075 = density of air at normal conditions, i.e., 68°F and 1 atm pressure (lb/ft^3)

C, H, O_2, S = weight percentages of carbon, hydrogen, oxygen, and sulfur, respectively

Small amounts of fuel may be added to the influent air stream if the odorous gas is below the lower flammable limit. A conventional direct-flame oxidation system is shown schematically in Fig. 5.87.

In the catalytic oxidation process, the presence of a catalyst allows oxidation of the odor-causing compounds to occur at lower temperatures than in the direct-flame system, resulting in significantly lower energy costs. Because of the heat released during the oxidation of the odor-causing compounds, a temperature gradient will exist across the catalyst bed. In general, the temperature of the air upstream from the catalyst bed is 600 to 900°F (315–480°C) and 800 to 1100°F (425–590°C) downstream from the catalyst. A schematic diagram of a catalytic oxidation system is shown in Fig. 5.88.

The catalyst employed in these odor-control systems typically consists of finely divided platinum or palladium metal implanted on a ceramic supporting structure. Although platinum or palladium typically is used, other metals such as nickel, copper, chromium, and manganese also may be employed as catalysts.

One major concern with catalytic combustion systems is the potential poisoning or fouling of the catalyst from contaminants in the incoming air. Table 5.21 lists various materials whose presence can result in poor performance of the catalyst. It should be noted that although a catalytic oxidation system will require less fuel than a direct-flame oxidation system, it may have significantly higher maintenance costs.

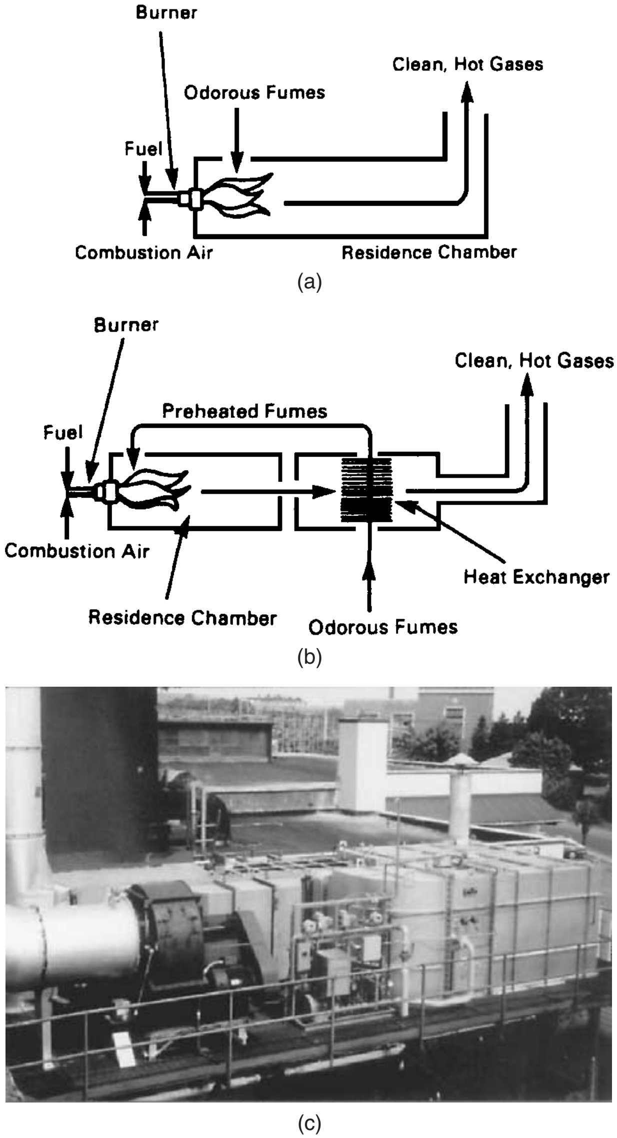


Figure 5.87 Schematic diagrams of direct-flame odor oxidation systems: (*a*) conventional system; (*b*) direct flame with heat recovery; (*c*) roof-mounted thermal oxidizer unit designed for odor control. (*Courtesy of Process Combustion Corporation.*)

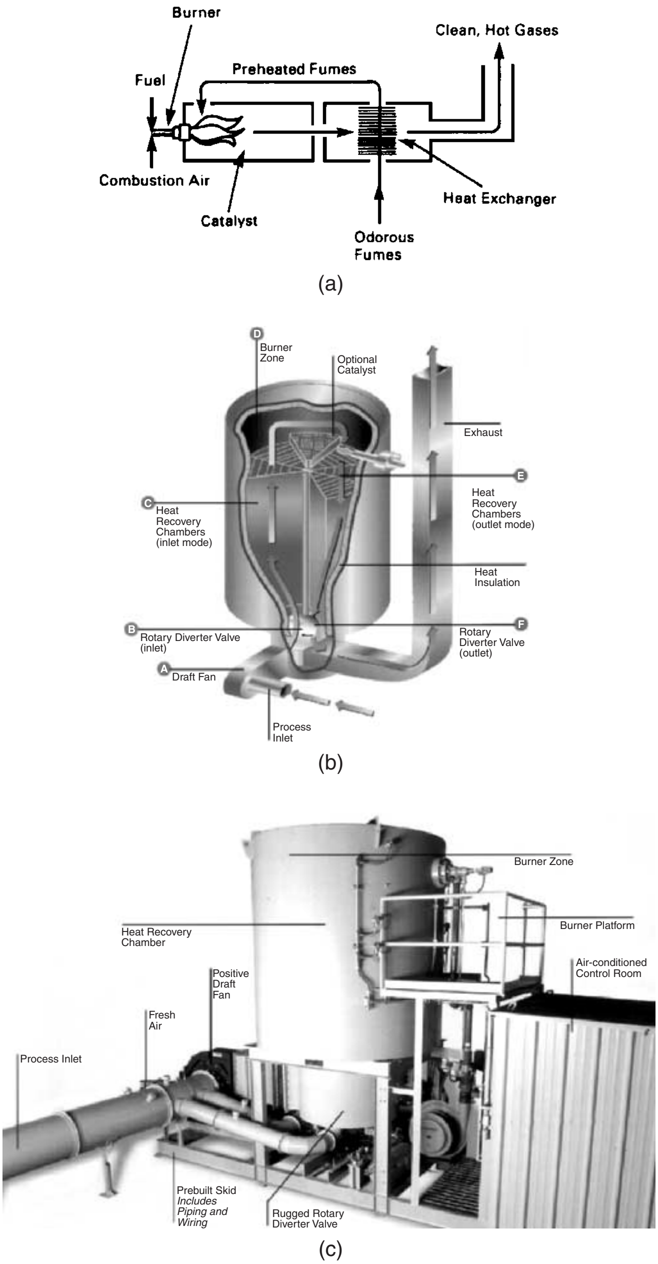


Figure 5.88 Schematic diagram of a catalytic odor oxidation system.

TABLE 5.21 Agents that Adversely Affect Catalysts of the Platinum Group Metals*

Type of agent	Examples
Poison	Heavy metals Phosphates Arsenic
Suppressant	Halogens Sulfur compounds
Fouling agents	Inorganic particles Alumina and silica dust Iron oxides

*Adapted from refs. [43,53].

5.10.5.5 Soil and compost filter beds. Soil and compost filter beds have been used successfully to remove odors generated at wastewater treatment plants. These systems can be used as a primary odor-control device or as a polishing step for exhausts from other odor-control systems. Soil/compost filters typically use sandy loam soil, compost, or mixtures of soil and peat moss as media. Typical media depths of 1 to 3 m (3–10 ft) are sufficient to remove odors. Performance of soil/compost filter beds depends on numerous factors, including (1) type and concentration of odorous compounds, (2) characteristics of the filter media (i.e., organic content, bulk density, porosity, etc.), (3) moisture content of the bed, (4) temperature, and (5) contact time. A schematic diagram of a soil/compost system is provided in Fig. 5.89.

During operation of soil/compost filter bed systems, contaminated gas is introduced into the bed through perforated pipes located near the bottom of the bed. As the gas passes upward through the bed, the media sorbs the odorous compounds that are subsequently oxidized by indigenous microorganisms. For proper operation, soil/compost filters require adequate amounts of nutrients and moisture to maintain biological activity. Although sufficient amounts of nutrients typically are available through the slow release of organic nitrogen contained in the soil and/or compost, nutrient supplements can be employed. The moisture content of the soil/compost filter bed is critical to its proper operation. If evaporative moisture losses are significant, watering of the bed may be required. Conversely, in moist climates, it may be necessary to provide an underdrain for removal of excess water or to cover the filter bed. A full-scale compost bed used for odor control is shown in Fig. 5.90.

Because of the many variables affecting design and performance, it is essential that laboratory and/or pilot tests be conducted prior to full-scale design and construction of soil/compost filter beds. The values of typical design parameters obtained in laboratory/pilot testing include (1) permissible air-loading rates, (2) rate of odor compound removal,

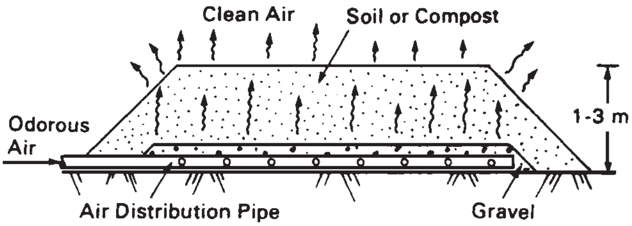


Figure 5.89 Schematic diagram of a soil/compost filter bed used for odor control.

Figure 5.90 Photograph of a soil/compost filter bed used for odor control.

and (3) nutrient/moisture requirements. A laboratory-scale soil/compost filter system used to evaluate full-scale design parameters is shown in Fig. 5.91.

5.10.5.6 Bioscrubber systems. A hybrid between a wet scrubber and a soil/compost filter bed is the bioscrubber odor-control system. The basic bioscrubber process recirculates biologically active, nutrient-rich scrubbing solutions over inert media while odorous air is forced upward. The bioscrubber media provide sites for microbial film attachment and mass transfer of odor compounds from the air into the scrubbing solution.

The primary metabolic pathway employed for odor compound oxidation varies in bioscrubber systems according to the type of compound being removed and the species of microorganisms present in the system. Bioscrubbers are characterized by the principal type of microbial populations that have colonized the system. For example, bioscrubbers

Figure 5.91 Photograph of a laboratory-scale soil/compost filter bed used to evaluate full-scale design parameters. (*Courtesy of Cache Environmental Laboratory, P.C.*)

may be designated as autotrophic (i.e., systems that remove hydrogen sulfide and/or ammonia) or heterotrophic (i.e., systems that remove organic compounds). Both types of bioscrubbers use an intermittent irrigation cycle in which a biologically active, nutrient-rich solution is trickled over the microbial film attached to the media.

To provide more effective odor removal and operational flexibility, both autotrophic and heterotrophic bioscrubber modes can be incorporated into one design. The most common application is to use a two-stage system connected in series. The two-stage system consists of an autotrophic first-stage bioscrubber used to remove hydrogen sulfide followed by a second-stage heterotrophic bioscrubber used to remove organic compounds. An advantage of employing a two-stage system is that removal of hydrogen sulfide in the first-stage bioscrubber minimizes acidification and potential pH depression in the second-stage bioscrubber, where heterotrophic microorganisms require neutral pH conditions for effective odorant compound removal (Fig. 5.92).

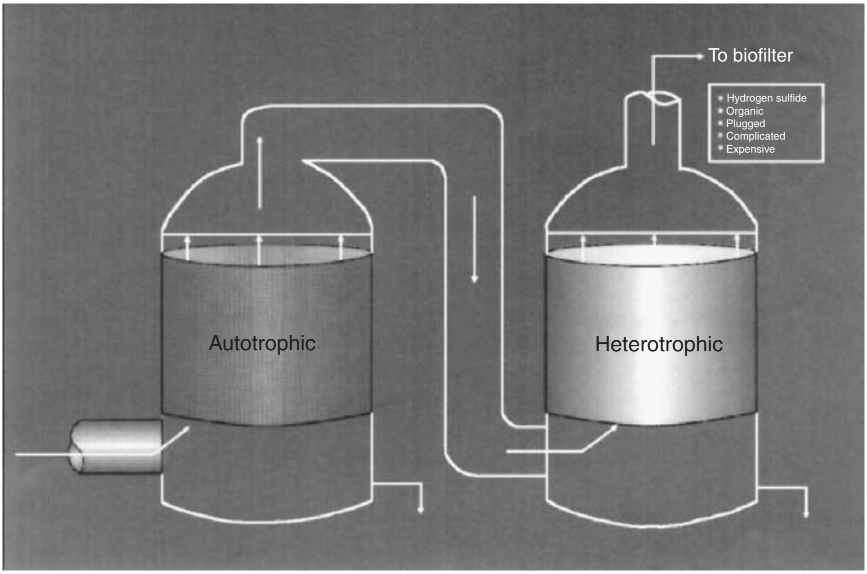


Figure 5.92 Schematic diagram of a two-stage bioscrubber system. (*Courtesy of BIOWAY.*)

Typical empty-bed contact times of between 10 to 15 seconds are suitable for most odor-removal applications. However, for bioscrubbers operating to remove only organic compounds, contact times of longer than 30 seconds have been employed [43].

Bioscrubber solutions must contain nutrients and other essential elements necessary for optimal microbial growth. At the wastewater treatment plant, plant effluent may be used as the bioscrubber solution. However, in many cases, plant effluent may need to receive nitrogen and/or phosphorus supplements depending on the type of odor compound to be removed (Fig. 5.93).

Flexibility and treatment efficiency can be increased for bioscrubber systems by manipulating the basic operational parameters for each stage. These operational parameters include irrigation cycle, medium, nutrients, and contact time. Additional design information on bioscrubber design may be found in the following references [43,53].

5.10.6 Use of existing biological stabilization processes for odor control

In many instances, odorous air can be collected and piped into the bottom sections of trickling filters and/or activated-sludge basins for effective odor control. Introducing air into the bottom of a trickling filter results in countercurrent flow similar to that found in a wet

Figure 5.93 Photograph of a two-stage bioscrubber system. (*Courtesy of BIOWAY.*)

scrubber system. For use of trickling filters in odor control, contact time of the gas with wastewater and filter media is critical. Minimum contact times of 8 to 10 seconds have been found effective in odor removal [28,37,43]. In some cases, where the influent to the trickling filter contains dissolved sulfides, introduction of odorous air into the bottom of the filter may strip H_2S from the liquid, resulting in the discharge of malodorous gas from the top of the filter. In this case, consideration should be given to covering the filters and directing effluent gases to an appropriate air-control device.

Activated-sludge basins have been used successfully for odor control by introducing malodorous air into the inlet side of the blowers. Biological deodorization of the gas occurs in the overlying aerated liquid of the aeration basin. This method of odor control is most applicable for controlling H_2S odors, since sulfides are rapidly oxidized in the aerobic mixed liquor.

Equipment requirements for utilizing existing biological stabilization processes for odor control are minimal and consist mainly of piping to convey the odorous gases to the unit process. Where trickling filters are used, an auxiliary blower may be required to force the gas through the media. For an activated-sludge system, existing blowers can be employed to inject the odorous gas into the basin. Because of the lack of design criteria and the wide variability in gas characteristics, pilot studies are recommended to evaluate the effectiveness of using existing biological stabilization processes for odor control.

5.10.7 Use of odor modification, counteraction, and maskants for odor control

The addition of various chemical agents to disguise odors, reduce their intensity, or render them less offensive has been met with variable success at wastewater treatment plants. *Odor modification* is the name given to the process in which two odorous substances are mixed, with the resulting odor of the mixture being less intense than that of the separate components. *Odor counteraction* is a term used to describe the process in which odor intensity is reduced by adding non-chemically reactive odor agents to a malodorous gas. Examples of pairs of counteraction compounds and odiferous gases that, in certain proportions, result in the mixture being odorless or nearly so include (1) ethyl mercaptan/eucalyptol and (2) skatoles/coumarin. Figure 5.94 depicts commercially available counteraction materials.

Odor masking is the name given to the process by which the quality of a malodor is overcome by mixing it with a substance having a strong, pleasant odor. The pleasant odor effectively masks the malodor, resulting in a less objectionable odor. Typical masking agents include (1) vanillin, (2) eugenols, and (3) phenylethyl alcohol [43].

In most cases, odor modification, counteractions, and masking are used as short-term solutions to severe odor problems. Prevention of odor emissions and positive control/removal of the odor-causing substances are preferred alternatives for effective odor control.

Figure 5.94 Photograph of commercially available counteraction materials. (*Courtesy of Odor Control Company*, *Scottsdale, Arizona*.)

5.11 Corrosion Control

Corrosion may be broadly defined as the destruction or deterioration of materials by direct chemical or electrochemical reaction with their environment. Oxidation is the most familiar type of corrosion at wastewater treatment plants and is readily observable in the form of rust. Oxidation involves an exchange of electrons between the metal and free oxygen present in the environment. Examples of the chemical reactions that occur during the oxidation of iron are given in Fig. 5.95.

In addition to oxygen, gases such as H_2S are also corrosive both in the gaseous state and after reaction with water and oxygen to form sulfuric acid. H_2S gas can directly attack metallic components of wastewater treatment systems such as steel tanks, conveyors, screens, etc. H_2S also reacts with copper electrical components to form copper sulfide.

In addition to chemical corrosion, galvanic corrosion can be a significant concern at wastewater treatment facilities. Galvanic corrosion occurs as a result of the electric current generated when two or more dissimilar metals are immersed in an electrolyte. Although wastewater is usually the electrolyte of interest, moist soils surrounding steel tanks or pipes also can serve as the electrolytic solution. The generation of the electric current between the two dissimilar metals is referred to as a *dissimilar electrode cell* and may occur under a wide range of conditions [19,43]. The tendency of metals to enter into this type of reaction is due to their electromotive force or electrical potential. The following metals are listed in the order of decreasing electric potential: (1) magnesium, (2) aluminum, (3) zinc, (4) chromium, (5) iron, (6) cadmium, (7) nickel, (8) tin, and (9) lead [37,43].

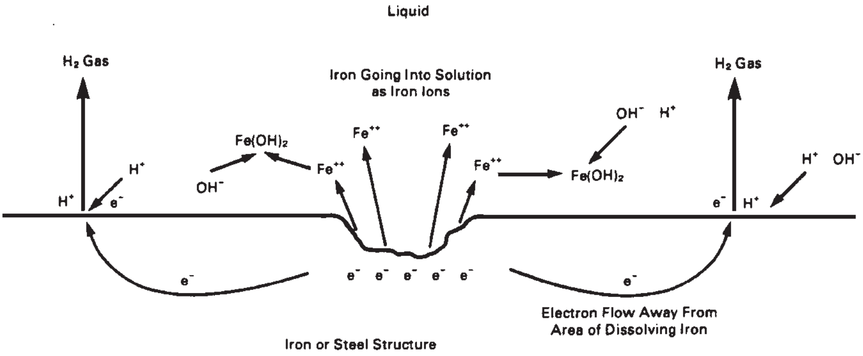


Figure 5.95 Schematic diagram of electron transfer reactions that occur during rust formation.

When two metals form a dissimilar electrode cell, the metal with the highest electrical potential serves as the anode (negative polarity), while the other metal acts as the cathode (positive polarity). During galvanic corrosion, the metal serving as the anode is oxidized. Factors that affect the rate of galvanic corrosion include the proximity of the two metals, the electrical potential of the two metals, conductivity of the electrolyte, temperature, and pH [19,37,43].

5.11.1 Corrosion protection

For corrosion protection from H_2S, oxidation using chemical addition has been found to be an effective approach. Chemicals used to remove H_2S for corrosion protection include (1) hydrogen peroxide, (2) nitrates, (3) metal salts, and (4) chlorine. In addition to chemical addition, protective coatings can be employed to reduce corrosion. Figure 5.96 illustrates the application of an inert coating for corrosion protection of storage tanks.

For protection from galvanic corrosion, cathodic protection should be employed in addition to the use of protective coatings. *Cathodic protection* is defined as the reduction or prevention of corrosion of a metal surface by rendering it cathodic through the use of a sacrificial anode and an imposed current. Cathodic protection has been used successfully for corrosion control of iron and steel structures at wastewater treatment plants such as clarifiers, aeration tanks, sludge storage tanks, and sludge digesters [28,37,43].

There are basically two approaches of cathodic protection currently in use at wastewater treatment plants: (1) sacrificial anode method and (2) impressed electromotive force (emf) method. Both methods

Figure 5.96 Photograph of coating applied to tanks for corrosion control. (*Courtesy of C.I.M. Industries, Inc.*)

result in imparting a negative charge to the metal structure to be protected with respect to its surroundings. By causing current to flow to the protected metal structure, the migration of metallic ions from the structure is inhibited.

In the typical application of sacrificial anode cathodic protection method, the metal structure to be protected is in electric contact with a metal of higher electrical potential (called the *sacrificial anode*). As the sacrificial anode corrodes, an electric current is generated that flows to the protected structure. For iron and steel protection, aluminum, zinc, and magnesium theoretically could be used as sacrificial anodes. However, since magnesium has the highest electrical potential, it is most often used as the sacrificial anode.

In the impressed emf cathodic protection method, the metal to be protected is electrically connected to the negative terminal of a current source, and the positive terminal is connected to a nonsacrificial anode (e.g., carbon) placed in the corrosive electrolytic solution (e.g., moist soil). Current from the anode passes through the electrolyte to the metal, making it cathodic and reversing the current at the anodes of localized galvanic cells. The source of current is typically a rectifier that supplies low-voltage direct current of several amperes [43]. The applied voltage required to provide adequate cathodic corrosion protection is determined by measuring the electrical potential of the structure to be protected. The effectiveness of the impressed voltage cathodic protection system depends on electrical potential. Since corrosion control is a specialized field, experts should be consulted whenever problems are encountered.

5.11.2 Ventilation and heating

Ventilation and heating of enclosed spaces for humidity control have been employed successfully to prevent condensation of corrosive vapors on components and structures in wastewater treatment facilities [2,43]. Adequate provision of heating and ventilation systems in enclosed spaces such as pumping rooms can reduce or eliminate moisture condensation and reduce the rate of corrosion while increasing the life of protective paints and coatings.

5.12 Problems

5.1 The Malindi City Wastewater Treatment Plant currently uses paved drying beds to dewater its aerobically digested biosolids. If the height of the liquid level in the digester is 35 ft above the outfall of the drying beds, estimate the velocity V and discharge rate Q of the biosolids if they are discharged by gravity through 500 ft of 8-in steel pipe. The pipeline contains four 45° elbows, and two gate valves that are fully opened during biosolids flow.

Assume that the Darcy-Weisbach friction factor is 0.015 and that the minor loss coefficients for the elbows and gate valve are 0.6 and 1.1, respectively.

5.2 The Karotho County Sewer Improvement District must take one of its gravity thickeners out of service for repair. If the contents of the thickener are to be pumped to an aerated storage tank through 900 ft of 10-in pipe, determine the head loss generated in the pipeline if the thickener volume were 100,000 gal and the entire thickener contents must be transferred within 1 hour. Assume that the average solids content of the pumped liquid is 3.2 percent and that a Hazen-Williams coefficient of 120 is suitable to describe the quality of the pipe.

5.3 The Douala County Water Reclamation Plant is currently pumping its thickened biosolids through 3000 ft of 8-in steel pipe to a drying lagoon. If the thickened biosolids have a solids content of 6 percent, estimate the head loss in the pipe if the pumping rate is maintained at 300 gal/min. Assume that, through laboratory tests, it was determined that the biosolids have a yield stress τ_o of 0.035 lb_f/ft^2, a coefficient of rigidity of 0.024 lb/ft·s, and a specific weight of 63.5 lb/ft^3.

5.4 The Gemena City Wastewater Treatment Plant is employing a positive-displacement pump to transfer 400 gal/min of anaerobically digested biosolids through 2100 ft of 10-in steel pipe to a facultative storage lagoon. If the electric motor used to operate the pump imparts 35 hp of mechanical energy, estimate the efficiency of the pump. Assume that the frictional head losses have been estimated to be 80 ft of H_2O at the given biosolids flow rate and that the maximum elevation difference occurring within the pipeline is 200 ft. The specific weight of the anaerobic biosolids was measured to be 63.0 lb/ft^3.

5.5 The Mandera County Water Reclamation Plant is currently employing a disk-nozzle centrifuge to dewater its thickened biosolids. If the average thickened biosolids production rate is 27,000 gal/day and the biosolids are processed through the centrifuge 3 days a week (Wednesday, Thursday, and Friday), estimate the minimum volume (in gallons) of the biosolids storage tank and the required biosolids processing rate (gallons per day). Assume that the following peaking factors have been determined from analysis of the daily biosolids volumetric flow rates (gallons per day):

Length of sustained peak (days)	Peaking factor
1	2.2
2	1.8
3	1.6
7	1.3
10	1.1

5.6 The Lodwar County Sewer Improvement District has completed construction of a series of sand drying beds for dewatering its stabilized biosolids. If the biosolids are to be dewatered from an initial solids content of 4.0 percent to a final solids content of 25 percent, estimate (A) the dewatering time for a single

12-in sludge layer, (B) the number of applications that can be made per year, and (C) the annual solids loading rate. Assume that 20 percent of the initial moisture discharged to the beds through biosolids application is removed through drainage and that the average pan evaporation rate is 4.5 in/month. The estimated reduction factor k_e for biosolids evaporation has been measured to be 0.6.

5.7 The Garsen City Wastewater Treatment Plant currently generates 14,000 gal/day of thickened biosolids. If the facility desires to use eight conventional sand drying beds year round to dewater the biosolids from an initial solids content of 3.0 percent to a final solids content of 30 percent, estimate the minimal area necessary for each drying bed (beds are to be of equal size). Assume that the average annual pan evaporation rate for the area is 3.9 ft and that 15 percent of the biosolids moisture is removed initially through gravity drainage. The estimated reduction factor k_e for biosolids evaporation has been measured to be 0.7.

5.8 The Garsen City Wastewater Treatment Plant (see Prob. 5.7) desires to evaluate the use of reed-enhanced sand drying beds to reduce the required bed size to dewater its biosolids from an initial solids content of 3.0 percent to a final solids content of 30 percent. If the facility generates 14,000 gal/day of thickened biosolids, estimate the minimum area required for reed-enhanced sand drying bed dewatering system if the volumetric loading rate is to be limited to 18 gal/ft^2·year.

5.9 The Kipini County Wastewater Treatment Plant has recorded the following average daily air temperatures during a 7-day winter period. Given the recorded data, estimate the freezing index for the 7-day period.

Day	Mean temperature (°C)
1	−4°C
2	−2°C
3	−7°C
4	−3°C
5	−2°C
6	−4°C
7	−9°C

5.10 The Niamey City Water Reclamation Plant has recorded the following average monthly temperatures over a 5-year period. Given the temperature data, what year should be used as the design year for a freeze-assisted sludge drying bed design? Assume that the freezing temperatures only occur over a 4-month period in the winter (December through March).

	Dec. (31 days)	Jan. (31 days)	Feb. (28 days)	March (31 days)
Year 1	−5°C	−7°C	−4°C	−1°C
Year 2	−3°C	−5°C	−4°C	−2°C
Year 3	−6°C	−9°C	−7°C	−4°C
Year 4	−2°C	−8°C	−5°C	−4°C
Year 5	−6°C	−8°C	−4°C	−3°C

5.11 Marsabit City Wastewater Treatment Plant is currently generating 8000 gal/day of thickened biosolids. If the facility is planning to use freeze-assisted sand drying to dewater the thickened biosolids from an initial solids content of 5.0 percent to a final solids content of 30 percent, estimate the potential depth of frozen biosolids and the minimum bed area needed if the average frost penetration for the area has been estimated to be 140 cm.

5.12 Lucira City Water Reclamation Plant currently employs freeze-assisted sand drying beds to dewater its thickened biosolids. If a preliminary evaluation indicated that an 80-cm depth of biosolids could be frozen during the year, estimate the number of days required to completely thaw the frozen layer of biosolids. Assume that the average monthly temperatures during the winter months are below freezing (i.e., December through February) and that the average monthly temperatures during the months of March (31 days), April (30 days), May (31 days), and June (30 days) are 8, 16, 24, and 32°C, respectively.

5.13 The Kondoa County Sewer Improvement District has decided to employ paved drying beds to dewater its thickened biosolids from an initial solids content of 4.0 percent to a final solids content of 28 percent. If the solids content of the biosolids after decanting has been estimated to be 14 percent, estimate the required bed area if the daily generation rate of thickened biosolids is 7500 gal/day. Assume that the annual precipitation and evaporation rates are 2.8 and 3.9 ft, respectively, and that the reduction factor for biosolids evaporation k_e has been measured at 0.8.

5.14 The Zambesi County Sewer Improvement District is currently employing thermal treatment to condition 17,000 gal/day of thickened biosolids having an initial solids content of 5.3 percent. If the biosolids flow consists of 70 percent primary biosolids and 30 percent secondary biosolids (waste-activated sludge), estimate the pounds of volatile suspended solids (VSS) solubilized during the conditioning process and the increase in BOD_5 as a result of VSS solubilization.

5.15 The Gambela City Water Reclamation Plant is currently displacing $1.2 \cdot 10^7$ ft^3/day of air from its biosolids conditioning facilities to a series of activated-carbon beds for treatment prior to discharge to the atmosphere. If the air contains an average of 15 ppm H_2S (volume basis), estimate the minimum number of pounds of carbon needed if it is desired that the carbon beds last at least 90 days before they are exhausted. Assume that the following conditions apply:

Carbon adsorption efficiency: 0.35
Molecular weight of H_2S: 34 lb/lb·mol
Molar volume of air at operating conditions: 386 ft^3/lb·mol

5.13 References

1. Aerstin, F., and G. Street (1978), *Applied Chemical Process Design,* Plenum Press, New York.

2. American Society of Civil Engineers (1990), *Waste Containment Systems,* Geotechnical Special Publication No. 26, edited by Rudolph Bonaparte, New York.
3. American Conference of Governmental Industrial Hygienists (1988), *Industrial Ventilation,* 22d ed.
4. Banks, L., and S. F. Davis (1983), "Desiccation and Treatment of Sewage Sludge and Chemical Slimes with the Aid of Higher Plants," in *Proceedings of the National Conference on Municipal and Industrial Sludge Utilization and Disposal,* Atlantic City, N.J.
5. Bastian, R. K. (1997), "Biosolids Management in the United States," *Water Environment and Technology,* pp. 45–50.
6. Daniel, D. E. (1993), *Geotechnical Practice for Waste Disposal,* Chapman & Hall, London.
7. Davis, M. L., and D. A. Cornwell (1991), *Introduction to Environmental Engineering,* McGraw-Hill, New York.
8. DeNevers, N. (1999), *Air Pollution Control Engineering,* McGraw-Hill, New York.
9. Dick, R. I. (1972), "Sludge Treatment," in *Physicochemical Processes for Water Quality Control,* Wiley, New York.
10. Eckenfelder, W. W. (1989), *Industrial Water Pollution Control,* 2d ed., McGraw-Hill, New York.
11. Goldstein, N., W. A. Yanko, J. M. Walker, and W. Jakubowski (1988), "Determining Pathogen Levels in Sludge Products," *Biocycle* 29:44–47.
12. Goldman, L. J., and L. I. Greenfield (1990), *Clay Liners for Waste Management Facilities,* Noyes Data Corporation.
13. Luthin, J. N. (1973), *Drainage Engineering,* Krieger Publishing Company.
14. McCabe, W. L., J. C. Smith, and P. Harriot (1993), *Unit Operations of Chemical Engineering,* McGraw-Hill, New York.
15. Mulbarger, M. C., S. R. Copas, J. R. Kordic, and F. M. Cash (1981), "Pipeline Friction Losses for Wastewater Sludges," *Journal of Water Pollution Control Federation* 51(8).
16. Oweis, I. S., and R. P. Khera (1998), *Geotechnology of Waste Management,* PWS Publishing Company.
17. Parkin, W. P. (1993), *The Complete Guide to Environmental Law,* Specialty Technical Publishers.
18. Peavy, H. S., D. R. Rowe, and G. Tchobanoglous (1985), *Environmental Engineering,* McGraw-Hill, New York.
19. Perry, R. H., and C. H. Chilton (1973), *Chemical Engineers Handbook,* McGraw-Hill, New York.
20. Reed, S. C. (1987), "Sludge Freezing for Dewatering," *Biocycle* 28(1).
21. Reed, S., R. Crites, and E. Middlebrooks (1994), *Natural Systems for Waste Management and Treatment,* McGraw-Hill, New York.
22. Rodriguez, F. (1982), *Principles of Polymer Systems,* McGraw-Hill, New York.
23. Ryan, J., and R. Chaney (1993), "Regulation of Municipal Sewage Sludge Under the Clean Water Act Section 503: A Model for Exposure and Risk Assessment for MSW-Compost, in *Science and Engineering of Composting,* Renaissance Publications, Worthington, Ohio.
24. Sanks, R. L., G. Tchobanoglous, D. Newton, B. E. Bosserman, and G. M. Jones (eds.) (1989), *Pumping Station Design,* Butterworths, London.
25. Sassaman, M. D., and T. R. Kaufman (1993), "Sludge Dewatering and Disposal Using the Reed System" *Operations Forum Magazine,* 10(6).
26. Sawyer, C. N., P. L. McCarty, and G. F. Parkin (1996), *Chemistry for Environmental Engineers,* McGraw-Hill, New York.
27. Tchobanoglous, G. (1981), *Wastewater Engineering: Collection and Pumping of Wastewater,* McGraw-Hill, New York.
28. Tchobanoglous, G. (1991), *Wastewater Engineering: Treatment Disposal and Reuse,* 3d ed., McGraw-Hill, New York.
29. Tchobanoglous, G., H. Theisen, and S. Vigil (1993), *Integrated Solid Waste Management: Engineering Principles and Management Issues,* McGraw-Hill, New York.

30. U.S. Army Corps of Engineers (1987), *Wetlands Delineation Manual,* Technical Report Y-87-1, Waterways Experiment Station, Vicksburg, Miss.
31. U.S. Department of Agriculture (1989), "Peer Review, Standards for the Disposal of Sewage Sludge. USEPA Proposed Rule 40 CFR Parts-257 and 503 USDA-CSRS Technical Committee," Univ. of California, Riverside.
32. U.S. Department of the Interior (1993), *Drainage Manual: A Water Resources Technical Publication,* 3d ed., Washington.
33. U.S. Environmental Protection Agency (1983), "Process Design Manual: Land Application of Municipal Sludge," EPA/625/1-83/016, Washington.
34. U.S. Environmental Protection Agency (1976), "Anaerobic Sludge Digestion: Operations Manual," EPA/430/9-76/001, Washington.
35. U.S. Environmental Protection Agency (1978), "Process Design Manual for Municipal Sludge Landfills," EPA 625/1-78-010, Washington.
36. U.S. Environmental Protection Agency (1979), "NPDES Compliance Sampling Manual," PB81-153215, Washington.
37. U.S. Environmental Protection Agency (1979), "Process Design Manual: Sludge Disposal and Treatment," EPA/625/1-79/011, Washington.
38. U.S. Environmental Protection Agency (1979), "Inspectors Guide for Evaluation of Municipal Wastewater Treatment Plants," EPA/430/9-79/010, Washington.
39. U.S. Environmental Protection Agency (1979), "NPDES Compliance Sampling Manual," PB81-153215, Washington.
40. U.S. Environmental Protection Agency (1982), "Design Manual for Dewatering Municipal Wastewater Sludges," EPA/625/1-82/014, Washington.
41. U.S. Environmental Protection Agency (1983), "Process Design Manual for Land Application of Municipal Sludge," EPA 625/1-83-016, Washington.
42. U.S. Environmental Protection Agency (1984), "Handbook: Septage Treatment and Disposal," EPA/625/6-84/009, Washington.
43. U.S. Environmental Protection Agency (1985), "Design Manual: Odor and Corrosion Control in Sanitary Sewerage Systems and Treatment Plants," EPA/625/1-85/018, Washington.
44. U.S. Environmental Protection Agency (1985), "Handbook Estimating Sludge Management Costs," EPA-625/6-85010, Washington.
45. U.S. Environmental Protection Agency (1987), "Design Manual for Dewatering Municipal Wastewater Sludges," EPA/625/1-87/014, Washington.
46. U.S. Environmental Protection Agency (1989), "POTW Sludge Sampling and Analysis Guidance Document," EPA/833/B-89/100, Washington.
47. U.S. Environmental Protection Agency (1990), "40 CFR Part 503, National Sewage Sludge Survey: Availability of Information and Data and Anticipated Impacts on Proposed Regulations," *Federal Register* 55(218):47210–47283.
48. U.S. Environmental Protection Agency (1990), "Guidance for Writing Case-by-Case Permit Requirements for Municipal Sewage Sludge," EPA/505/8-90/001, Washington.
49. U.S. Environmental Protection Agency (1990), "Motor Freight Terminals/Railroad Transportation," EPA/530/SW-90/027K, Washington.
50. U.S. Environmental Protection Agency (1991), "Cooperative Testing of Municipal Sewage Sludges by the Toxicity Characteristic Leaching Procedure and Compositional Analysis." EPA 430/09-91-007, Washington.
51. U.S. Environmental Protection Agency (1991), "Residential and Commercial Toxic Pollutant Loadings and POTW Removal Efficiency," PB93-209872, Washington.
52. U.S. Environmental Protection Agency (1991), "Sewer System Infrastructure Analysis and Rehabilitation," EPA/625/6-91/030, Washington.
53. U.S. Environmental Protection Agency (1991), "Control Technologies for Hazardous Air Pollutants," EPA/625/6-91/014, Washington.
54. U.S. Environmental Protection Agency (1992), "Determining Wastewater User Service Charge Rates—A Step-by-Step Manual," EPA/832/B-92/003, Washington.
55. U.S. Environmental Protection Agency (1992), "Organic Air Emissions from Waste Management Facilities," EPA/625/R-92/003, Washington.

56. U.S. Environmental Protection Agency (1992), "Environmental Regulations and Technology—Control of Pathogens and Vector Attraction in Sewage Sludge," EPA/625/R-92/013, Washington.
57. U.S. Environmental Protection Agency (1993), "Preparing Sewage Sludge for Land Application or Surface Disposal: A Guide for Preparers of Sewage Sludge on the Monitoring, Recordkeeping, and Reporting Requirements of the Federal Standards for the Use or Disposal of Sewage Sludge, 40 CFR Part 503," EPA/831/B-93/002a, Washington.
58. U.S. Environmental Protection Agency (1993), "Regulatory Impact Analysis of the Part 503 Sewage Sludge Regulation," EPA/821/R-93/006, Washington.
59. U.S. Environmental Protection Agency (1993), "Standards for Use or Disposal of Sewage Sludge. Final Rule, 40 CFR Part 503," *Federal Register* 58(32):9248–9415.
60. U.S. Environmental Protection Agency (1993a), "Technical Support Document for Land Application of Sewage Sludge," Vol. I," PB93-110575; Vol. II, PB93110583, Washington.
61. U.S. Environmental Protection Agency (1994), "A Plain English Guide to the EPA Part 503 Biosolids Rule," EPA/832/R-93/003, Washington.
62. U.S. Environmental Protection Agency (1995), "A Guide to the Biosolids Risk Assessments for the EPA Part 503 Rule," EPA/832/B-93/005, Washington.
63. U.S. Environmental Protection Agency (1995), "Ground-Water and Leachate Treatment Systems," EPA/625/R-94/005, Washington.
64. U.S. Environmental Protection Agency (1995), "Part 503 Implementation Guidance," EPA/833/R-95/001, Washington.
65. U.S. Environmental Protection Agency (1995), "Process Design Manual: Land Application of Sewage Sludge and Domestic Septage," EPA/625/R-95/001, Washington.
66. U.S. Environmental Protection Agency (1995), "Process Design Manual: Surface Disposal of Sewage Sludge and Domestic Septage," EPA/625/R-95/002, Washington.
67. U.S. Environmental Protection Agency (1999), "Environmental Regulations and Technology: Control of Pathogens and Vector Attraction in Sewage Sludge," EPA/625/R-92/013, Washington.
68. Vesilind, P. A., G. C. Hattman, and E. T. Shene (1986), *Sludge Management and Disposal for the Practicing Engineer,* Lewis Publishers.
69. Viessman, W., and M. J. Hammer (1985), *Water Supply and Pollution Control,* 4th ed., Harper Collins, New York.
70. Volpe, R. L., and W. E. Kelly (1985), *Seepage and Leakage from Dams and Impoundments,* American Society of Civil Engineers, New York.
71. Water Pollution Control Federation (1983), *Sludge Dewatering, Manual of Practice,"* No. 20.
72. Water Pollution Control Federation (1985), *Sludge Stabilization, Manual of Practice,* FD-9.
73. Weber, W. (1972), *Physicochemical Processes for Water Quality Control,* Wiley, New York.

Chapter 6

Fundamentals of Soil and Water Interactions

6.0 Introduction

The U.S. biosolids regulations (40 CFR Part 503) specify the legal requirements to which all biosolids beneficial-use programs must comply. The basis for these legal requirements was extensive risk-assessment studies conducted by the U.S. Environmental Protection Agency (USEPA) in which the fate of nutrients (e.g., nitrogen, phosphorus, etc.), heavy metals, pathogens, and toxic organics contained in land-applied biosolids was determined [65,67]. This chapter is an overview of the chemical and physical characteristics of soil that affect both the fate and transport of these biosolids constituents as well as the health and vitality of crops. It is not meant to be an exhaustive discussion but rather an introduction to the fundamental soil chemical and physical relationships that engineering students or practicing engineers should thoroughly understand before attempting to design and/or evaluate biosolids land-application systems. For more information on soil chemical and physical characteristics, the reader is directed to the following references [6,17,33,43,44].

6.1 General Definition of Soil

The word *soil* means different things to different people. The geotechnical/civil engineer normally considers soil to be earth material that supports foundations and roads. From this perspective, soil is all the material that covers the bedrock of the earth's crust. On the other hand, soil scientists regard soil as that part of the earth's crust which undergoes modification as a result of soil-forming processes (e.g., weathering). Normally, this definition of soil applies only to the first 5 to 10 ft of material. However, for the individual responsible for land application of biosolids, both the engineer's and soil scientist's definition of soil must be taken into account. The reason that both perspectives must be considered is that knowledge of the surface layers that support crop production, as well as the deeper soil layers that may be used for the transmission of moisture (i.e., drainage), are of interest.

6.2 Properties of Soils

Soils consist of three major components: solid (composed of both minerals and organic matter), water (i.e., soil solution), and air (i.e., soil gas). Of the total soil volume, about half is pore space, 45 percent is mineral matter, and 5 percent is organic matter [6]. The proportions of soil that consist of water and air are subject to rapid fluctuations depending on weather as well as other factors (Fig. 6.1). At the optimal soil moisture content for plant growth, the pore space is divided about evenly between soil solution and soil gas. The proportions and properties of minerals and organic matter in soils are a function of time, climate, topography, vegetation, and parent material [14].

Soil may be created by the weathering of solid materials derived from biological, geologic, and hydrologic processes that proceed over extended periods of time [43]. During the weathering process, large rocks erode into smaller ones and eventually into individual minerals. These minerals, which are subjected to biological and chemical processes, are transformed into new substances. Thus weathering is a combination of natural destructive and synthetic processes. Weathered soils tend to have gradual transitions (or horizons) between different soil layers, as illustrated in Fig. 6.2*a*.

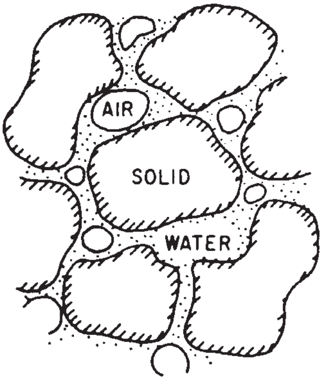


Figure 6.1 Schematic diagram of the three phases of soil. (*Adapted by permission from ref.* [*32*]).

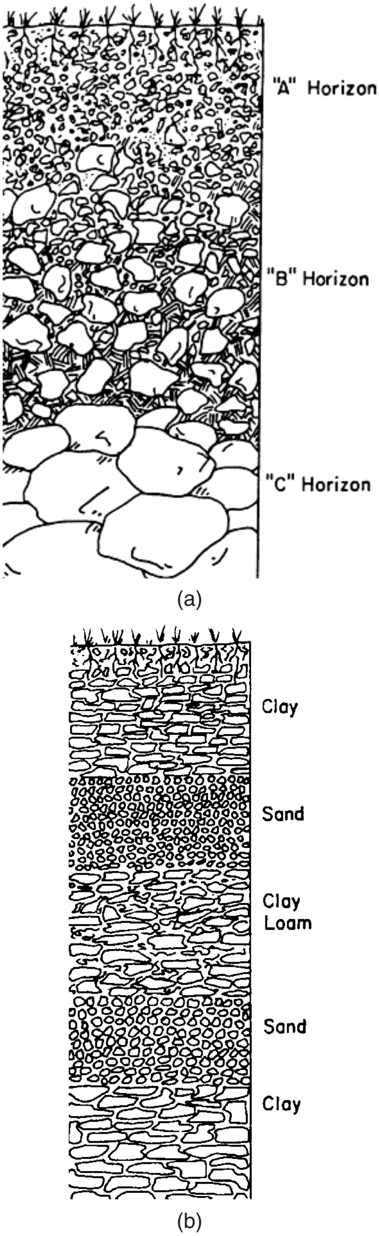


Figure 6.2 (*a*) Transitions characteristic of weathered soils. (*b*) Alluvial soil with stratification. (*Adapted by permission from ref.* [*32*]).

In addition to weathering, soils may be formed by deposition of sediments that are transported by either wind or water. Water-borne sediments form what are termed *alluvial deposits* [43]. The three general classes of alluvial deposits include (1) flood plains, (2) alluvial fans, and (3) deltas. Alluvial deposits tend to be stratified and have abrupt transitions between individual layers (Fig. 6.2*b*).

Whether formed by natural weathering processes or stratified deposition of sediments, soils are not as uniform over a field as the biosolids land-application engineer might anticipate. Thicknesses of various layers, distribution of chemical constituents, and physical properties of the soil can vary substantially over a field. Distribution of thin lenses of chemically and/or physically distinct materials over an agricultural field can result in significant changes in soil properties [6].

6.3 Soil Chemistry

Understanding the principles of soil chemistry is critical for the biosolids land-application engineer because of their impact on the fate of toxic elements, moisture, and nutrients. Moreover, with respect to agricultural crop production, land application of biosolids can influence the accumulation of salts and phytotoxic elements within the plant-soil system. Although the biosolids land-application rates identified in the 40 CFR Part 503 regulations minimize the probability of toxic element accumulation in crops, there are no special precautions cited in the regulations with regard to the adverse problems of salt accumulation. It is generally assumed that the standard agricultural guidelines for salt management on irrigated farmland will suffice for protecting crops grown on biosolids land-application sites [43].

6.3.1 Inorganic soil species

The inorganic fraction of soil consists primarily of clay minerals including silicates, oxides, and carbonates. Table 6.1 lists some of the most common soil minerals found in agricultural soils. Clay minerals are composed of layered sheets of tetrahedrally and/or octahedrally coordinated cations. Clays have been classified into two major groups: montmorillonite and kaolinite. The structure of montmorillonite clay is characterized by sheets of silica tetrahedra and aluminum octahedra organized in a 2:1 configuration. While kaolinite structure is comprised of the same minerals as montmorillonite, its silica/aluminum sheets are organized in a 1:1 configuration [43,44].

Because of its physical structure, montmorillonite clay readily absorbs moisture and is sometimes referred to as *swelling clay*. When subjected to dry conditions, montmorillonite will readily loose moisture and form shrinkage or desiccation cracks. Conversely, the kaolinite structure is

TABLE 6.1 Typical Composition of Soil Minerals*

	SiO_2 (%)	Fe_2O_3 (%)	Al_2O_3 (%)	TiO_2 (%)	CaO (%)	MgO (%)
Sand	86.3	5.19	6.77	1.05	0.37	1.02
Coarse silt	81.3	3.11	7.21	1.05	0.41	0.82
Fine silt	64.0	9.42	12.00	1.05	0.32	2.22
Coarse clay	45.1	13.50	21.10	0.96	0.38	2.09
Fine clay	30.2	17.10	22.80	0.88	0.08	1.77

*Adapted from ref. [6].

compact, and there is no room for ions or small molecules to be inserted between the silica/aluminum sheets. Kaolinite, therefore, does not swell or shrink appreciably with the addition or loss of moisture.

6.3.1.1 Cation exchange capacity. During the chemical weathering of minerals, trivalent aluminum or iron is often substituted for tetravalent silicon and/or divalent magnesium in the clay lattice [6]. This chemical substitution results in imparting a permanent net negative charge on the surface of the clay mineral. This negative charge can only be satisfied (i.e., neutralized) by surface retention (i.e., adsorption) of a positively charged ionic species (i.e., cation). The magnitude of the negative charge on clay minerals is referred to as its *cation exchange capacity* (CEC). In the soil science literature, the cation exchange capacity is measured as the total cationic charge that is held by 100 g of soil. CEC is normally expressed in units of milliequivalents per 100 g of soil. Some representative CEC values for clay minerals are provided in Table 6.2.

The amount and type of cations that adsorb to clays have a significant impact on the flocculation and dispersion properties of soils. These properties, in turn, affect the soil water transmission rates. Soil flocculation is characterized by the coalescing (e.g., aggregation) of individual clay particles to form larger particles or flocs. Water transmission is facilitated within highly flocculated soils because of the existence of a significant number of large-size pores. The reverse of flocculation is dispersion. Soil dispersion is characterized by the disintegration of clay aggregates into individual particles. Dispersed soils are characterized by poor drainage and low moisture infiltration [29].

There are two factors that influence flocculation and dispersion of clay particles. These include the valence or magnitude of the electronic charge on the adsorbed cation and the salt concentration of the soil solution. In general, the larger the valence of the adsorbed cation, the closer the cation is held to the clay. For example, calcium with a valence of 2 is held more closely to clay particles than is sodium with a valence of 1. The practical implications of this is that soils that have a relative predominance of calcium adsorbed to the clay particles will be highly floc-

TABLE 6.2 Cation Exchange Capacity of Clay Minerals (meq/100 g)*

Clay	CEC
Kaolinite	3–15
Halloysite·H_2O	5–10
Montmorillonite	80–150
Illite	10–40
Vermiculite	100–150
Chlorite	10–40

*Adapted from ref. [6].

culated and have a large water transmission potential (i.e., an increased number and size of soil pores) compared with those clays that are characterized by an extensive level sodium adsorption (i.e., sodic soils).

For successful agricultural operations, sodic soils must be engineered to improve their soil structure and water transmission capabilities. This is normally accomplished by leaching the soils with water to extract the sodium (a process called *elutriation*). Following the leaching procedure, the soils are treated with aqueous solutions of a divalent metal (e.g., calcium, magnesium, or aluminum) salt [14]. Addition of the divalent cations promotes soil aggregation and increases the soil water transmission characteristics.

6.3.1.2 Salinity. The salt content of soil is termed the *soil salinity.* Saline soils are of particular concern in arid regions, where evaporation, together with consumptive use of irrigation water, results in soils with excessively high salt concentrations [14]. High salt concentrations can reduce both the water transmission potential of soils and vegetative growth. Table 6.3 gives salt tolerances for various forage crops typically grown on biosolids land-application sites.

Salt concentrations in soil vary widely both vertically and horizontally depending on the extent of changes in soil texture, plant growth, and water transmission potential. The extent of salination is governed by the rate of evapotranspiration of soil water and the leaching of moisture from the root zone. Maintenance of a favorable salt balance is a necessary requirement for sustained agricultural production at biosolids land-application sites. However, some soils have such high concentrations of salts prior to biosolids application that an initial leaching is required before agricultural production can begin. Quantifying the impact of salinity on crop yield is necessary to compute the potential economic benefits from salt leaching practices or installation of land drainage systems.

6.3.1.3 Sodicity. A term related to salinity is *sodicity.* Sodicity refers to the amount of sodium present in soil. The soil structure and mois-

TABLE 6.3 Salt Tolerance Levels for Specific Crops*

	Yield potential†				
Crop	100% EC_e	90% EC_e	75% EC_e	50% EC_e	Maximum EC_e
Alfalfa	2.0	3.4	5.4	8.8	16.0
Barley hay	6.0	7.4	9.5	13.0	20.0
Bermuda grass	6.9	8.5	10.8	14.7	23.0
Clover	1.5	3.2	5.9	10.3	19.0
Corn (forage)	1.8	3.2	5.2	8.6	16.0
Harding grass	4.6	5.9	7.9	11.1	18.0
Orchard grass	1.5	3.1	5.5	9.6	18.0
Perennial rye	5.6	6.9	8.9	12.2	19.0
Sordan grass	2.8	5.1	8.6	14.4	26.0
Tall fescue	3.9	5.8	8.6	13.3	23.0
Wheat grass (tall)	7.5	9.9	13.3	19.4	32.0
Wheat grass	7.5	9.0	11.0	15.0	22.0
Trefoil, big	2.3	2.8	3.6	4.9	8.0
Trefoil, small	5.0	6.0	7.5	10.0	15.0

*Adapted from ref. [14].

†EC_e is the electrical conductivity of soil extract (mmhos/cm). [1 mmho/cm = 1 dS/m (dS = decisiemen).]

ture transmission potential of soil are significantly affected by the percentage of sodium that comprises a soil's cation exchange capacity [6]. The proportion of sodium on a particular clay particle relative to the cation exchange capacity is called the *exchangeable sodium percentage* (ESP). For example, if a soil has a CEC of 100 meq per 100 g and it also contains 15 meq of sodium, the ESP for this soil is 15 percent.

The deleterious effect of exchangeable sodium on water transmission capacity is moderated by increased levels of exchangeable calcium and magnesium. To predict the potential hazards from high sodium levels in soil, a cation ratio was developed that takes into account calcium and magnesium as well as sodium. This ratio is called the *sodium adsorption ratio* (SAR) and is defined by Eq. (6.1):

$$\mathrm{SAR} = \frac{[\mathrm{Na}^+]}{\sqrt{[\mathrm{Ca}^{2+}] + [\mathrm{Mg}^{2+}]}} \tag{6.1}$$

where

SAR = sodium adsorption ratio

$[\mathrm{Na}^+], [\mathrm{Ca}^{2+}], [\mathrm{Mg}^{2+}]$ = molar concentrations of sodium, calcium, and magnesium, respectively, from an aqueous extract of soil

Salt-affected soils are classified as normal. saline, sodic, or saline-sodic according to the electrical conductivity of a saturated extract (EC_e) measured at 25°C (77°F) at a given sodium adsorption ratio (Table 6.4). It should be noted that electrical conductivity is normally reported in units of millimhos per centimeter (i.e., mmhos/cm) or

TABLE 6.4 Classification of Soils Based on Salinity/Sodicity*

Criteria†	Normal	Saline	Sodic	Saline-sodic
EC_e	<4	>4	<4	>4
SAR	<13	<13	>13	>13

*Adapted from ref. [14].
†EC_e is electrical conductivity (mmhos/cm); SAR is in $(\text{mmol/liter})^{0.5}$.

decisiemens per meter (dS/m). The use of Table 6.4 in classifying soils is illustrated in Example 6.1.

Example 6.1 The Wallace County Water Reclamation Plant is considering land applying its biosolids on adjacent pasture land. Given the following chemical characteristics of an aqueous extract of the soil from the site, classify the soil based on its electrical conductivity and sodium adsorption ratio.

Electrical conductivity (EC_e): 15.0 mmhos/cm

$[\text{Ca}^{2+}]$: 35.0 mg/liter

$[\text{Mg}^{2+}]$: 51.0 mg/liter

$[\text{Na}+]$: 350.0 mg/liter

solution

Step 1. Convert the metal concentrations from mg/liter to meq/liter using their respective equivalent weights (e.g., milligrams per milliequivalent). For example, calcium has an atomic weight of 40.08 and a valence of 2. Therefore, its equivalent weight is 40.08 divided by 2, or 20.04 mg/meq.

$$[\text{Ca}^{2+}] = \frac{35 \text{ mg/liter}}{20.04 \text{ mg/meq}} = 1.75 \text{ meq/liter}$$

$$[\text{Mg}^{2+}] = \frac{51 \text{ mg/liter}}{12.15 \text{ mg/meq}} = 4.2 \text{ meq/liter}$$

$$[\text{Na}^{+}] = \frac{350 \text{ mg/liter}}{23.0 \text{ mg/meq}} = 15.2 \text{ meq/liter}$$

Step 2. Compute the sodium adsorption ratio using Eq. (6.1):

$$\text{SAR} = \frac{[\text{Na}^+]}{\sqrt{[\text{Ca}^{2+}] + [\text{Mg}^{2+}]}}$$

$$= \frac{15.2}{(1.75 + 4.2)^{1/2}} = 6.23 \ (\text{mmol/liter})^{1/2}$$

Given its electrical conductivity and SAR, the soil is classified as saline (see Table 6.4).

6.3.1.4 Soil pH. Another important property of the soil is its acidity, which is normally expressed as the negative logarithm of the hydrogen ion activity, or pH. Soil pH, which is given in Eq. (6.2), is used to assess whether a soil is acidic, neutral, or alkaline.

$$\text{pH} = -\log \{\text{H}^+\} \tag{6.2}$$

where $\{\text{H}^+\}$ is the hydrogen ion activity (mol/liter).

The pH of soils varies from values of 3 or less in acid soils to more than 10 in alkaline soils. However, in general, the acceptable pH range for agriculturally productive soils is pH 5 to 7 for humid regions and pH 7 to 9 for arid regions.

A major concern regarding soil pH is its impact on the concentration of inorganic ions found in the soil solution. For example, when the pH of soils is low (i.e., pH of 5 or less), the soluble concentrations of aluminum, iron, and magnesium may be elevated to levels that are toxic to plants. To decrease acidity and to reduce the soluble concentrations of these metals, agricultural lime is usually added to soils [6]. Although chemical lime, Ca(OH)_2, can be used to reduce soil acidity, in most cases agricultural lime (which is composed of oxides, hydroxides, and carbonates of calcium and magnesium) is used to control soil pH. The net effect of liming a soil is (1) increased soil pH, (2) enhanced microbial activity, and (3) increased crop production. It should be noted that in many cases, biosolids will contain substantial amounts of lime and therefore can mitigate the adverse effects of soil acid conditions [29].

Although acid conditions should be avoided to minimize the bioavailability of toxic metals, excess lime addition and its effect on soil pH may be detrimental to crop yields. For example, as the pH of acid soils is raised from 5 to 9, essential plant nutrients such as iron, manganese, and zinc may become unavailable to plants, thus limiting crop yield. Employing correct liming procedures is essential for proper management of biosolids land-application sites.

6.3.2 Soil organic matter

Soil organic matter is formed through the accumulation of partially decayed and synthesized plant and animal residues. The organic matter content of a soil is relatively small, normally varying from 2 to 6 percent (by weight). Despite its small fraction of the total soil solids material, soil organic matter has a significant impact on both soil properties and crop yield. Soil organic matter is a major source of mineral elements necessary for plant growth, including nitrogen, phosphorus, and sulfur [30]. Moreover, through its affinity for moisture, organic matter also influences the water-holding capacity of soil and the proportion of this water available to vegetation.

There are two major categories of soil organic matter, namely, humic and nonhumic substances. Humic substances are complex, high-molecular-weight organic materials whose presence results from the chemical and enzymatic transformation of plant, animal, and microbial residues. Humic substances can be further divided into fulvic acids and humic acids. Although quantitative differences exist in chemical composition, all humic substances are characterized by a nonpolar core with attached polar functional groups. The average compositions of humic and fulvic acids are $C_{187}H_{186}O_{89}N_9S$ and $C_{135}H_{182}O_{95}N_5S_2$, respectively [6]. The nonpolar nature of humic substances accounts for the strong affinity of soil organic matter for some toxic organic compounds such as hydrocarbons, herbicides, insecticides, etc.

Nonhumic substances are the intact or partially degraded compounds that are generated from plant, animal, or microbial residues. In general, nonhumic substances account for less than 25 percent of soil organic matter [43]. With time, these constituents decompose, and a fraction of the degradation products becomes incorporated into humic substances.

Functional groups found in soil organic matter include carbonyl (—COOH), phenolic and alcoholic hydroxyl (—OH), amino ($—NH_2$), and sulfhydryl (—SH) groups. All of these functional groups exhibit an acid-base character that serves to buffer the soil pH [43]. Furthermore, the ionization of the weakly acidic functional groups results in soil organic matter possessing a net negative charge. Like the negatively charged clay particles, the negative charge associated with organic matter is satisfied by the adsorption of a cation. Therefore, soil organic matter contributes to the overall CEC of soil. However, unlike the CEC of clay, the CEC of soil organic matter is strongly influenced by soil pH. Soils that possess higher pH values normally have larger CECs.

6.4 Trace Elements in Soil

A *trace element* is defined as any substance whose mass concentration in the solid phase is less than 100 milligrams per kilogram of dry soil [6]. In biosolids land-application systems, heavy metals normally are the trace elements of greatest concern. In soils, clay minerals often serve as reservoirs for heavy metals, releasing them slowly into the soil solution during the weathering of the soil minerals. Soil chemical and physical properties such as pH, redox potential, and water content will affect trace element solubility [6].

The principal processes by which heavy metals are retained by soil minerals include soil inclusion, adsorption, and solid solution formation [43]. Inclusion is characterized by the coprecipitation of the host mineral and heavy metal as morphologically distinct solids (e.g., copper sulfide in silicates). This type of soil-metal retention process normally occurs when the heavy metal and the host mineral have very different atomic structures. On the other hand, adsorption occurs when the heavy metal and the host mineral have limited compatibility so that a mixed solid phase is restricted to the interfacial region. Finally, if compatibility is high, a major element in the host mineral can be replaced by the trace element [43]. This type of soil metal retention process is termed *solid solution formation* (e.g., zinc replacing aluminum in aluminosilicate minerals).

Land application of biosolids will add significant amounts of heavy metals to soils. The U.S. biosolids regulations (40 CFR Part 503) allow metals to be applied to land but only at rates that ensure protection of public health and the environment. The metal content of soils and plants are variable depending on the soil type and plant species. Often, the interpretation of metal toxicity to plants is complicated because of interaction between nutrients (e.g., phosphorus-induced zinc deficiency). Of greater concern is the enrichment of food-chain crops with metals potentially harmful to humans and animals (e.g., arsenic, cadmium, lead, and mercury). One of the important benefits inherent in the U.S. biosolids land-application regulations is that they restrict metal additions to soil based on environmental risk impacts, which include phytotoxicity and crop yield considerations.

6.4.1 Phytotoxicity of trace elements

Phytotoxicity refers to the toxic affects imparted by substances to plants. Trace elements identified as potentially harmful to plant growth or as elements whose concentration in crops may reach levels considered to be hazardous to human and animals include aluminum, arsenic, boron, cadmium, chromium, copper iron, lead, mercury, manganese, molybdenum, nickel, selenium, and zinc [6]. In terms of their phytotoxic effects, the amounts of heavy metals that are available to plants are more important that the total quantity present in soils. The soil pH is the most important factor influencing the availability of heavy metals to plants. Except for molybdenum, the availability of heavy metals for plant uptake increases as the pH of the soil decreases [29]. Consequently, heavy metal phytotoxicity and plant accumulation are much more common in acid rather than neutral or alkaline soils. Because plant species differ in their tolerance to heavy metals, it is not possible to develop criteria associated with levels in soils that are applicable to all plant species [29]. Table 6.5 presents general information on the fate of important trace elements in the plant-soil system.

TABLE 6.5 Phytotoxicity of Trace Elements in the Plant-Soil System*

Manganese. The concentrations of manganese in most soils far exceed those which may be found in biosolids. Where plants suffer from manganese toxicity, the condition is easily corrected through liming the soil to pH levels greater than 5.5.

Chromium. The two principal oxidation states of chromium are III and VI. The Cr(III) form is found in most soils. Cr(VI) in soils is adsorbed by plants and has been shown to be phytotoxic.

Arsenic. Arsenic concentrations in soils where phytotoxicity has been observed were at levels greater than 100 kg arsenic per hectare.

Lead. Lead concentrations in biosolids are much greater than those found in soils. A number of cases of lead poisoning of large animals caused by the ingestion of forage crops contaminated by industrial emissions of lead have been reported.

Mercury. Mercury, like lead, can be harmful to the health of human beings when excessive amounts are ingested. Although above-ground parts of plants can be injured by mercury vapor, there is no existing evidence linking soil-applied mercury to phytotoxicity.

Molybdenum. Crops grown in soils high in molybdenum will absorb sufficient amounts of this element to adversely affect the health of animals that consume them.

Copper. In acid soils, copper can be phytotoxic. The tolerance of plants to copper in soil varies among species.

Nickel. The concentration of nickel in soils is highly variable. Like copper, nickel toxicity to plants normally occurs only in acid soils.

Cadmium. The concentration of cadmium in the leaf tissue of plants tends to increase as the concentrations of cadmium in soil increases. The phytotoxic tolerance of plant species to cadmium is highly variable.

Zinc. Zinc is an element essential for the growth of plants. Deficiencies of plant-available zinc are encountered frequently. Biosolids land application could be beneficial in correcting zinc deficiencies in some soils.

Boron. Although boron is essential for crop growth, when present in soil solutions at concentrations greater than 1.0 mg/liter, it is toxic to many plants.

---

*Adapted from ref. [49].

6.5 Nutrient Cycles in Soils

Of the essential plant nutrients found in biosolids, nitrogen is of the greatest concern. Plants receiving insufficient nitrogen are characterized by restricted root systems and are stunted in growth. Although essential for vegetative growth, an excess of nitrogen may adversely affect the grain and fruit quality of some crops. Reduced yields of crops with substantial economic value such as sugar cane, sugar beets, barley, apples, and peaches have been reported when excess nitrogen is supplied [6]. On the other hand, the yields of many crops such as grasses are not as susceptible to excessive nitrogen loading.

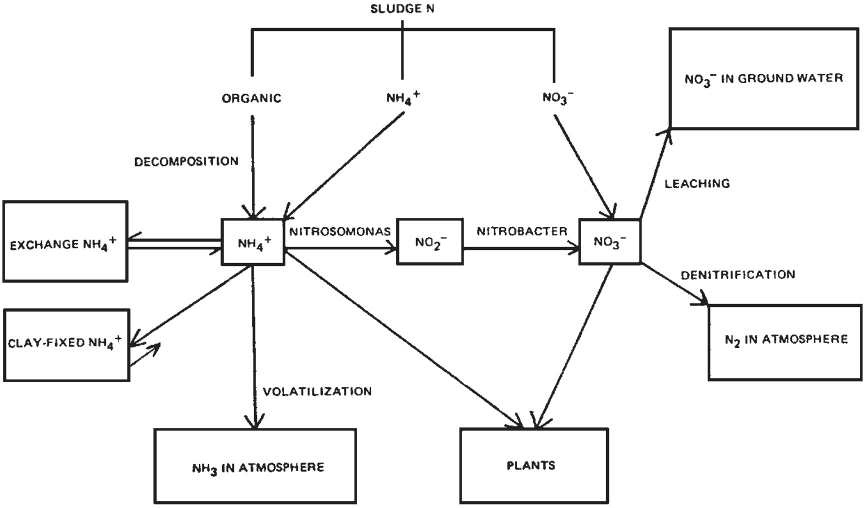


Figure 6.3 Nitrogen cycle in the soil ecosystem.

6.5.1 Soil nitrogen

There are three major forms of nitrogen in soils: (1) organic nitrogen, (2) ammonia nitrogen (NH_4^+), and (3) nitrate (NO_3^-). A simplified schematic diagram of the nitrogen cycle is shown in Fig. 6.3.

Both organic and inorganic nitrogen are added to soils during the land application of biosolids. While soluble inorganic nitrogen (NH_4^+ and NO_3^-) is readily available for plant uptake, the organic nitrogen is not and must be converted (i.e., mineralized) to inorganic forms to render it plant available. The rate at which organic nitrogen is mineralized to plant-available nitrogen is highly variable and depends on the physical and chemical properties of the biosolids applied, the physical and chemical properties of the soil, temperature, and soil water content [8,29].

In the agricultural literature, the process of mineralization of organic nitrogen is termed *nitrogen mobilization* [8]. The first step in nitrogen mobilization results in the breakdown of organic nitrogen found in plant, animal, and microbial tissue to ammonia, followed by oxidation to nitrate. The enzymatic reactions that characterize nitrogen mobilization are given by Eqs. (6.3), (6.4), and (6.5):

$$R - NH_2 + H_2O \xrightarrow[\text{Microbial activity}]{} R - OH + NH_3 + \text{energy} \qquad (6.3)$$

$$2NH_3 + H_2CO_3 \rightarrow 2NH_4^+ + CO_3^{2-} \qquad (6.4)$$

$$NH_4^- + {}^5\!/_2O_2 \rightarrow NO_3^- + 2H_2O \qquad (6.5)$$

6.5.1.1 Carbon-to-nitrogen (C/N) ratio. The organic matter content of soils will affect the quantity and type of nitrogen found in soils. This relationship is often expressed as the *carbon-to-nitrogen mass ratio* (i.e., C/N) of soils. The generally acceptable C/N ratio ranges from 8:1 to 15:1 for most agricultural soils [6]. However, for plant material, this ratio is highly variable, ranging from 20:1 for legumes to 400:1 for woody crops. The C/N ratio for biosolids (from secondary wastewater treatment) is ordinarily within the range of 4:1 to 10:1 [6,30]. From this elemental analysis, it is clear that most plant residues entering the soil are characterized by large amounts of carbon relative to nitrogen, whereas biosolids often supply nitrogen at levels in excess of plant requirements.

The C/N ratio is an important soil characteristic that controls the plant-available nitrogen, the total quantity of organic matter, and the rate of organic matter mineralization. The primary mechanism by which the C/N influences these soil processes is through its impact on the competition between soil microbes and plants for available nitrogen. Soil microbes require large amounts of soluble nitrogen as they rapidly metabolize organic residues. Therefore, if large amounts of rapidly degrading organic residues with a high C/N ratio (e.g., 75:1 or greater) are added to soils, plant-available nitrogen (e.g., nitrates) rapidly disappears from soil because microbial demand for nitrogen is high. The use of nitrate in the production of microbial tissue is described biochemically in Eq. (6.6):

$$\underset{\text{Nitrate}}{NO_3^-} + \underset{\substack{\text{Organic}\\\text{matter}}}{CHO} + CO_2 + H^+ \rightarrow \underset{\text{Microbial tissue}}{C_5H_7O_2N} + H_2O \qquad (6.6)$$

During the period of elevated microbial growth and nitrogen uptake, higher plants may die because of their inability to compete with soil microbes for soluble forms of nitrogen. This process of incorporating soluble forms of nitrogen into microbial tissue is termed *nitrogen immobilization.* The effects of nitrogen immobilization will persist until the organic residue has been sufficiently exhausted to reduce the microbial nitrogen demand. Once this occurs, the microbial growth rate will decrease, permitting plants to compete successfully for available nitrogen. As soil microbes begin to die off (i.e., endogenous respiration), the organic nitrogen contained in the microbial tissue will become available to plants. The biochemical reaction characterizing microbial endogenous respiration and the release of plant-available nitrogen is described in Eq. (6.7):

$$\underset{\text{Microbial tissue}}{C_5H_7O_2N} + 7O_2 \rightarrow \underset{\text{Nitrate}}{NO_3^-} + 5CO_2 + 3H_2O + H^+ \qquad (6.7)$$

The duration of plant nitrogen limitations can be significant depending on soil conditions and the C/N ratio of the added organic material. The greater the amount of biodegradable carbonaceous residue applied to soil, the longer is the duration of nitrogen limitation to vegetation. Therefore, the lower the C/N ratio of the organic material added to the soil, the less likely it is that the material will interfere with the crop-available nitrogen. On the other hand, land application of materials that have excessively low C/N ratios (i.e., 4:1 or less) can result in production of high nitrates within the soil solution. The elevated nitrate concentration may result in severe crop damage as well as contamination of drinking water supplies.

With regard to biosolids land application, C/N ratios of 10 or less are desirable for maximum nitrogen mineralization. However, because of the wide variety of factors affecting nitrogen mineralization rates, it is advisable to determine nitrogen mineralization rates on a site-specific basis.

6.5.1.2 Ammonia volatilization. Of importance when considering surface application of biosolids is ammonia volatilization. In situations where liquid biosolids are applied and not incorporated in the soil by injection or plowing, essentially all the ammonia may be lost through volatilization. To properly manage ammonia volatilization, it should be recognized that it is the undissociated $NH_3(g)$ form of ammonia that is volatile and not the ionized $NH_4^+(aq)$ form. The chemical relationship between these two ammonia species is described by the equilibrium expression given in Eq. (6.8):

$$NH_4^+\,(aq) + OH^- \leftrightarrow NH_3\,(g) + H_2O \tag{6.8}$$

It is clear from equilibrium considerations that as soil pH increases (i.e., $[OH^-]$ increases), the reaction shifts to the right, resulting in the increased formation of the unionized and more volatile form of ammonia, $NH_3(g)$. Therefore, ammonia volatilization losses are particularly significant in alkaline (i.e., high-pH) soils. In cases where surface application of biosolids on alkaline soils is conducted, plowing of the soil should be done immediately to minimize ammonia loss [4,68].

6.5.1.3 Nitrification. Ammonium nitrogen (NH_4^+) has a net positive charge and therefore may be held on the soil surface as an exchangeable cation. If the soil structure can be maintained to promote moisture and air movement, a large portion of the adsorbed NH_4^+ will be converted to nitrate (NO_3^-) through a microbial process called *nitrification*. Nitrification involves two coordinated steps in which NH_4^+ is first oxidized to nitrite by the bacterium *Nitrosomonas*. The second step consists of the oxidation of nitrite to nitrate by the bacterium *Nitrobacter*. The microbially mediated reactions are described in Eqs. (6.9) and (6.10):

$$\underset{\text{Ammonia}}{NH_4{}^+} + O_2 \xrightarrow[\textit{Nitrosomonas}]{} \underset{\text{Nitrite}}{NO_2{}^-} \qquad (6.9)$$

$$\underset{\text{Nitrite}}{NO_2{}^-} + O_2 \xrightarrow[\textit{Nitrobacter}]{} \underset{\text{Nitrate}}{NO_3{}^-} \qquad (6.10)$$

In aerobic soils and at temperatures greater than 15°C (59°F), essentially all $NH_4{}^+$ in soil will be converted to $NO_3{}^-$ within 2 to 4 weeks after biosolids application [6]. Under conditions favoring nitrification, the reaction rates of Eqs. (6.9) and (6.10) are comparable, so there is no net accumulation of nitrite in the environment.

The formation of nitrate is important because, unlike ammonia, nitrate can be lost from the soil through leaching. Negatively charged nitrate is repelled by the negatively charged soil colloids by a process termed *anion exclusion* [44]. Nitrate in soils in excess of crop requirements can leach and result in nitrate contamination of ground and surface water. The two areas of environmental concern involving nitrate from land-application sites are direct health effects and surface water eutrophication [29].

Regulatory limits on the rate of land application of biosolids are based on the premise that a growing crop will reduce the nitrate concentration in the soil solution to levels that will result in minimal environmental risks. Thus, in agricultural applications, the annual amount of nitrogen in biosolids applied to soils is limited by the nitrogen required by the crop grown. Typical nutrient requirements for crops grown on biosolids land-application sites are provided in Table 6.6.

6.5.1.4 Denitrification. Nitrate subjected to reducing conditions in poorly drained soils can be transformed microbially to volatile end products as described in Eq. (6.11):

$$\underset{\text{Nitrate}}{NO_3{}^-} \longrightarrow \underset{\substack{\text{Nitric}\\\text{oxide}}}{NO} \longrightarrow \underset{\text{Nitrous oxide}}{N_2O\,(g)} \longrightarrow \underset{\substack{\text{Elemental}\\\text{nitrogen}}}{N_2\,(g)} \qquad (6.11)$$

This process, termed *denitrification,* occurs when facultative bacteria use nitrate as a terminal electron acceptor during the decomposition of organic matter [6]. The end products of denitrification, nitrous oxide and elemental nitrogen, are volatile and are lost rapidly to the atmosphere.

For denitrification to occur, it is important that the soil oxygen content be limiting. The presence of oxygen in soil gas will effectively inhibit the denitrification process. In soils with good aggregate formation, it is still possible that denitrification can occur because the center of the soil aggregates may be anaerobic. Denitrification may be a significant nitrogen-removal mechanism in soils that receive liquid biosolids application due to the high moisture content and readily biodegradable organic matter [6,29].

TABLE 6.6 Nutrient Uptake Rates of Selected Crops at Biosolids Land-Application Sites*

	Nitrogen (lb/acre·yr)	Phosphorus (lb/acre·yr)	Potassium (lb/acre·yr)
Forage crops			
Alfalfa	225–540	22–35	175–225
Brome grass	130–225	40–55	245
Coastal bermuda grass	400–675	35–45	225
Kentucky blue grass	200–270	45	200
Quack grass	235–280	30–45	275
Reed canary grass	335–450	40–45	315
Rye grass	200–280	60–85	270–325
Sweet clover	175	20	100
Tall fescue	150	30	300
Orchard grass	250–350	20–50	225–315
Field crops			
Barley	125	15	20
Corn	175–200	20–30	110
Cotton	75–110	15	40
Grain sorghum	135	15	70
Potatoes	230	20	245–325
Soybeans	250	10–20	30–55
Wheat	160	15	20–45

*Adapted from refs. [51,68].

6.5.2 Estimation of plant-available nitrogen from biosolids

To estimate the plant-available nitrogen furnished from biosolids, the contributions from both the inorganic and organic forms of nitrogen in biosolids must be quantified. In this approach, it is assumed that both the ammonia and nitrate present in soil after biosolids application are available for plant uptake during the present crop growing season [68]. In addition to the inorganic nitrogen, it is assumed that a certain percentage of organic nitrogen from biosolids will be mineralized and therefore be plant available during the present growing season, with the remainder being available in future years.

The fraction of organic nitrogen mineralized from biosolids is related to its processing history and its resulting characteristics. In general, the greater the degree of biosolids processing within the wastewater treatment plant, the lower is the rate of organic nitrogen released from biosolids after their application to soils. The nitrogen mineralization percentages (i.e., *F* factors) provided in Table 6.7 can be employed to calculate the plant-available nitrogen from previous biosolids applications.

The amount of plant-available nitrogen also depends on the biosolids application method used. For example, significant amounts of ammonia are assumed lost to the atmosphere through volatilization

TABLE 6.7 Organic Nitrogen Mineralization Percentages (F factors) for Various Biosolids*

Time after biosolids application (years)	Unstabilized primary and waste-activated biosolids (% of N_o)	Aerobically digested biosolids (% of N_o)	Anaerobically digested biosolids (% of N_o)	Composted biosolids (% of N_o)
0–1	0.40	0.30	0.20	0.10
1–2	0.20	0.15	0.10	0.05
2–3	0.10	0.08	0.05	0.03
3–4	0.05	0.04	0.03	0.03
4–5	0.03	0.03	0.03	0.03
5–6	0.03	0.03	0.03	0.03
6–7	0.03	0.03	0.03	0.03
7–8	0.03	0.03	0.03	0.03
8–9	0.03	0.03	0.03	0.03
9–10	0.03	0.03	0.03	0.03

NOTE: % of N_o = percent organic nitrogen in biosolids.
*Adapted from ref. [51].

when liquid biosolids are applied to the soil surface and allowed to dry before being incorporated. If liquid biosolids are injected into soil, or if dewatered biosolids are applied, ammonia loss is assumed to be negligible.

Given the assumptions that describe the fate of the various forms of nitrogen in land-aplied biosolids, the plant-available nitrogen in a particular year is the sum of the following terms:

- All the nitrate present in the biosolids is assumed to be plant-available.
- All or a fraction of the ammonia present in the biosolids is assumed to be plant-available. If biosolids are liquid and surface applied, 50 percent of the ammonia is assumed to be lost through volatilization. If the biosolids are liquid and incorporated (i.e., injected), or if the biosolids are dewatered and applied in any manner, 100 percent of the ammonia in the biosolids is assumed to be plant-available.
- A fraction of the organic nitrogen present in biosolids is mineralized during the first year after application and is assumed to be plant-available. This magnitude of this fraction will depend on the degree of biosolids processing (see Table 6.7).
- A fraction of the organic nitrogen in the biosolids applied during previous years (if any) that is mineralized during the current year would be assumed available for plant uptake.

For the first year of biosolids application, Eq. (6.12) can be used to estimate the plant-available nitrogen N_p. The use of Eq. (6.12) is illustrated in Example 6.2.

$$N_p = S\,[\,(NO_3^-) + K_v\,(NH_4^+) + F_{(\text{year } 0\text{–}1)}\,(N_o)\,]\,10 \qquad (6.12)$$

where N_p = plant-available nitrogen from this year's biosolids application only, kg/ha
S = biosolids application rate, metric tons per hectare
NO_3^- = percent nitrate in biosolids
K_v = volatilization factor (0.5 for surface-applied unincorporated liquid biosolids and 1.0 for incorporated or dewatered biosolids)
NH_4^+ = percentage ammonia in biosolids
$F_{(\text{year } 0\text{–}1)}$ = first year mineralization factor for organic nitrogen in biosolids (see Table 6.7)
N_o = percentage organic nitrogen in biosolids
10 = conversion factor (1000 kg/metric ton) (0.01/1%) = (10 kg/mt · %)

NOTE: One hectare equals 2.47 acres, and one metric ton equals 1000 kg (2204 lb).

Example 6.2 The Dubois County Water Improvement District is considering land applying its biosolids to a nearby mining site. From preliminary estimates, a biosolids surface application rate of 5 metric tons (dry weight) per hectare of anaerobically digested liquid biosolids will be applied at the site. For this application rate, estimate the plant-available nitrogen for the first year of biosolids application. The biosolids chemical analysis indicated no nitrate (i.e., $NO_3^- = 0$ percent), whereas the ammonia and organic nitrogen were 1.5 and 3 percent, respectively (all reported on a dry weight basis).

solution Using Eq. (6.12), estimate the plant-available nitrogen from this year's biosolids application:

$$N_p = S\,[\,(NO_3^-) + K_v\,(NH_4^+) + F_{(\text{year } 0\text{–}1)}\,(N_o)\,]\,10$$

$$= 5\,[\,(0) + 0.5\,(1.5) + 0.2\,(3.0)\,]\,10$$

$$= 67.5 \text{ kg nitrogen/ha (60.2 lb/acre)}$$

NOTE: When liquid biosolids are surface applied, $K_v = 0.5$ (i.e., 50 percent of the ammonia is lost through volatilization).

For subsequent years, a fraction of the organic nitrogen will be available to plants through the process of mineralization. Using the *F* factors (Table 6.7), the fraction of organic nitrogen mineralized can be estimated. Example 6.3 illustrates this approach.

Example 6.3 The Dubois County Water Improvement District (Example 6.2) has been permitted to apply biosolids to the mining site in one single application. Given the limited biosolids application rate of 5 metric tons per hectare, estimate the fraction of organic nitrogen that will be plant available during years 1, 2, and 3 from the one-time biosolids application.

solution

Step 1. Estimate the mass of organic nitrogen applied per hectare:

$$\text{Organic nitrogen} = 0.03\ (5\ \text{metric tons/ha})\ (1000\ \text{kg/metric ton})$$

$$= 150\ \text{kg/ha}$$

Step 2. Using the *F* factors from Table 6.7, estimate the amount of organic nitrogen mineralized. The *F* factors for anaerobically digested biosolids for years 1, 2, and 3 are given as follows:

Year	*F* factor
1	0.2
2	0.10
3	0.05

Using the *F* factors, the amount of organic nitrogen mineralized for years 1, 2, and 3 can be estimated as shown in the following:

$$\text{N}_\text{o}\ \text{mineralized in year 1} = 0.2 \cdot 150 = 30\ \text{kg/ha}$$

$$\text{N}_\text{o}\ \text{remaining in year 2} = 150 - 30 = 120\ \text{kg/ha}$$

$$\text{N}_\text{o}\ \text{mineralized in year 2} = 0.1 \cdot 120 = 12\ \text{kg/ha}$$

$$\text{N}_\text{o}\ \text{remaining in year 3} = 120 - 12 = 108\ \text{kg/ha}$$

$$\text{N}_\text{o}\ \text{mineralized in year 3} = 0.05 \cdot 108 = 5.4\ \text{kg/ha}$$

If the question is how much organic nitrogen is mineralized in a given year, Eq. (6.13) can be used in lieu of the *F* factors. It should be noted that use of Eq. (6.13) requires knowledge of the mineralization factor K_m, which has the units of kilograms of nitrogen released per metric ton of biosolids per percent of organic nitrogen in biosolids (i.e., kg/metric ton·% N_o).

$$\text{N}_\text{m} = K_m \text{N}_\text{o} S \tag{6.13}$$

where N_m = quantity of organic nitrogen mineralized in the year under consideration

K_m = mineralization factor for the year under consideration (Table 6.8)

N_o = percentage organic nitrogen originally present in the biosolids

S = biosolids application rate (metric tons/ha)

TABLE 6.8 Mineralization Factors K_m for Various Types of Biosolids*

Time after biosolids application (years)	Unstabilized primary and waste-activated biosolids	Aerobically digested biosolids	Anaerobically digested biosolids	Composted biosolids
0–1	4.00	3.00	2.00	1.00
1–2	1.20	1.05	0.80	0.45
2–3	0.48	0.45	0.36	0.25
3–4	0.22	0.21	0.21	0.25
4–5	0.12	0.16	0.20	0.24
5–6	0.12	0.15	0.19	0.23
6–7	0.12	0.15	0.19	0.23
7–8	0.11	0.15	0.18	0.22
8–9	0.11	0.15	0.18	0.21
9–10	0.11	0.15	0.17	0.21

*Adapted from ref. [51].

Mineralization factors for the various types of biosolids are sumarized in Table 6.8. Use of Eq. (6.13) is illustrated in Example 6.4.

Example 6.4 For the Dubois County Water Improvement District biosolids beneficial-use program (see Example 6.3), estimate the mass of organic nitrogen mineralized in years 1, 2, and 3 from the one-time biosolids application using the mineralization factors K_m.

solution

Step 1. Obtain the mineralization factors for anaerobically digested biosolids from Table 6.8. The mineralization factors for years 1, 2, and 3 are given as follows:

Year	K_m
1	2.0
2	0.8
3	0.36

Step 2. Using the mineralization factors, estimate the amount of organic nitrogen mineralized in years 1, 2, and 3 using Eq. (6.13):

$$\mathrm{N_m}\ \text{year } 1 = 2.00 \cdot 3 \cdot 5 = 30\ \text{kg/ha}$$

$$\mathrm{N_m}\ \text{year } 2 = 0.80 \cdot 3 \cdot 5 = 12\ \text{kg/ha}$$

$$\mathrm{N_m}\ \text{year } 3 = 0.36 \cdot 3 \cdot 5 = 5.4\ \text{kg/ha}$$

Thus far, calculations of the plant-available nitrogen are based on a one-time biosolids application. Biosolids management programs that apply biosolids annually are more complex because the organic nitrogen mineralized from all previous years' biosolids applications must be accounted for in the plant-available nitrogen estimation. Example 6.5 illustrates the general approach for estimating the plant-available nitrogen for programs employing annual biosolids application.

Example 6.5 The Perry County Wastewater Treatment Plant is annually applying 5 metric tons per acre (dry weight) of liquid anaerobically digested biosolids containing no nitrate, 1.5 percent ammonia, and 3.0 percent organic matter. If the biosolids are being surface applied, estimate the plant-available nitrogen during each of the first 3 years of application.

solution

Step 1. Using Eq. (6.12), the plant-available nitrogen (N_p) for the first year may be estimated as follows:

$$\begin{aligned} N_p &= S\,[\,(NO_3^{-}) + K_v\,(NH_4^{+}) + F_{(\text{year }0\text{–}1)}\,(N_o)\,]\,10 \\ &= 5\,[\,(0) + 0.5\,(1.5) + 0.2\,(3.0)\,]\,10 \\ &= 67.5 \text{ kg nitrogen/ha} \end{aligned}$$

Step 2. For year 2, the plant-available nitrogen will be the sum of the plant-available nitrogen from the biosolids application from year 2 (N_p) and the organic nitrogen mineralized from the first year's biosolid```s application (N_m).

$$\begin{aligned} N_p &= N_p\text{ (second year)} + N_m\text{ (first year)} \\ &= 67.5\text{ kg/ha} + (K_m N_o S) \\ &= 67.5\text{ kg/ha} + (0.8 \cdot 3 \cdot 5) \\ &= 79.5\text{ kg/ha} \end{aligned}$$

Step 3. For year 3, the plant-available nitrogen will be the sum of the plant-available nitrogen from the biosolids application from year 3 (N_p) and the organic nitrogen mineralized from the first and second years biosolids applications (N_m).

$$\begin{aligned} N_p &= N_p\text{ (third year)} + N_m\text{ (first year)} + N_m\text{ (second year)} \\ &= 67.5\text{ kg/ha} + (0.8 \cdot 3 \cdot 5) + (0.36 \cdot 3 \cdot 5) \\ &= 84.9\text{ kg/ha} \end{aligned}$$

6.6 Phosphorus in Soil

After nitrogen, the most critical nutrient influencing plant growth and yield is phosphorus. Sources of phosphorus in soil include (1) biosolids, (2) commercial fertilizers, (3) animal manure, (4) plant residues, (5) industrial and domestic wastes, and (6) native forms of phosphorus

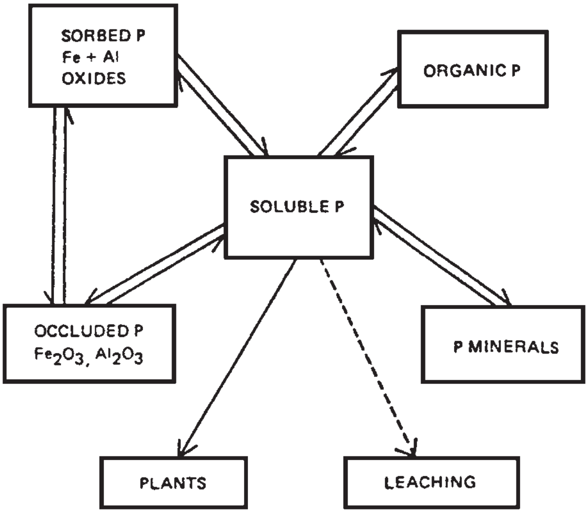


Figure 6.4 Cycling of phosphorus in soil.

present in the soil [6]. The biological cycling of phosphorus from soil to higher plants is illustrated in Fig. 6.4.

Plant roots absorb inorganic phosphorus and to a lesser extent organic phosphorus compounds and translocate them to above-ground plant parts [6]. Typical uptake rates of phosphorus by crops are given in Table 6.9. When biosolids are applied to soil, the rapid growth of soil microorganisms results in large portions of the applied phosphorus becoming immobilized in the form of microbial tissue. However, because of endogenous microbial decay, immobilized phosphorus ultimately is released in either organic or inorganic forms.

The three major forms of organic phosphorus found in soils are (1) phytin, (2) nucleic acids, and (3) phospholipids [6]. As microorganisms decompose these compounds, the phosphorous released is available to react with inorganic compounds or to be metabolized by vegetation.

The soluble or plant-available forms of phosphorus found in soil are present mainly as calcium phosphates. The other major forms of phosphate found in soil (i.e., iron and aluminum phosphates) are extremely insoluble and normally unavailable for plant uptake [6]. However, as plants deplete the soluble calcium phosphates, the equilibria with adsorbed phosphorus and phosphorus minerals are shifted, resulting in replenishment of soluble phosphorus.

6.6.1 Phosphorus control at biosolids land-application sites

Although leaching of phosphorus from soils is minimal, losses of phosphorus from biosolids land-application sites can be significant if soil

TABLE 6.9 Typical Phosphorus Uptake Rates for Crops*

	Uptake (lb/acre · yr)
Forage crops	
Alfalfa	22–35
Brome grass	40–55
Bermuda grass	35–45
Kentucky blue grass	45
Quack grass	30–45
Reed canary grass	40–45
Rye grass	60–85
Sweet clover	20
Tall fescue	30
Orchard grass	20–50
Field crops	
Barley	15
Corn	20–30
Cotton	15
Grain sorghum	15
Wheat	15

*Adapted from refs. [51,68].

erosion is not taken into account during site design [29]. Crops that have dense covers such as grass and forests offer the best protection against soil erosion. Small grains, such as wheat, offer an intermediate level of erosion protection, whereas row crops, such as corn and soybeans, offer relatively little protection against soil erosion.

The major differences in the ability of crops to maintain soil cover emphasizes the need for appropriate crop rotation to reduce soil erosion. The inclusion of a close-growing forage crop in rotation with row crops will help control both erosion and surface runoff. Other methods that can be used to minimize erosion and phosphorus losses are called *support practices*. These include contour tillage, contour strip cropping, terrace systems, and conservation tillage [6].

Contour tillage involves the cultivation of crops at approximate right angles to the slope of the land surface. On long-sloped surfaces, the fields may be laid out in narrow strips across the incline, alternating the tilled crops (e.g., corn) with alfalfa. Such a layout is called *strip cropping*. The width of the strips will depend primarily on the slope angle and the permeability of the land.

Contour strip cropping is often guarded by diversion ditches between fields [11,12,27]. Contour strip cropping is illustrated in Fig. 6.5. When less expensive methods of retarding erosion such as contour strip cropping are not feasible, terraces can be constructed across the slope (Fig. 6.6). The terraces catch the overland flow of moisture and allow it to be directed away from the site at a gentle grade.

(a)

(b)

Figure 6.5 Illustrations of contour strip cropping practice.

Conservation tillage includes reduced tillage and no-till systems. In other words, in these types of erosion control practices, plowing for weed control and planting is reduced or eliminated. By reducing the acreage that is laid bare, soil erosion is minimized. The various types of conservation tillage practices are described in Table 6.10.

In evaluating conservation tillage practices, it is noted that conventional tillage involves plowing (from one to three passes with a harrow disk plow or equivalent), crop planting, and sometimes subsequent tillage with a cultivator (Fig. 6.7). On the other hand, conservation tillage practices vary from reduced excess tillage to the no-tillage approach.

Field research has confirmed that conservation tillage imparts greater soil protection than conventional tillage practices. While conventional tillage methods (e.g., moldboard plow systems) leave only 1

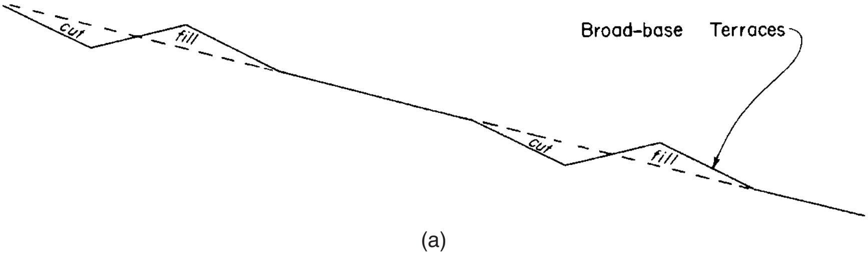
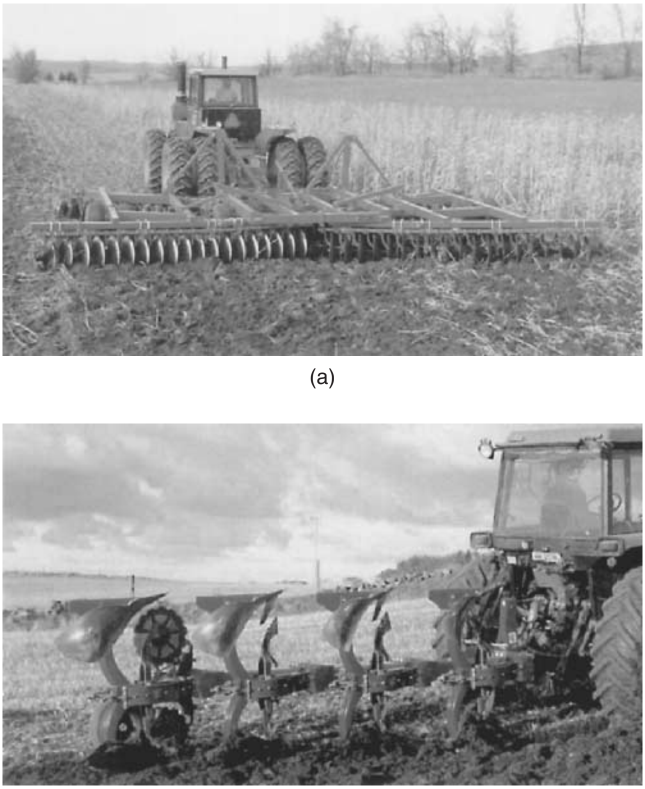


Figure 6.6 (*a*) Terrace construction on a slope. (*b*) Schematic diagram of a terraced slope.

TABLE 6.10 Conservation Tillage Practices Applicable to Biosolids Land-Application Sites*

Tillage Practice	Operations
Stubble mulch	Sweeps, blades, or disks undercut residues, loosen soil and kill weeds
Reduced tillage	
Fall plow or chisel, cultivate	Moldboard plow or chisel with minimum secondary tillage
Spring plow, plant in wheel track	Moldboard plow; no other tillage
Disk fall and/or spring and plant	Tandem disks; most residues left on surface
Till plant	No plowing; sweeps provide trash-free zone for planting
No tillage	No primary tillage; rotary tillers or coulters open soil for seed planting

*Adapted from ref. [51].



(a)

(b)

Figure 6.7 (*a*) A harrow disk plow. (*Courtesy of Hutch Master Division, Lear Siegler, Inc.*) (*b*) A deep-furrow plow. (*Courtesy of Kverneland Klepp AS.*)

to 5 percent of the soil covered with crop residues, reduced-tillage systems (e.g., chiseling or disking) typically leave 15 to 25 percent crop residue coverage, and the no-tillage system can be expected to leave from 50 to 75 percent of the land covered with crop residues. These differences in land cover have significant effects on both soil erosion and runoff. Section 6.10.2 describes an approach for estimating the soil erosion potential of a biosolids land-application site using the universal soil-loss equation.

6.7 Soil Physics

There are a number of different ways in which a soil can be characterized physically. For the biosolids land-application engineer, the characteristics of soils that are of interest include particle size, gradation, texture, and structure.

6.7.1 Particle size

The physical process of separating soil into its particle size groups is known as *mechanical analysis*. The soil is first passed through a series of sieves to separate the particles larger than silt size. The sieve analysis consists of separation of the soil into its fractions by shaking the dry, loose material through a nest of sieves of increasing fineness. The practical lower size limit for the use of sieves is a number 200 sieve, which can be used to quantify the fraction of particles with diameters of at least 0.1 mm (*ca.* 0.075 in).

Because of its impact on water retention and cation exchange capacity (CEC), it is often desirable to determine the distribution of particle sizes that have a diameter smaller than 0.1 mm. Separation of these silt and clay fractions is accomplished by a process known as *wet mechanical analysis*. The two most common wet mechanical methods are (1) the pipette method and (2) hydrometer analysis. Descriptions of these methods are beyond the scope of this text. For information on wet mechanical methods, the reader is, are referred to refs. [6,17,33].

The combined results of particle size analyses (i.e., both wet and dry methods) may be summarized in several different ways. The engineer normally presents the results in the form of a grain size distribution curve (Fig. 6.8). The particle diameters (or sieve sizes) are plotted horizontally on a logarithmic scale of a semilog plot, whereas the percentage by weight of the material passing any given sieve size is plotted vertically on the arithmetic scale. These points are then connected to form a smooth curve that is called the *grain size distribution*.

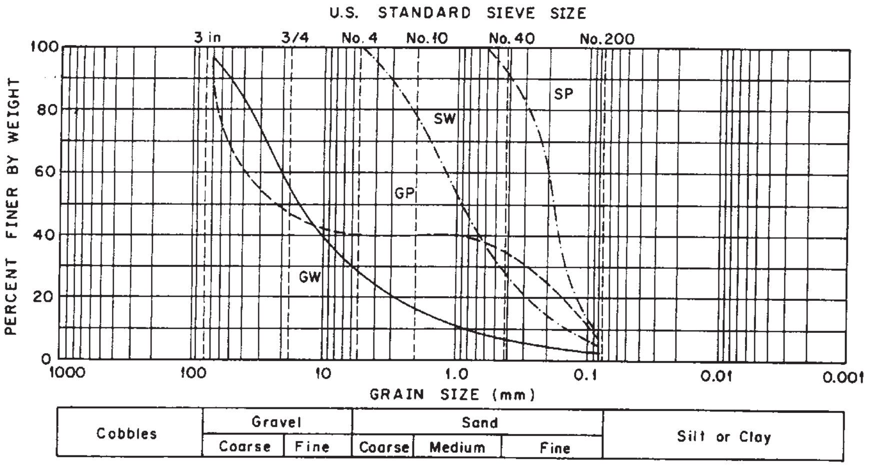


Figure 6.8 Soil grain size distribution curve. GW, well-graded gravels (little or no fines); GP, poorly graded gravels (little or no fines); SW, well-graded sands (little or no fines); SP, poorly graded sands (little or no fines). (*Adapted by permission from ref.* [*32*].)

6.7.2 Gradation

The distribution of soil particle sizes is known as its *gradation.* The effective particle size of a given soil is called the *Hazen effective size* and is defined as a particle size for which 10 percent of the soil (by weight) is smaller than that size. The Hazen effective size is designated by the symbol D_{10}. The Hazen effective sizes of sands and gravel can be related to their permeability and therefore are of importance in designing the drainage systems for biosolids land-application projects.

A soil that has a nearly vertical grain size distribution curve is called a *uniform soil.* Uniform soils are soils that have particles of basically one size. If the grain size distribution curve extends over a large particle size range, the soil is termed *well graded,* which implies that various particle sizes are found in the soil. The difference between a uniform and a well-graded soil can be defined numerically by the *uniformity coefficient* C_u. The uniformity coefficient is defined as the ratio between the grain diameter for which 60 percent of the soil (by weight) is smaller than on the grain size distribution curve (i.e., D_{60}) and D_{10}. The expression for the uniformity coefficient is given in Eq. (6.14):

$$C_u = \frac{D_{60}}{D_{10}} \tag{6.14}$$

Soils with a C_u less than 4 are termed *uniform soils,* whereas those with uniformity coefficients greater than 4 are *well graded* [14] . For agricultural biosolids land-treatment practices, a well-graded soil is desirable because such a soil normally has better tillage characteristics and a higher moisture retention capacity [14]. Conversely, a uniform soil (e.g., sand) normally has a lower moisture retention capacity (i.e., high moisture transmission potential) and therefore is ideal for construction of drainage systems [14,29,30].

6.7.3 Soil texture

The term *soil texture* describes the proportions of the various sizes of particles in a soil. Texture is a soil characteristic that affects both water transmission properties and water retention. In general, coarse-textured soils have a higher water transmission capacity and lower water retention than fine-textured soils. Texture is readily measurable in agricultural soils by determining the amount (by weight) of sand, silt, and clay [6,14]. Once the weight percentage of these particles is determined, the soil can be classified using the U.S. Department of Agriculture (USDA) soil texture classification triangle (Fig. 6.9).

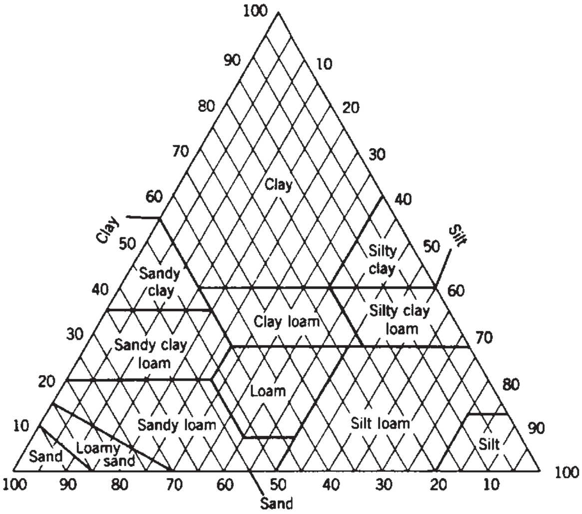


Figure 6.9 USDA soil texture classification triangle.

6.7.4 Soil structure

Soil structure refers to the arrangement of soil particles into aggregates. Regardless of texture, soil particles can have different structures depending on whether a mass of particles is relatively porous or dense [6,14,33]. The size, shape, and arrangement of the aggregates and the shape and size of the pore spaces give the soil its structure. Important terms related to soil structure include *soil type, soil class,* and *soil grade.* The type of soil structure is determined by the shape and arrangement of aggregates. The class of soil structure refers to the size of aggregates, whereas the grade of soil structure refers to the particle size distribution [6]. Soil structure is a characteristic that is useful in evaluating and correlating the water-transmission capacity of soils with similar textures. The principal types of soil structures with which the biosolids land-application engineer should be familiar are given in Table 6.11.

Based on the descriptions provided in Table 6.11, the granular structure is the most suitable structure for agricultural land application of biosolids. However, it should be noted that soil structure is one of the soil properties that can be affected by human activity. For example, soil compaction due to overland traffic and/or animal grazing will affect soil structure. In addition to field traffic, soil structure is also influenced by root density, soil organic matter, and soil depth. Soil scientists use the term *optimal soil tilth* (or *friable*) as the physical condition in which the soil is a loose, porous aggregate that allows rapid movement of air and water and unobstructed root growth. Crops with vigorous root systems that provide a high degree of vegetative cover

TABLE 6.11 Major Types of Soil Structures*

Platy. In this type of structure, the aggregates are arranged in horizontal sheets. The water-transmission potential varies with the class of structure and is usually at its highest for medium-platy material.

Prismatic. These structure types are usually found in the upper horizons of a soil profile. The aggregates form prisms that have longer vertical than horizontal axes.

Angular blocky. The aggregates are in dense blocks bounded by planes intersecting at relatively sharp angles. A soil with this structure usually has good water-transmission properties in both horizontal and vertical directions.

Granular. The granular type of structure is formed of uniformly sized, relatively nonporous aggregates, spherical or polyhedral in shape. Soils with this type of structure usually have good water-transmission potential both vertically and horizontally.

Crumb. The type of structure is the same as granular except aggregates are very porous. It has good water-transmission potential in both vertical and horizontal directions.

Massive. Structure type is massive when the soil is coherent and there is no observable aggregation or definite orderly arrangement of natural lines of weakness. A soil with massive structure has neither class nor grade and negligible water-transmission potential.

Single grain. Single-grain structure is a noncoherent soil with no observable aggregation, such as sand. Usually soil with single-grain structure has good vertical and horizontal water-transmission potential. A single-grain soil has neither structural class nor grade.

*Adapted from ref. [6].

and do not require intensive mechanical cultivation during the growing season tend to promote soils of optimal tilth. Finally, the soil salinity can affect soil structure by influencing aggregate formation. In general, saline soils result in a dispersed soil structure characterized by a low water- and air-transmission capacity [14].

6.8 Soil Water

There are various categories of soil water about which the biosolids land-application engineer should be familiar prior to the development of the design and/or the establishment of spwecific management practices at a biosolids land-application site. These include gravitational water, capillary water, and hygroscopic water (Fig. 6.10). Proper management of moisture is necessary to meet the evapotranspiration needs of the crops as well as for the proper management of salts (irrigated systems).

6.8.1 Soil moisture content

The moisture content of soil is expressed in one of two ways. In the dry-weight method, the soil moisture is expressed as a percentage of

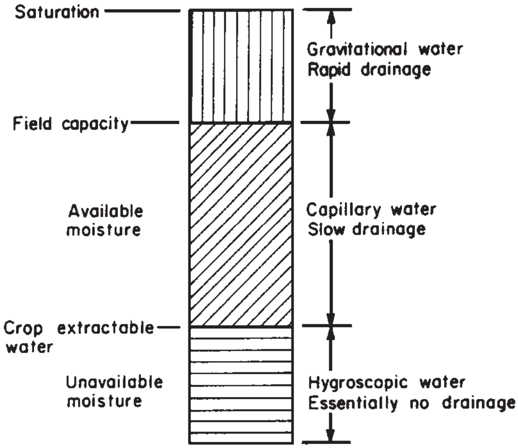


Figure 6.10 Classes of soil water. (*Adapted by permission from ref.* [*14*].)

the dry weight of soil. To determine the soil moisture content using this approach, a sample of soil is weighed and then put in a drying oven set at 103 to 105°C (217 to 221°F) [14]. After 6 hours in the oven, the sample is removed and reweighed to obtain a dry sample weight. Equation (6.15) is then used to estimate the soil moisture content on a dry-weight basis:

$$M = \frac{w - d}{d} \cdot 100 \tag{6.15}$$

where M = moisture content, %
w = weight of original soil sample, lb (kg)
d = weight of dried sample, lb (kg)

Soil moisture also can be expressed as a volume percentage of the soil sample. Typical units used to express volumetric water content include cubic feet of H_2O per cubic feet of soil, inches of H_2O per foot of soil depth, centimeters of H_2O per centimeter of soil depth, etc. It is important to note that the volumetric units given in terms of length can be converted into a percentage. For example, 4 in of H_2O per foot of soil is equivalent to a moisture content of 33.3 percent (volume basis).

To express the soil moisture content on a volumetric basis, the volume of the soil sample (or its bulk density) must be known. It should be noted that volumetric moisture contents normally are used in agricultural production because rainfall and irrigation normally are expressed in depths of water. Example 6.6 illustrates the equivalence of the soil moisture content measured on a dry-weight basis and volumetric basis.

Example 6.6 A 10-lb soil sample loses 2.5 lb of moisture on drying in an oven set at 105°C (221°F). What is the moisture content of this soil on a dry-weight and a volumetric basis (inches of H_2O per inch of soil depth) if the initial soil sample has a volume of 0.15 ft^3? Assume that the density of water is 62.4 lb/ft^3.

solution

Step 1. Estimate the moisture content on a dry-weight basis using Eq. (6.15):

$$M = \frac{w - d}{d}\,100 = \frac{10\ \text{lb} - 7.5\ \text{lb}}{7.5\ \text{lb}}\,100 = 33.3\ \text{percent}$$

Step 2. Estimate the moisture content on a volumetric basis taking into account the fact that there was 2.5 lb of water in the 0.15-ft^3 soil sample.

Moisture content (volumetric basis)

$$= (2.5\ \text{lb}\ H_2O/0.15\ \text{ft}^3\ \text{soil}) \cdot (1\ \text{ft}^3\ H_2O/62.4\ \text{lb}\ H_2O) \cdot (12\ \text{in}/1\ \text{ft})$$

$$= 3.2\ \text{in}\ H_2O/\text{ft of soil (or 26.6 percent)}$$

6.8.2 Specific yield

Gravitational water is defined as that water that is drained from the soil profile by the force of gravity within a 24- to 48-hour period [32,50]. In agricultural texts, gravitational water is sometimes referred to as the *specific yield S*. Soil moisture retention, as well as drainage design, is strongly influenced by specific yield. Specific yield can be expressed as a percentage of the total volume of moisture in saturated soil, as illustrated by Eq. (6.16):

$$\text{Specific yield } S = \frac{\text{volume of water drained}}{\text{total volume of saturated soil}}\,100 \qquad (6.16)$$

The optimal specific yield in the root zone of soil should be approximately 6 to 10 percent [6,32,50]. An agricultural soil in this range would have sufficient aeration, water transmission, and water-holding properties for optimal crop growth. When the specific yield is less than 3 percent, drainage becomes difficult and expensive. For specific yields greater than 16 to 18 percent, aeration and water transmission are good, but the soil moisture-holding capacity is low [6,32,50].

6.8.3 Field capacity and permanent wilting point

The soil moisture content remaining after 2 days (i.e., 48 hours) of drainage is termed the *soil field capacity* (FC). Field capacity repre-

TABLE 6.12 Field Capacities of Typical Agricultural Soils*

Soil texture	Range†	Typical value
Sand	0.10–0.20	0.15
Sandy loam	0.15–0.27	0.21
Loam	0.25–0.36	0.31
Clay loam	0.31–0.42	0.36
Silty clay	0.35–0.45	0.40
Clay	0.39–0.49	0.44

*Adapted from ref. [6].
†Units of feet of H_2O per foot of soil.

sents the moisture retained by soil micropores under free-drainage (i.e., unsaturated) conditions. Soil texture has a significant impact on field capacity. Due to their proportionately larger micropore spaces, soils having a large clay content will possess a larger field capacity than either sandy or gravely soils. Table 6.12 summarizes the range of field capacities measured in typical agricultural soils.

Field capacity can be evaluated under field conditions by measuring the moisture content of soil after 48 hours of gravitational drainage or in the laboratory using a pressure-plate apparatus [6,29]. Using a pressure-plate apparatus, a negative pressure is imposed on the soil sample for a specified period of time (usually 6 hours). The soil water tension at field capacity varies from soil to soil, but laboratory tests have shown that field capacity can be approximated by the soil moisture content after a saturated soil sample has been subjected to a tension of $-\frac{1}{3}$ atm for 6 hours [6,29].

Because of the attraction of water to soil particles, only a fraction of the field capacity will be available for plant use. As water is removed through the negative tension exerted by plant roots, a soil water content is reached where the water is held too tightly by soil particles to be removed by crops. This soil moisture content is called the *permanent wilting point* (PWP). The permanent wilting point is defined as the moisture content at which plants cannot extract the required amount of moisture from soil necessary for growth [6,14]. The permanent wilting point is a function of soil texture, structure, and plant variety. Like field capacity, the permanent wilting point can be measured in the laboratory using a pressure-plate apparatus. Although PWP varies between soils and plant species, it is generally accepted that the PWP is the moisture content in a soil sample after a saturated soil sample has been subjected to a tension of −15 atm for 1 hour [29]. Table 6.13 lists permanent wilting points of typical agricultural soils. It should be noted that clayey soils have much higher PWPs than those soils having little or no clay [6,29]. Clay

TABLE 6.13 Permanent Wilting Points of Typical Agricultural Soils*

Soil texture	Range†	Typical value
Sand	0.03–0.10	0.07
Sandy loam	0.06–0.12	0.09
Loam	0.11–0.17	0.14
Clay loam	0.15–0.20	0.18
Silty clay	0.17–0.22	0.20
Clay	0.19–0.24	0.21

*Adapted from ref. [6].
†Units of feet of H_2O per foot of soil.

particles, with their large specific surface areas, hold moisture more persistently than do sandy or gravely soils.

6.8.4 Available water capacity

The difference in moisture content between field capacity (FC) and the permanent wilting point (PWP) of a soil is termed the *available water capacity* (AWC). Available water capacity represents the moisture reservoir in soil that supports crop growth (Fig. 6.11).

As crops transpire moisture, this reservoir is depleted. The AWC can be depleted until the PWP is reached without adversely affecting crops. Conversely, moisture in terms of precipitation and/or irrigation water can increase the AWC reservoir until FC is achieved. Water added above this moisture content will result in the formation of soil leachate. The approach for estimating AWC is illustrated in Example 6.7.

The AWC varies with the amount of soil micropore space, which, in turn, depends on the soil texture, structure, and organic matter content. For example, a coarse sand will have a much smaller AWC than a silt loam soil. The relatively large AWC of the silt loam soil is a result of its significant quantities of organic matter and clay which translates into greater amounts of micropore space. However, in moving from a silt loam to a finer soil texture (e.g., clay soil), the AWC actually may decrease. The reason for this is that the increase in FC due to larger micropore space may be negated by a larger increase in the clay's PWP [6].

Example 6.7 A 5-lb soil sample having a volume of 0.06 ft^3 is brought into the laboratory for water retention analyses. The sample is first saturated with moisture, after which it is transferred into a pressure-plate apparatus. The results of the pressure-plate analyses are as follows:

Analytic conditions	Moisture content (dry-weight basis)
$-^1/_3$ atm tension for 6 h	24%
−15 atm tension for 1 h	8%

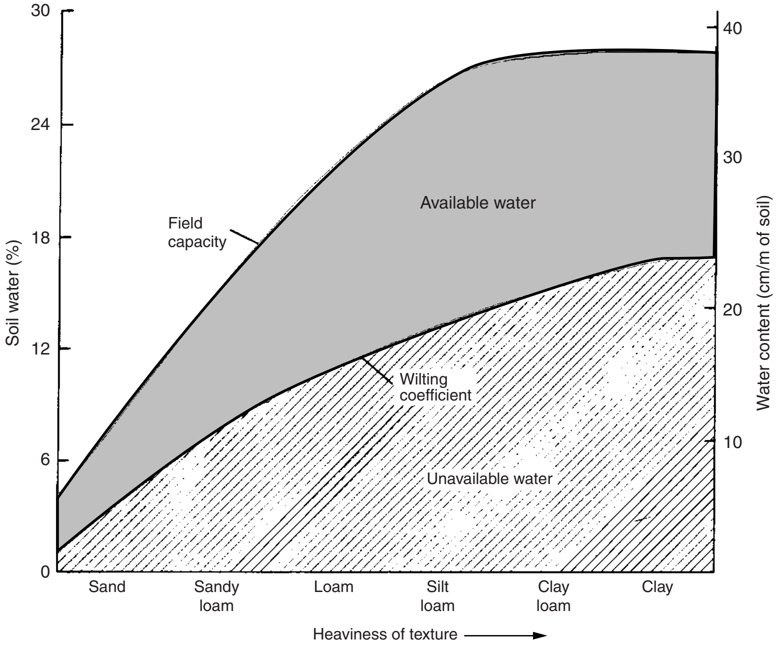


Figure 6.11 General relationship between field capacity, permanent wilting point, and available water capacity. (*Adapted by permission from ref.* [*6*].)

From the laboratory data, estimate the soil field capacity (FC), permanent wilting point (PWP), and available water capacity (AWC).

solution

Step 1. To determine field capacity and permanent wilting point, convert the moisture contents on a dry-weight basis to a volumetric basis. This procedure requires estimating the amount of moisture lost during each test using Eq. (6.15):

Field capacity ($-^1/_3$ atm tension):

$$M = \frac{W - d}{d} 100$$

$$24 = \frac{5 \text{ lb} - d}{d} 100$$

$$d = 4.03 \text{ lb}$$

Moisture lost during field capacity test $= 5 - 4.03 = 0.97$ lb H_2O

Field capacity (volumetric basis)

$$= (0.97 \text{ lb } H_2O/0.06 \text{ ft}^3 \text{ soil}) \cdot (1 \text{ ft}^3 \text{ } H_2O/62.4 \text{ lb } H_2O) \cdot (12 \text{ in/ft})$$

$$= 3.11 \text{ in } H_2O/\text{ft of soil (or 25.9 percent)}$$

Permanent wilting point (−15 atm tension):

$$M = \frac{W - d}{d} 100$$

$$8 = \frac{5 \text{ lb} - d}{d} 100$$

$$d = 4.63 \text{ lb}$$

Moisture lost during field capacity test = 5 − 4.63 = 0.37 lb H_2O

Permanent wilting point (volumetric basis)

$$= (0.37 \text{ lb } H_2O/0.06 \text{ ft}^3 \text{ soil}) \cdot (1 \text{ ft}^3 \text{ } H_2O/62.4 \text{ lb } H_2O) \cdot (12 \text{ in}/1 \text{ ft})$$

$$= 1.19 \text{ in } H_2O/\text{ft of soil (or 9.9 percent)}$$

Step 2. Estimate the available water capacity (AWC) by subtracting the permanent wilting point moisture content from the field capacity:

$$\text{Available water capacity (AWC)} = \text{FC} - \text{PWP}$$

$$= 3.11 \text{ in } H_2O/\text{ft of soil} - 1.19 \text{ in } H_2O/\text{ft of soil}$$

$$= 1.92 \text{ in } H_2O/\text{ft of soil (16.0 percent)}$$

6.9 Soil Water Properties

The various soil water properties may be defined using the soil element diagram presented in Fig. 6.12. It should be noted that the mass of soil gas (or air) is assumed to be negligible. Using Fig. 6.12, the important soil mass and volume relationships are described and summarized in Table 6.14, while Example 6.8 illustrates the utility of these relationships in the design and management of a biosolids land-application site

Example 6.8 To fully characterize a potential biosolids beneficial-use site, several baseline soil parameters should be evaluated, including moisture content, bulk density, and soil porosity. Assuming that a 200-ml sample of

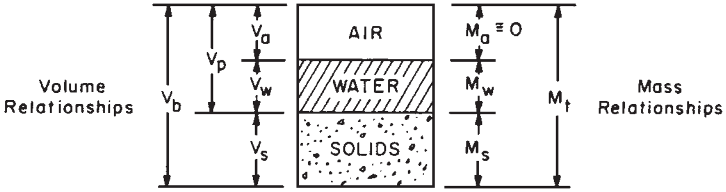


Figure 6.12 Soil element diagram. (*Adapted by permission from ref.* [*14*].)

TABLE 6.14 Summary of Soil Mass and Volume Relationships*

1. Water content (volume basis):

$$\theta_V = \frac{\text{volume of water}}{\text{total (bulk) volume of soil}} = \frac{V_w}{V_b} = \frac{V_w}{V_a + V_w + V_s}$$

2. Water content (mass basis): $$\theta_M = \frac{\text{mass of water}}{\text{mass of dry soil}} = \frac{M_w}{M_s}$$

3. Soil bulk density: $$\rho_b = \frac{\text{mass of dry soil}}{\text{total (bulk) soil volume}} = \frac{M_s}{V_b}$$

4. The volumetric and mass-based moisture contents are related by the following equation:

$$\theta_M = \theta_V\,(\rho_w/\rho_b)$$

5. Another important soil-moisture parameter is soil porosity N. This parameter is defined by the following equation:

$$N = \frac{\text{total pore volume}}{\text{total (bulk) soil volume}} = \frac{V_w + V_a}{V_s + V_w + V_a}$$

6. Soil porosity N may be related to the soil bulk density ρ_b by the following relationship:

$$N = 1 - (\rho_b/\rho_s)$$

where ρ_s is the particle density (for most agriculture soils $\rho_s = 2.65$ g/cm^3)

*Adapted from ref. [14].

soil taken from a potential site weighs 380 g, compute the moisture content on a mass and volume basis, bulk density, and soil porosity if, after drying the sample at 105°C (221°F) for 6 hours, the sample weighs 295 g. Assume that the soil particle density is 2.7 g/cm^3.

solution

Step 1. Compute the soil moisture content on a mass basis:

$$\theta_M = \frac{\text{mass of water}}{\text{mass of dry soil}} = \frac{380\text{ g} - 295\text{ g}}{295\text{ g}}$$

$$= 0.288 \text{ (or 28.8 percent)}$$

Step 2. Compute the soil moisture content on a volume basis:

$$\theta_V = \frac{\text{volume of water}}{\text{total (bulk) volume of soil}} = \frac{(380\text{ g} - 295\text{ g})\,/1\text{ g/ml}}{200\text{ ml}}$$

$$= 0.425 \text{ (or 42.5 percent)}$$

NOTE: Volumetric moisture content can be expressed as 0.425 ft of H_2O per foot of soil, 0.425 in of H_2O per inch of soil, etc.

Step 3. Compute the soil porosity N:

$$N = \frac{\text{total pore volume}}{\text{total (bulk) soil volume}} = \frac{V_w + V_a}{V_s + V_w + V_a}$$

Note that the bulk soil volume is the sum of the solid, liquid, and air phases. This relationship can then be employed to estimate the volume of air V_a from which the soil porosity N may be calculated.

$$V_b = V_s + V_w + V_a = 200 \text{ ml}$$

$$V_s = \frac{M_s}{\rho_s} = \frac{295 \text{ g}}{2.7 \text{ g/ml}} = 109.26 \text{ ml}$$

$$V_w = \frac{380 \text{ g} - 295 \text{ g}}{1 \text{ g/ml}} = 85 \text{ ml}$$

Therefore, the volume of the air phase $V_a = 200 - 85 - 109.26 = 5.74$ ml. Thus

$$N = \frac{V_w + V_a}{V_s + V_w + V_a} = \frac{85 \text{ ml} + 5.74 \text{ ml}}{200 \text{ ml}}$$

$$= 0.454 \text{ (or 45.4 percent)}$$

6.9.1 Soil moisture potential

Due to its impact on nutrient and trace element transport, a thorough understanding of the principles governing soil moisture potential is critical for the biosolids land-application engineer. The importance of soil moisture potential stems from the fact that moisture movement in plant-soil systems is influenced solely by the magnitude of the soil moisture potential difference that exists between locations.

Soil moisture potential is the sum of three individual potentials: gravitational, pressure, and osmotic potential [14]. *Gravitational potential* is the potential energy of soil moisture due to its elevation relative to some datum. The position of the datum is arbitrary in calculating the contribution of gravitational potential to total potential but is normally chosen so that the gravitational potential is positive.

The *pressure potential* is related to the energy associated with water pressure. The pressure potential can be either positive or negative depending on whether or not the soil location is at saturated or unsaturated moisture conditions. If the soil location is unsaturated (i.e., the moisture tension is less than atmospheric pressure), the pressure potential is negative. When the soil location is at saturated conditions (i.e., the moisture tension is greater than atmospheric pressure), the pressure potential is positive. Using this convention, the pressure potential at the water table is always zero.

Osmotic potential is a chemical potential resulting from the attraction of moisture to soil locations of higher soluble salt concentrations.

The osmotic potential becomes increasingly negative with increasing salt concentration in the soil solution.

The *total potential* T is the sum of the individual potentials and is described mathematically by Eq. (6.17). Example 6.9 illustrates the concept of soil moisture potential used in a typical biosolids land-application program.

$$T = z + p + p_{\mathrm{os}} \tag{6.17}$$

where T = total potential (ft, kPa, N/m^2)
z = gravitational potential (ft, kPa, N/m^2)
p = pressure potential (ft, kPa, N/m^2)
p_{os} = osmotic potential (ft, kPa, N/m^2)

The direction and rate of moisture movement from one point to another in soil is a function of the magnitude of the difference in total potential. Moisture always moves from a point of higher potential to a point of lower potential. In the absence of roots and when the osmotic potential is negligible, the total potential is equivalent to the piezometric or hydraulic potential H_p [Eq. (6.18)].

$$H_p = z + p \tag{6.18}$$

where H_p = piezometric potential (ft, kPa, N/m^2)
z = gravitational potential (ft, kPa, N/m^2)
p = pressure potential (ft, kPa, N/m^2)

Example 6.9 As shown in the figure on the next page, two points in a soil are to be monitored for their moisture potential. If the measured pressure potentials at point A and B were -30 and -45 cmH$_2$O, respectively, determine the following:

1. The total potential at points A and B (ignore osmotic potential).
2. The direction of moisture flow, i.e., is moisture moving from point A to B or the reverse?
3. Is point A or B at saturated moisture conditions?

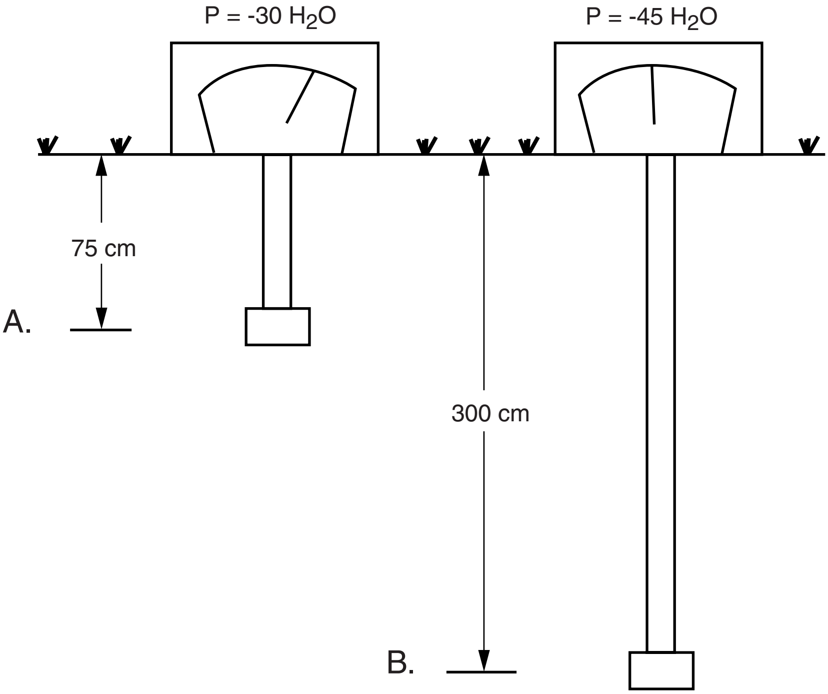
solution

Step 1. The total potential of points A and B can be estimated as follows:

$$\text{Total potential at point } A = z + p = 75\text{ cm} + (-30\text{ cm}) = 45\text{ cm}$$

$$\text{Total potential at point } B = z + p = 300\text{ cm} + (-45\text{ cm}) = 255\text{ cm}$$

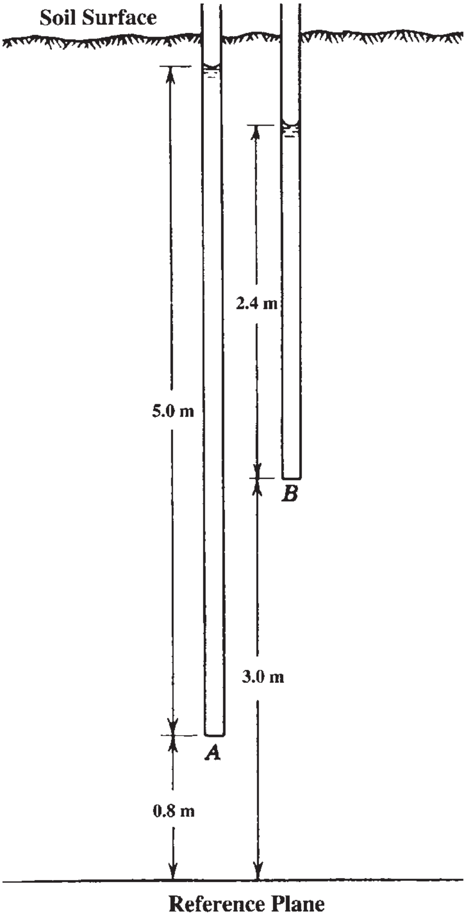
Step 2. Moisture always flows from a point of higher potential to a point of lower potential. Therefore, moisture is flowing from point B to point A.



Step 3. Since the pressure potential at both points is negative, both locations are at *unsaturated* conditions.

Under unsaturated moisture conditions, a tensiometer (or equivalent) is used to measure the hydraulic potential of a soil. Under saturated moisture conditions, a device called a *piezometer* can be used to measure the hydraulic potential and to evaluate the vertical direction and rate of moisture flow. A piezometer is essentially an open-ended tube placed at a given depth in a saturated soil. The difference in water heights between two piezometers placed at different depths reflects the variation in hydraulic potentials between the two depths. Example 6.10 illustrates the use of piezometers to estimate the direction of moisture flow.

Example 6.10 The Tubman County Water Reclamation Plant is considering a potential site for biosolids land application. To fully characterize the site, the regulatory agency has required that the publicly owned treatment works (POTW) evaluate the hydraulics of groundwater flow. If two piezometers are installed in the same vicinity but at different depths (as shown in figure on the next page), estimate the hydraulic potentials of the two points and the vertical direction of water flow. (Ignore osmotic potential.)



solution

Step 1. Estimate the hydraulic potential at points *A* and *B*.

$$h_A = z + p = 0.8 \text{ m} + 5.0 \text{ m} = 5.8 \text{ m}$$

$$h_B = z + p = 3.0 \text{ m} + 2.4 \text{ m} = 5.4 \text{ m}$$

Step 2. Since the hydraulic potential at point *A* is larger than at point *B*, moisture is flowing upward (i.e., from point *A* to point *B*).

NOTE: This is an example of artesian conditions.

6.9.2 Measurement of soil water potential

In all land-application systems, biosolids are applied to unsaturated soils. Soil water potential in unsaturated soils is measured in the

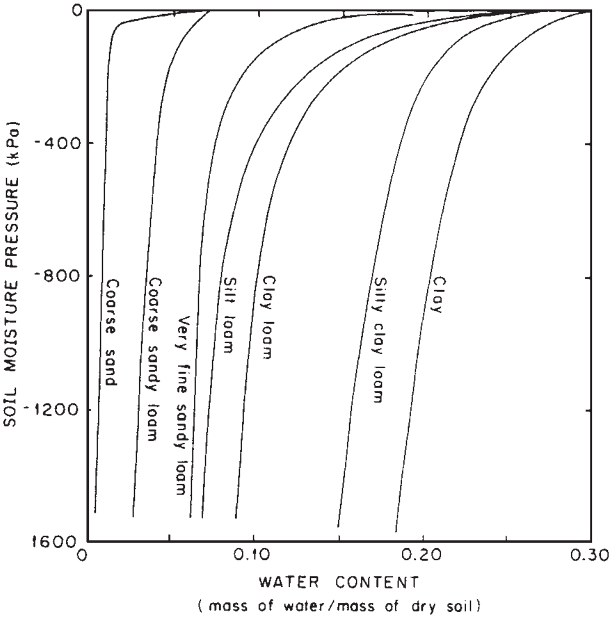


Figure 6.13 Typical soil water characteristic curve. (*Adapted by permission from ref.* [*14*].)

field by installation of tensiometers or resistance meters (e.g., gypsum blocks). Tensiometers measure the suction pressure (or tension) of the soil, whereas resistance meters read an electrical resistance that is correlated with tension by means of a calibration curve. detailed instructions for the installation and measurement of tension using tensiometers and resistance meters are provided in standard soil testing handbooks [6,14].

The relationship between soil water content and the soil water potential is termed the *soil water characteristic curve*. Knowledge of the soil water characteristic curve is important in the design and management of biosolids land-application systems because it is necessary to convert tension readings to a soil moisture content. Figure 6.13 provides an illustration of a soil water characteristic curve.

It should be noted that soil types made up of fine particles (e.g., clays) have a higher moisture content at the same tension as soils with coarser particles (e.g., sands).

6.9.3 Hydraulic conductivity

The velocity of moisture through a soil profile is proportional to the difference in total potential between two points. This difference in potential measured over some defined distance is termed the *hydraulic*

TABLE 6.15 Estimation of Hydraulic Gradient in a Soil-Water System*

Step 1. Assuming that the magnitude of the osmotic potential between two points is negligible, the total potential is equal to the piezometric potential:

$$\text{Piezometric potential at point } 1 = z_1 + p_1 = h_1$$

$$\text{Piezometric potential at point } 2 = z_2 + p_2 = h_2$$

Step 2. Using these definitions, the potential for moisture flow (i.e., hydraulic gradient) may be defined by the following equation:

$$\text{Hydraulic gradient} = \frac{dh}{ds} = \frac{h_1 - h_2}{s_1 - s_2} = \frac{(z_1 + p_1) - (z_2 + p_2)}{s_1 - s_2}$$

where $s_1 - s_2$ is the distance between points 1 and 2.

*Adapted from ref. [14].

gradient. Table 6.15 illustrates the approach for estimating the hydraulic gradient in a soil system.

Estimating the hydraulic gradient between two points in a soil is illustrated in Example 6.11. The proportionality constant that relates velocity to the hydraulic gradient is termed the *hydraulic conductivity K*. Equation (6.19), which is known as *Darcy's law,* illustrates the functional relationship between soil moisture velocity, hydraulic gradient, and the hydraulic conductivity.

Example 6.11 In order to increase the depth of the unsaturated zone at a biosolids land-application site, the biosolids engineer decides to install a well to lower the groundwater table. If the steady-state draw-down from the well is given in the figure on the next page, estimate the hydraulic gradient between points A and B. Assume that the following data apply: $s_1 = 25$ ft, $s_2 = 15$ ft, $h_A = 12.5$ ft, and $h_B = 11$ ft.

solution

Step 1. Estimate the piezometric potential at heights h_A and h_B. Note that since both points are at the water table, their pressure potentials are equal to zero. Therefore, the piezometric potential for points A and B are given as follows:

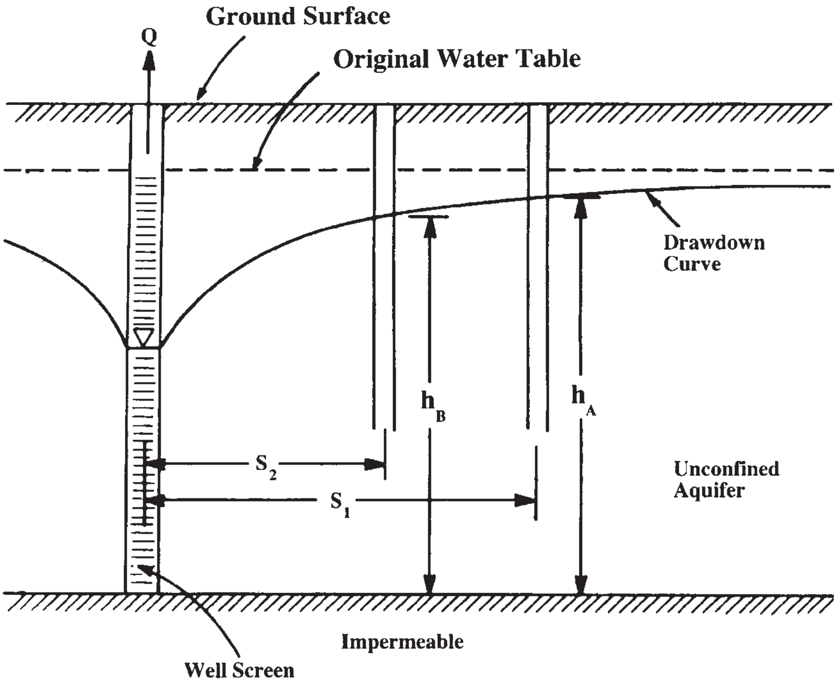
$$h_A = 12.5 \text{ m}$$

$$h_B = 11.0 \text{ m}$$

Step 2. The distance between points A and B is 10 m. Therefore, the hydraulic gradient is calculated as follows:

$$\frac{dh}{ds} = \frac{h_A - h_B}{s_1 - s_s} = \frac{12.5 \text{ m} - 11.0 \text{ m}}{25 \text{ m} - 15 \text{ m}}$$

$$= \frac{0.15 \text{ m}}{\text{m}}$$



$$\text{Velocity } (v) = -K\,\frac{dh}{ds} \tag{6.19}$$

where v = moisture velocity (cm/s, ft/s)
K = hydraulic conductivity (cm/s, ft/s)
$\frac{dh}{ds}$ = hydraulic gradient (dimensionless)

NOTE: The minus sign in Eq. (6.19) is required because water flow is always from a point of higher to lower total potential.

The volumetric moisture flow rate (ft^3/s, m^3/min, gal/day, etc.) in soil can be obtained by multiplying the moisture velocity by the cross-sectional area. The approach used to estimate moisture flow rate in soil is illustrated in Example 6.12.

Hydraulic conductivity K is a function of a number of parameters, including texture, temperature, porosity, and soil water content. It should be noted that Eq. (6.19) defines the Darcy velocity as the velocity perpendicular to a cross-sectional area between two points. Since water moves only through the pore space, the pore velocity will always be higher than the Darcy velocity. The pore velocity is defined by Eq. (6.20):

$$\text{Pore velocity } (v_p) = \frac{v}{N} \tag{6.20}$$

where v = Darcy velocity
N = soil porosity

In most biosolids land-treatment applications, it is the Darcy velocity rather than the pore velocity that is of concern. The pore velocity is an important parameter in the analysis of subsurface contaminant transport.

Example 6.12 A proposed biosolids land-application area is approximately 1000 m by 200 m in aerial size. The depth to the water table is 1.5 m. If the unsaturated soil has an average tension of −200 cmH_2O and the hydraulic conductivity of the soil is approximately 10^{-5} cm/s, estimate the vertical volumetric flow rate of groundwater through the land-application area. Assume that the depth to the water table remains constant and that the osmotic potential can be neglected.

solution

Step 1. Determine the hydraulic potential at the soil surface and at water table. Choose the water table as the datum.

$$h_p \text{ (soil surface)} = z + p = 1.5 \text{ m} + (-2.0 \text{ m}) = -0.5 \text{ m}$$

$$h_p \text{ (water table)} = z + p = 0 \text{ m} + 0 \text{ m} = 0.0 \text{ m}$$

Step 2. Estimate the hydraulic gradient:

$$\frac{dh}{ds} = \frac{-0.5 \text{ m}}{1.5 \text{ m}} = -0.33 \text{ m/m}$$

Step 3. Estimate the flow velocity:

$$\text{Velocity} = -K\frac{dh}{ds}$$

$$= -10^{-5} \text{ cm/s} \cdot -0.33 \text{ m/m} \cdot 1 \text{ m/100 cm} \cdot 3600\text{s/h}$$

$$= 1.188 \cdot 10^{-4} \text{ m/h}$$

Step 4. Estimate the volumetric flow rate through the biosolids land-application site in cubic meters per day:

$$\text{Volumetric flow rate } Q = \text{velocity} \cdot \text{area}$$

$$= 1.188 \cdot 10^{-4} \text{ m/h} \cdot (1000 \text{ m} \cdot 200 \text{ m}) \cdot 24 \text{ h/day}$$

$$= 570 \text{ m}^3\text{/day}$$

6.9.4 Measurement of hydraulic conductivity

Hydraulic conductivity typically is measured in the laboratory using undisturbed soil cores or in the field under saturated soil conditions.

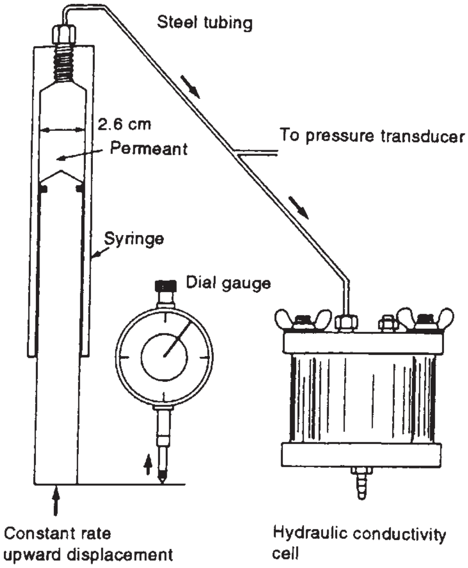


Figure 6.14 Laboratory permeameter for measurement of hydraulic conductivity. (*Adapted by permission from F. Fernandez and R. M. Quigly (1985), Canadian Geotechnical Journal, 22:205–214.*)

Values of saturated hydraulic conductivity vary over a wide range under field conditions. Figure 6.14 depicts a typical laboratory permeameter used to measure hydraulic conductivity.

Typical ranges of hydraulic conductivity as a function of soil texture are given in Table 6.16.

6.10 Infiltration

Infiltration is the process by which water passes through the soil surface and enters the subsoil. The units of infiltration are normally depth of water per unit time (e.g., inches per hour, feet per day, etc.). The rate at which infiltration can be maintained in a particular soil is an important parameter in the design of biosolids land-application systems. Infiltration will affect not only irrigation scheduling but also surface and subsurface drainage requirements. The permissible range in biosolids moisture content for a given biosolids land-application program is often governed by the infiltration characteristics of the soil. For example, application of liquid biosolids would be undesirable on soils with low or marginal infiltration rates [68].

In the agricultural literature, there are various infiltration terms with which the biosolids land-application engineer should become familiar. *Cumulative infiltration* refers to the depth of moisture that has penetrated the soil surface after a particular amount of time. Figure 6.15 depicts the differences in cumulative infiltration versus time for various soil types.

TABLE 6.16 Hydraulic Conductivity as a Function of Soil Texture*

Soil type	Hydraulic conductivity (m/day)
Coarse gravel	150
Coarse sand	45
Medium sand	12
Fine sand	2.5
Silt	0.08
Clay	0.0002

*Adapted from ref. [14].

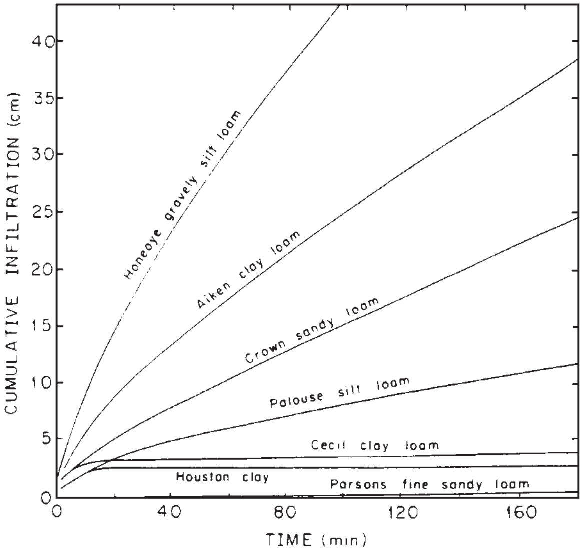


Figure 6.15 Cumulative infiltration versus time in typical agricultural soils. (*Adapted by permission from ref.* [*14*].)

Infiltration rate refers to the slope of the cumulative infiltration curve and represents the rate of change of infiltration. Instantaneous and average infiltration rates normally decrease with time. It should be noted that the rate of infiltration is a function of the initial soil moisture content, with drier soils demonstrating a significantly greater instantaneous infiltration rate.

Numerous approaches and empirical equations have been developed to represent observed infiltration data. An early equation used to model infiltration was the Kostiakov equation, which is still used in many agricultural crop production applications [Eq. (6.21)]:

$$i = c\,(t)^{\alpha} \tag{6.21}$$

where i = depth of infiltration (cm, in)
t = time of infiltration (cm, in)
c, α = empirical constants

Equation (6.21) has been found to fit field measurement infiltration data especially over relative short time periods (e.g., few hours). The constants used in this equation are determined through field tests. Other, more complex infiltration models have been developed for estimating infiltration. They include the Philip and the U.S. Soil Conservation Service infiltration equations [14]. For a description of these more sophisticated approaches, the reader is directed to refs. [32,50].

6.10.1 Measurement of infiltration

Under most conditions, sufficient information regarding infiltration rates can be achieved using either a single- or double-ring type of infiltrometer. Figure 6.16 presents a schematic diagram of a double-ring infiltrometer. The moisture level within the inner ring is monitored over time. The results are then plotted to create the cumulative infiltration curve.

The infiltration data also can be used to estimate the constants for one of the empirical infiltration models. An example of how the results from infiltrometer tests can be used to determine empirical constants for the Kostiakov equation is illustrated in Example 6.13.

The double-ring infiltrometer has been proposed as a more representative method of measuring vertical infiltration under steady-state conditions than the single-ring device. This is due to the fact that the influence of boundary conditions is minimized. In other words, use of the double-ring infiltrometer minimizes the horizontal movement of moisture from the inner ring so that only vertical infiltration is measured. It is important that the water levels within the two rings are maintained at equal depths to promote vertical infiltration for the inner ring.

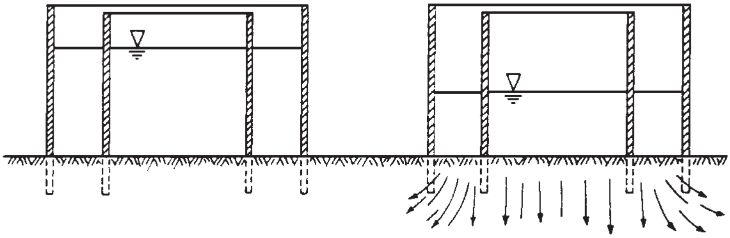


Figure 6.16 Schematic diagram of a double-ring infiltrometer: (*a*) at time equals zero; (*b*) at time equals *t*. (*Adapted by permission from ref.* [*14*].)

Example 6.13 A design engineer uses a double-ring infiltrometer to measure the infiltration rate of a potential biosolids land-application site. The following data were obtained from field tests:

Time (min)	Depth of infiltration (cm)
60	2.0
180	3.8

From these data, compute the coefficients of the Kostiakov equation and estimate the infiltration rate (cm/h) for this soil after 6 hours.

solution

Step 1. Rewrite Eq. (6.21) in logarithmic form:

$$\log i = \log c + \alpha \log t$$

Step 2. Using field measurements, develop independent equations for log c.

First measurement:

$$\log (2.0) = \log c + \alpha \log (60)$$

or

$$\log c = \log (2.0) - \alpha \log (60)$$

Second measurement:

$$\log (3.8) = \log c + \alpha \log (180)$$

or

$$\log c = \log (3.8) - \alpha \log (180)$$

Step 3. Equate the two expressions developed for log c in step 2 and solve for α:

$$\log (2.0) - \alpha \log (60) = \log (3.8) - \alpha \log (180)$$

$$\alpha = 0.585$$

Step 4. Substituting the value for α into any of the expressions in step 2 will allow you to solve for c:

$$\log (2.0) = \log c + \alpha \log (60)$$

$$\log (2.0) = \log c + 0.585 \log (60)$$

$$c = 0.183$$

Step 5. To estimate the infiltration rate (cm/h), take the derivative of Eq. (6.21):

$$\frac{di}{dt} = \alpha\, [c\, (t)^{\alpha - 1}]$$

Step 6. Substitute values of constants and a time of 360 minutes (6 hours):

$$\frac{di}{dt} = \alpha\, [c\, (t)^{\alpha - 1}] = 0.585\, [0.183\, (360)^{0.585 - 1}]$$

$$= 0.0092 \text{ cm/min, or } 0.55 \text{ cm/h}$$

6.10.2 Estimating soil erosion

As a result of extensive agricultural research, the rate of soil erosion from an agricultural field may be estimated using the universal soil-loss equation [Eq. (6.22)]. The factors that enter into Eq. (6.22) require the design engineer to estimate the amount of water that enters the soil, how much runs off, and the manner and rate of its removal. Each of the factors in Eq. (6.22) is briefly described in the following sections.

$$A = R \cdot K \cdot LS \cdot C \cdot P \tag{6.22}$$

where A = soil loss in metric tons per hectare per year
R = rainfall and runoff factor (see Fig. 6.17)
K = soil erodibility factor (see Table 6.17)
LS = slope length and gradient (i.e., topographic) factor (see Table 6.18)
C = cover and management factor
P = support practice factor (see Table 6.19)

The rainfall and runoff factor R measures the erosive force of rainfall and runoff. Of the two characteristic properties of rainfall, i.e., amount of total rainfall and its intensity, the latter is usually the more important with regard to soil erosion. The R factor is called the *rainfall erosion index* and is computed from the total kinetic energy of each storm plus the average rainfall during the 30-minute period of greatest storm intensity. An average of such indexes for several years is used in the universal soil-loss equation. Rainfall indexes computed for the United States are shown in Fig. 6.17.

The soil erodibility factor K indicates the inherent erodibility of a soil. The two most significant soil characteristics influencing erosion are (1) infiltration capacity and (2) structural stability. The soil erodibility or K factor normally varies from near zero to about 0.6. The value of K is low for soils into which water readily infiltrates, such as well-drained sandy soils (K values are typically less than 0.2). Soils with intermediate infiltration capacities and moderate soil structural stability generally have a K factor of 0.2 to 0.3, whereas the more easily eroded solids with low infiltration capacities will have a K factor of 0.3 or higher (Table 6.17).

The topographic factor LS reflects the influence of length and steepness of slope on soil erosion. The LS factor is the ratio of soil loss from the field in question to that of a unit plot with 9 percent slope, 22 m long, and continuously fallowed (i.e., natural vegetative cover, no tilling). The greater the steepness of slope, other conditions remaining constant, the greater is the erosion due to increased velocity of water flow. Table 6.18 provides LS values for selected slope characteristics.

The value of the cover and management factor C reflects the influence of cropping systems and management variables on soil loss.

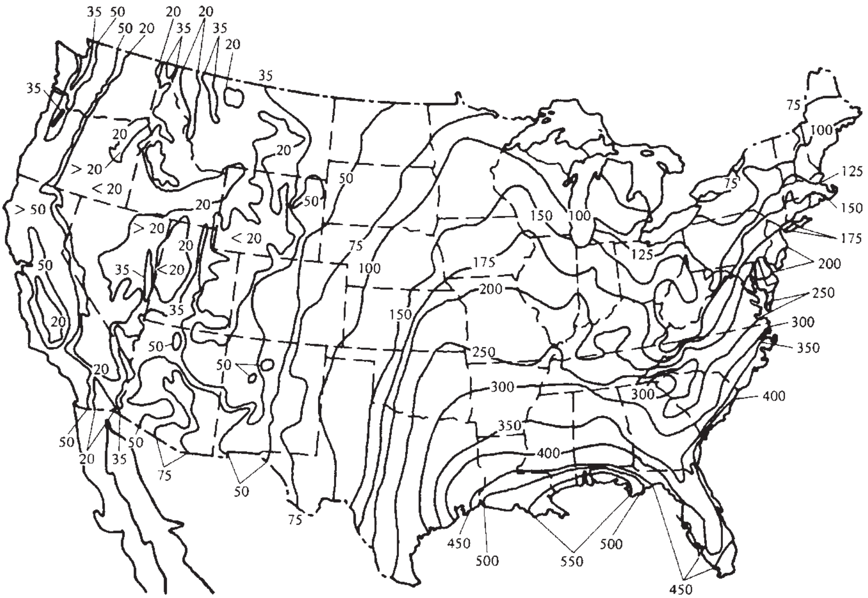


Figure 6.17 Average annual values of the rainfall erosion index in the United States. (*Adapted by permission from ref. [6].*)

TABLE 6.17 Computed Soil Erodibility Factors for Various Soils*

Soil type	*K* value
Dunkirk silt loam	0.69
Keene silt loam	0.48
Cecil sandy clay loam	0.36
Austin silt	0.29
Cecil sandy loam	0.23
Tifton loamy sand	0.10

*Adapted from ref. [6].

TABLE 6.18 Topographic Factor *LS* for Selected Combinations of Slope Length and Steepness*

	Slope length (m)				
Slope (%)	15.35	30.5	45.75	61.0	91.5
2	0.163	0.201	0.227	0.248	0.280
4	0.303	0.400	0.471	0.528	0.621
6	0.476	0.673	0.824	0.952	1.17
8	0.701	0.992	1.21	1.41	1.72
10	0.968	1.37	1.68	1.94	2.37
12	1.280	1.80	2.21	2.55	3.13

*Adapted from ref. [6].

Forests and grasses are effective in minimizing soil erosion. Small grains such as wheat and oats provide intermediate soil erosion protection, whereas row crops such as corn and soybeans offer relatively little cover during the early growth stage and thereby are characterized by minimal erosion protection. Fallowed areas, i. e., those areas on which no crop is grown and all the residues have been incorporated into the soil, are characterized by significant amounts of soil erosion. The C value for a specific location depends on a number of factors, including the type of crop or crops being grown, crop stage, tillage, and other management factors. Technically, the C value is the ratio of soil loss under the conditions found in the field in question to that which would occur under clean-tilled, continuous-fallow conditions. The C value will be low (e.g., less than 0.1) where large amounts of crop residues are on the land or in areas of dense forests, whereas the C value will approach 1.0 when there is little soil cover such as bare soil in the spring before a crop canopy develops. The actual C values for an area are available through the state offices of the U.S. Department of Agriculture Soil Conservation Service.

The support practice factor P reflects the benefits of contouring, strip cropping, and other supporting factors. It is the ratio of soil loss with a given support practice to the corresponding loss when crop culture is positioned directly up and down the slope. The P value for a given support practice is the ratio of soil loss using that practice to the loss that would occur with up and downhill culture. P values for contour tillage and strip cropping at different slope gradients are provided in Table 6.19. Note that P values increase with land slope and that they are low for strip cropping, illustrating the importance of this practice to erosion control.

Example 6.14 illustrates use of the universal soil-loss equation to estimate the expected soil loss from biosolids land-application sites.

Example 6.14 The Poole County Water Reclamation Plant is currently applying biosolids to 160 acres of agricultural land as a low-cost fertilizer and soil conditioner. The land is primarily pastureland with an average

TABLE 6.19 ***P* Values for Contour-Farmed Terrace Fields in Relation to Slope Gradient***

Slope	Contour factor	Stripcrop factor
1–2	0.6	0.30
3–8	0.50	0.25
9–12	0.60	0.30
13–16	0.70	0.35
17–20	0.80	0.40
21–25	0.90	0.45

*Adapted from ref. [6].

slope of 3 percent. If strip cropping is being employed to reduce soil erosion, estimate the annual soil loss from the site in tons per year given the following conditions:

Rain and runoff factor R: 140
Soil erodibility factor K: 0.29
Topographic factor LS: 0.25
Cover and management factor C: 0.20
Support practice factor P: 0.24

solution

Step 1. Use the universal soil-loss equation [Eq. (6.22)] to estimate the annual soil loss in pounds per acre per year:

$$\begin{aligned}(\text{metric tons/ha} \cdot \text{yr}) &= R \cdot K \cdot LS \cdot C \cdot P \\ &= 140 \cdot 0.29 \cdot 0.25 \cdot 0.20 \cdot 0.24 \\ &= 0.4872 \text{ metric tons/ha} \cdot \text{yr } (433.8 \text{ lb/acre} \cdot \text{yr})\end{aligned}$$

Step 2. For the entire biosolids application site, estimate the annual tons of soil loss expected from the site:

$$\begin{aligned}\text{Soil loss from application area (tons/yr)} \\ &= (433.8 \text{ lb/acre} \cdot \text{yr}) \cdot (160 \text{ acres}) \cdot (\text{ton}/2000 \text{ lb}) \\ &= 34.7 \text{ tons/yr}\end{aligned}$$

NOTE: If this soil loss is deemed excessive, the site must be managed to further reduce soil erosion.

6.11 Drainage Systems

Proper drainage design is critical for the successful management of biosolids land-application systems. The principal objective of a drainage system is to (1) protect water supplies, (2) increase crop production, and (3) sustain crop yields over long periods of time (Fig. 6.18). The presence of adequate levels of soil oxygen in the root zone soil is as necessary as moisture and nutrients for both seed germination and plant growth. The balance between soil moisture and oxygen is maintained through proper drainage design.

The lack of soil oxygen is a particular concern in humid areas with relatively high precipitation. In arid areas, agricultural drainage has the primary objective of salt (or salinity) management. An adequate salt balance must be maintained within the plant root zone to ensure adequate crop growth and to maintain soil porosity. With increasing salt concentrations in the crop root zone, plants expend more of their available energy on adjusting the salt concentration within their

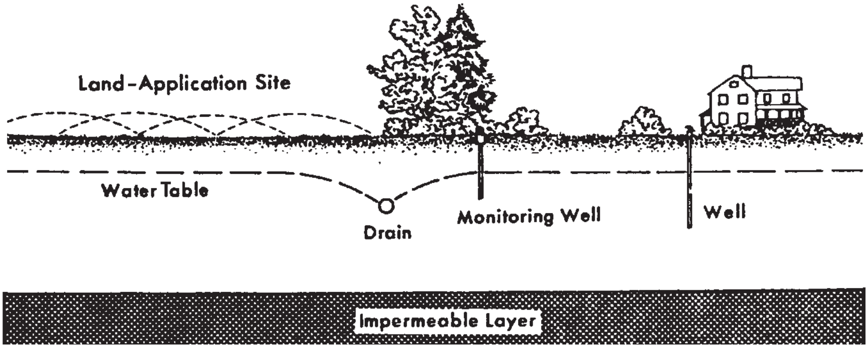


Figure 6.18 Schematic diagram illustrating the use of drains to protect a water supply. (*Adapted by permission from ref.* [*29*].)

tissue to obtain soil moisture (osmotic adjustment). As a consequence, there is less energy available for growth. Moreover, the groundwater in arid areas is often saline. If allowed to rise into the crop root zone, these salts can severely damage crop yield.

Despite their obvious importance, drainage systems are often overlooked in preliminary biosolids land-application design evaluations. The following sections attempt to highlight some of the important drainage issues likely to be encountered at a biosolids land-application site. This is not meant to be an exhaustive discussion on the topic of drainage. Many excellent drainage texts are available to the practicing engineer or engineering student [2,14,32,50].

6.11.1 Drainage terminology

Most drainage terminology has been developed from considering such aspects as the source of water to be removed, when and where the drains are to be built, and their function. Table 6.20 lists the various functional classifications of drains.

TABLE 6.20 Functional Classes of Field Drains*

Surface drains	Used to remove water from land surface.
Subsurface drains	Used to remove or control groundwater and then remove and/or control salts.
Open and pipe drains	Open drains are channels with an exposed water surface; pipe drains are buried pipe regardless of material size or shape.
Deferred drains	Drains that are provided after the site is in operation. The deferred construction of such drains is necessary because of the difficulty in anticipating actual drainage needs.

*Adapted from ref. [32].

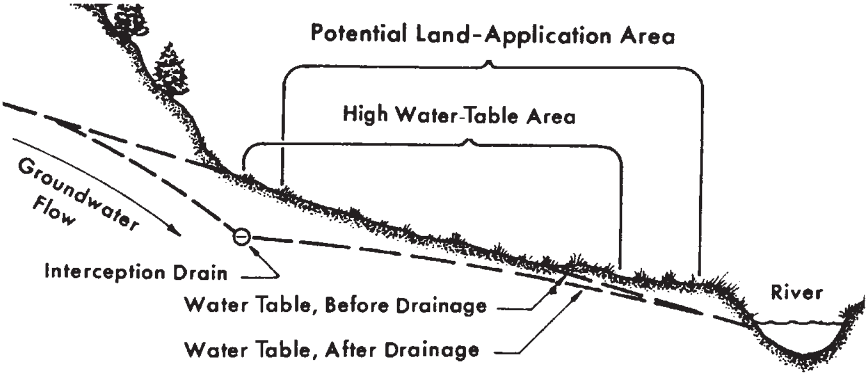


Figure 6.19 Schematic illustration of an interceptor drain. (*Adapted by permission from ref. [29].*)

In addition to their functional classifications, drains are categorized in terms of their technical designations. Table 6.21 lists the various technical designations of field drains.

The five types of technical drainage designations are illustrated in Fig. 6.20.

6.11.2 Drainage requirements at biosolids land-application sites

In general, large areas of land are used in biosolids land-application systems. These areas may contain low spots, including springs and seeps, that will require earthmoving or artificial drainage. Water that fills the soil pores not only displaces the soil air but also obstructs the escape of gases. The oxygen content in wet soils is limited not only because of the reduced oxygen concentration in water but also because of the slow rate of atmospheric oxygen diffusion through such soils. If left in a saturated state, these soils will become anaerobic within a few hours. Anaerobic soils are characterized by objectionable odors (primarily H_2S and volatile fatty acids) that can lead to public complaints and eventual termination of the biosolids land-application program. To avoid anaerobic conditions, the drainage system must have the capacity to remove moisture rapidly from the site.

Another important goal of a drainage system is the maintenance of a favorable salt balance in irrigated areas. In regions of high evapotranspiration and low natural precipitation, irrigation water must be used to sustain crop production. Both irrigation water and land-applied biosolids contribute to salt accumulation within these soils. To mitigate the adverse impact of salt accumulation, the engineer must design an effective irrigation and drainage plan to continuously remove salts.

TABLE 6.21 Technical Designations of Field Drains*

Relief drains	Used to lower groundwater over large flat areas. These drains are normally used when the hydraulic gradient does not provide sufficient water movement.
Interceptor drains	Used to cut off or intercept groundwater that is moving down slope. These drains are often used to dry up existing seeps and springs (see Fig. 6.19).
Collector drains	Used to receive water from both subsurface relief or interceptor drains and from surface drains.
Suboutlet drains	Used to convey water from collector drains to the outlet drain. They are normally located in topographic low areas.
Outlet drains	Used to convey collected water away from the drained area. The outlet drain is normally a natural channel in the topographic low area for the region to be drained.

*Adapted from refs. [32,50].

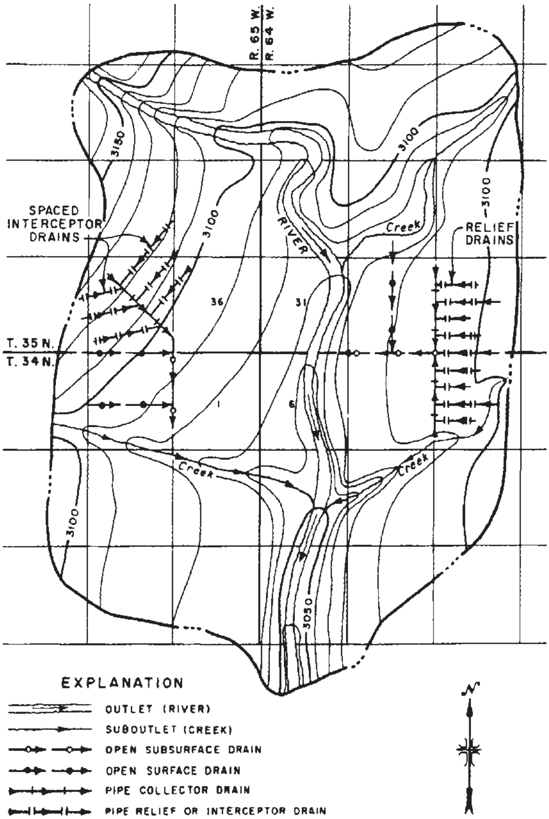


Figure 6.20 Illustration of technical drain designations with drain outlet to a river [50].

Consideration of a drainage system in the site-planning phase of a biosolids land-application design increases the number of potential siting alternatives that can meet the performance criteria. An example of such a site would be one in which all other factors are favorable except a high seasonal water table. The cost of drainage improvements should be compared with the cost of transportation of biosolids to a more distant site not requiring the same degree of drainage development.

6.11.3 Drainage investigation

In evaluating a potential site for its suitability for biosolids land application, the drainage needs are identified systematically through a drainage investigation. A drainage investigation is conducted to identify the total drainage requirements at a site. At a minimum, the drainage investigation must determine (1) the availability of a drainage outlet, (2) the availability of sufficient land slope, (3) the type of soil that must be excavated, and (4) the presence of drainage obstructions (natural or artificial).

To identify the total drainage requirements at a sits, the drainage investigation also should obtain information concerning the depth of the water table, soil texture and structure, irrigation water quality, land topography, and other factors that influence drainage system design. Following the drainage investigation, a site drainage plan is developed that includes (1) the number and type of drains, (2) drainage flow rates, (3) the length and direction of drainage flow, and (4) identification of an adequate outlet.

At biosolids land-application sites, the primary objectives for the drainage system include (1) surface runoff control, (2) groundwater control, and (3) salinity management. Each of these are discussed in the following sections.

6.11.4 Surface runoff control

Regardless of the region in which the biosolids land-application site is located (i.e., humid or arid), overland moisture flow must be controlled to ensure that the hydraulic loading to the site is not excessive. Excessive hydraulic loading at the site can lead to anaerobic conditions, whereas large overland flow velocities can result in significant soil erosion. To properly design drainage systems for control of overland flow, a means of estimating the volume of surface runoff must be available.

Surface runoff occurs when precipitation and/or the irrigation water application rate exceeds the soil infiltration rate. When this occurs, water begins to collect on the soil surface. When the surface detention requirements are exceeded, overland flow of moisture occurs. Surface runoff from natural precipitation or irrigation varies with many factors, including soil texture, type of irrigation system, land slope, length of irrigation run, and irrigation efficiency. For biosolids application sites, eliminating uncontrolled surface runoff is essential.

In most circumstances, the structures and facilities constructed to control surface runoff are designed for a particular storm of a certain intensity and duration. The capacity of the drains must be adequate to handle the anticipated runoff. The challenge to the engineer is to choose a reasonable design storm. The first step is to tabulate all available data on rainfall. From these data, the most intense storms are normally selected as the design basis.

Both the characteristics of the rainfall and area characteristics (e.g., topography, geology, surface vegetation etc.) contribute to the amount of surface runoff. The most practical approach to estimating surface drainage requirements from surface runoff is by examining the flow data from existing channels and culverts in the drainage area. When existing drainage data are unavailable, empirical models are often employed for drainage design. For small watersheds, the rational formula is widely used for estimating the peak surface runoff rates [Eq. (6.23)].

$$Q = CiA \tag{6.23}$$

where Q = design peak runoff rate (ft^3/s)
C = runoff coefficient (Table 6.22)
i = rainfall intensity (in/h) for the design recurrence interval and for the duration equal to the time of concentration (TOC) of the watershed
A = watershed area (acres)

The time of concentration (TOC) of the watershed is the time required for water to flow from the most remote part of the area to the outlet. It is assumed that when the duration of the storm equals the time of concentration, all parts of the watershed are contributing simultaneously to the drainage discharge. The runoff coefficients to be used in the rational formula are given in Table 6.22. It should be noted that the drainage system must have the capacity of removing moisture at a rate equal to the rate estimated using the rational formula. Insufficient capacity will result in the ponding of moisture and excessive hydraulic loads at the biosolids land-application site.

The rainfall intensity i is determined using the design storm curve and the time of concentration (TOC) for the drainage area (Fig. 6.21). For example, given a design storm of 2 in/h and a time of concentration of 20 min, the rainfall intensity i is estimated by first going to the x axis (Fig. 6.21) and finding the 20-min storm duration. Next, moving

TABLE 6.22 Surface Runoff Factors*

Type of surface	Value of C
Pervious soils, comparatively flat, heavy turf	0.1–0.3
Pervious soils, comparatively sharp surface slope, sparse turf	0.3–0.7
Wooded areas (depending on surface slope and soil cover)	0.02–0.3
Rocky, barren soils (depending on surface slope, soil cover, etc.)	0.3–0.8
Asphalt or concrete pavement	0.95–1.0

*Adapted from ref. [32].

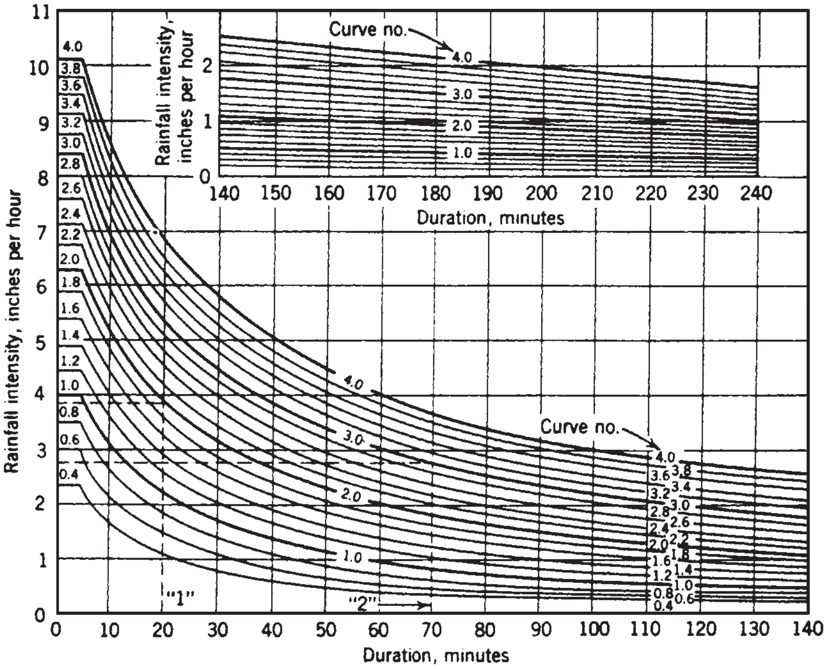


Figure 6.21 Standard rainfall intensity-duration curve. (*Used by permission from ref. [32].*)

vertically, the design storm of 2 in/h is intersected from the family of curves. From the point of intersection, the rainfall intensity is found by moving horizontally to the left until the y axis is intersected. In this case, a rainfall intensity of 3.9 in/h is found.

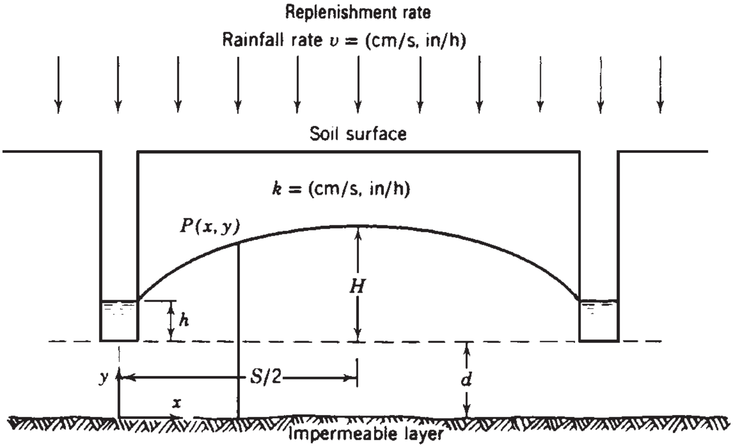
6.11.5 Groundwater control

In areas of high precipitation, groundwater levels must be controlled to provide sufficient retention time for biosolids constituents to undergo chemical and/or biological transformation within the unsaturated soil zone. In arid areas, groundwater salinity is a concern. The rate of groundwater recharge through irrigation practices must be controlled to prevent saline groundwater from reaching the crop root zone.

To properly control groundwater levels, the necessary spacing between the drains must be estimated. The simplest approach to estimate the required drain spacing is obtained using the Hooghoudt equation [Eq. (6.24)]:

$$S^2 = \frac{4KH}{v}(2d + H) \tag{6.24}$$

where S = distance between drains (ft)
K = hydraulic conductivity (ft/h)
H = maximum height of water table between drains (ft)
v = rate of precipitation and/or irrigation (ft/h)
d = depth between drain and impermeable barrier or aquiclude (ft)



NOTE: Variables in Eq. (6.24) are defined by the above figure, obtained by permission from ref. [32].

The Hooghoudt equation allows the determination of drain spacing as a function of the maximum height of the groundwater table, precipitation (or irrigation) rate, soil permeability, and depth of drain. The Hooghoudt equation assumes that the water table will rise until the flow into the drains is just equal to the amount of rain or irrigation water infiltrating through the soil surface. Other factors such as the rate of plant use of water, deep seepage, soil stratification, etc. are ignored. Table 6.23 summarizes the assumptions used in the development of the Hooghoudt approach.

TABLE 6.23 Summary of Assumptions Used in Development of the Hooghoudt Expression*

Soil is homogeneous and of hydraulic conductivity K.
Drains are evenly spaced a distance S apart.
Hydraulic gradient at any point is equal to the slope of the water table above the point.
Darcy's law is valid for flow of water though soils.
An impermeable layer (i.e., aquiclude) underlies the drain at a depth d.
Rate of replenishment of the water table is v (equal to rainfall or irrigation intensity).

*Adapted from ref. [32].

It is important to recognize that the hydraulic conductivity used in the Hooghoudt equation will be the average hydraulic conductivity for the soil. The identification of an impermeable layer (i.e., aquiclude) is often difficult from field data. Under normal circumstances, if the layer has a hydraulic conductivity of one-tenth or less than the overlying soil, then it can be considered to be an aquiclude.

6.11.6 Salinity management

The application of biosolids to land may result in the addition of a significant amount of salts. In humid areas, there is less concern with salt accumulation because natural precipitation often can wash salts below the crop root zone. However, in arid climates where irrigation water is used, salts present in biosolids can have an adverse affect on crop yield. The presence of excessive amounts of salt can damage crop growth in three ways: (1) osmotic stress caused by elevated levels of total dissolved solids (TDS) in the soil water, (2) limited oxygen availability due to soil particle dispersion caused by high sodium levels, and (3) specific ion toxicity.

The salt content (or salinity) of soil is determined by measuring the electrical conductivity of an aqueous extract of soil (EC_e, measured in mmhos/cm or dS/m). The electrical conductivity is used as a surrogate for total dissolved solids (TDS). For design purposes, the values of EC_e and TDS are related by Eq. (6.25):

$$\text{TDS (mg/liter)} = EC_e \text{ (mmhos/cm)} \cdot 640 \qquad (6.25)$$

Since, in many regions, effluent from the wastewater treatment plant is used to irrigate the biosolids land-application site, irrigation water may contain substantial quantities of salts (0.1–4 metric tons/1000 m^3). Although plants absorbe some of the applied salt, most salt must be removed from the soil by adding irrigation water in excess of the crop needs (e.g., evapotranspiration). If more water is applied than the plants use, the excess water will percolate below the root zone carrying with it a portion of the accumulated salts. After repeated irrigation events, the soil salinity will reach an equilibrium value that will be a function of the moisture leaching fraction (LF). The engineer (or irrigation/drainage specialist) must determine an appropriate moisture leaching fraction to ensure an acceptable crop yield at a reasonable cost. After successive irrigation events, the steady-state salt-balance equation is satisfied within the crop root zone [Eq. (6.26)]:

$$\text{Salt input (irrigation water)} = \text{salt output (drainage water)} \tag{6.26}$$

It should be noted that Eq. (6.26) assumes that the salt input from biosolids is small relative to the salt input from the continuous application of irrigation water (wastewater treatment plant effluent). In theory, this assumption is correct, since biosolids normally are added to a site no more than a few times a year. However, in cases where biosolids are added more frequently, the salt content in the biosolids should be included as a continuous input loading.

The amount of salt in the irrigation and drainage water may be expressed in terms of electrical conductivity using Eqs. (6.27) and (6.28), respectively:

$$\text{Salt in irrigation water} = EC_{iw}D_{iw} \tag{6.27}$$

$$\text{Salt in drainage (i.e. leached) water} = EC_{dw}D_{dw} \tag{6.28}$$

where EC_{iw} = electrical conductivity of the irrigation water (mmhos/cm)
D_{iw} = depth of irrigation water added (in)
EC_{dw} = electrical conductivity of drainage water (mmhos/cm)
D_{dw} = depth of drainage water added (in)

It should be noted that the electrical conductivity of the drainage water includes the salts contributed by irrigation water, the soil, and the applied biosolids. To maintain electrical neutrality, the steady-state salt balance [i. e., Eq. (6.26)] may be rewritten as Eq. (6.29):

$$EC_{iw}D_{iw} = EC_{dw}D_{dw} \tag{6.29}$$

The *leaching fraction* (LF) is defined as the fraction of applied irrigation water that passes through the entire rooting depth (i.e., the amount of moisture added in excess of crop needs). To maintain a favorable salt balance in the root zone, an equal or greater amount of salt must be leached from the soil by the drainage water than is introduced into the soil by biosolids and/or irrigation water. It should be noted that the increase in salt concentration in the drainage water over the concentration in irrigation water is a consequence of the consumptive use of water by the crop and moisture evaporation. This term is collectively called *evapotranspiration* ET_c. The amount of irrigation water required to maintain suitable crop growth will equal the sum of the consumptive use (i.e., ET_c) and drainage water [Eq. (6.30)]:

$$D_{iw} = ET_c + D_{dw} \tag{6.30}$$

where D_{iw} = depth of irrigation water added (in)
ET_c = crop evapotranspiration (in)
D_{dw} = depth of drainage water (in)

Using the definitions given in Eqs. (6.29) and (6.30), the leaching fraction (LF) may be defined by Eq. (6.31):

$$LF = \frac{D_{dw}}{D_{iw}} = \frac{D_{iw} - ET_c}{D_{iw}} = 1 - \frac{ET_c}{D_{iw}} \tag{6.31}$$

where LF = leaching fraction (%)
D_{iw} = depth of irrigation water added (in)
D_{dw} = depth of drainage water removed (in)
ET_c = crop evapotranspiration (in)

A high leaching fraction results in more salts being removed from the crop root zone. At steady state, the leaching fraction also may be defined in terms of the electrical conductivity of the irrigation and drainage water [Eq. (6.32)]:

$$LF = \frac{EC_{iw}}{EC_{dw}} \tag{6.32}$$

where LF = leaching fraction (%)
EC_{iw} = electrical conductivity of the irrigation water (mmhos/cm)
EC_{dw} = electrical conductivity of drainage water (mmhos/cm)

Example 6.15 illustrates the procedure of using the leaching fraction to manage salt accumulation at biosolids land-application sites.

Example 6.15 At the Carter County biosolids land-application site, sugar cane was determined to be the most profitable crop that could be grown. The crop was irrigated with wastewater effluent whose electrical conductivity was measured to be 0.75 mmhos/cm. Because of the high salt content in the applied biosolids, the crop was irrigated to achieve a leaching fraction of 0.25 (i.e., only 75 percent of the applied water is used to meet evapotranspiration demand). For this situation, determine the following: (1) the salinity of the leaching fraction and (2) the appropriate leaching fraction to maintain crop yield. Assume that sugar cane is known to suffer significant loss in yield when the TDS of the soil water exceeds 4000 mg/liter.

solution

Step 1. The salinity of the drainage water below the root zone can be estimated by rearranging Eq. (6.32):

$$EC_{dw} = EC_{iw}/LF = (0.75\text{ mmhos})/\text{cm}/0.25 = 3.0\text{ mmhos/cm}$$

Step 2. The total dissolved solids in the drainage water may be estimated using Eq. (6.32):

$$\text{TDS (mg/liter)} = EC_{dw} \cdot 640 = 3.0\text{ mmhos/cm} \cdot 640 = 1920\text{ mg/liter}$$

Step 3. The appropriate leaching fraction required to prevent loss in crop yield may be estimated using Eq. (6.32):

$$LF = \frac{EC_{iw}}{EC_{dw}} = \frac{0.75\text{ mmhos/cm} \cdot [(640\text{ mg/liter / mmhos/cm})]}{4000\text{ mg/liter}} = 0.12$$

Therefore, to prevent loss in yield, 12 percent of the applied water will be needed to carry salts below the crop root zone, and 88 percent will be consumed by evapotranspiration.

In normal agricultural operations, the electrical conductivity of the drainage water EC_{dw} is not measured because it does not accurately reflect the salt concentration to which the crop roots are exposed. In most cases, the electrical conductivity of an aqueous extract of the root zone soil is measured (EC_e). Substitution of EC_e for EC_{dw} provides a conservative approach to estimating the required leaching fraction for salinity management decisions. Many studies have been conducted that have correlated the reduction of various crop yields with EC_e (Tables 6.24 and 6.25).

It should be noted that forage crops are most resistant to high levels of salinity, followed by field and vegetable crops. Fruit crops normally are the most sensitive to high levels of soil salinity. For proper management of agricultural operations, the engineer must compare the cost of providing a larger leaching fraction (i.e., increased irrigation flow rate) versus the loss in economic revenue due to reduction in crop yield caused by excessive levels of salt.

At a biosolids land-application site where crop sale is a major source of revenue, a suitable leaching fraction (i.e., volume of irriga-

TABLE 6.24 Salt Tolerance Levels for Various Reductions in Crop Yield*

	100%		90%		75%		50%	
Crop	EC_e	EC_{iw}	EC_e	EC_{iw}	EC_e	EC_{iw}	EC_e	EC_{iw}
Field crops								
Barley	8.0	5.3	10.0	6.7	13.0	8.7	18.0	12. 0
Beans (field)	1.0	0.7	1.5	1.0	2.3	1.5	3.6	2.4
Corn	1.7	1.1	2.5	1.7	3.8	2.5	5.9	3.9
Cotton	7.7	5.1	9.6	6.4	13.0	8.4	17.0	12.0
Cowpeas	1.3	0.9	2.0	1.3	3.1	2.1	4.9	3.2
Rice (paddy)	3.0	2.0	3.8	2.6	5.1	3.4	7.2	4.8
Safflower	5.3	3.5	6.2	4.1	7.6	5.0	9.9	6.6
Sorghum	4.0	2.7	5.1	3.4	7.2	4.8	11.0	7.2
Soybean	5.0	3.3	5.5	3.7	6.2	4.2	7.5	5.0
Sugar beet	7.0	4.7	8.7	5.8	11.0	7.5	15.0	10.0
Wheat	6.0	4.0	7.4	4.9	9.5	6.4	13.0	8.7
Forage crops								
Alfalfa	2.0	1.3	3.4	2.2	5.4	3.6	8.8	5.9
Barley hay	6.0	4.0	7.4	4.9	9.5	6.3	13.0	8.7
Bermuda grass	6.9	4.6	8.5	5.7	10.8	7.2	14.7	9.8
Clover, berseem	1.5	1.0	3.2	2.1	5.9	3.9	10.3	6.8
Corn (forage)	1.8	1.2	3.2	2.1	5.2	3.5	8.6	5.7
Orchard grass	1.5	1.0	3.1	2.1	5.5	3.7	9.6	6.4
Perennial rye	5.6	3.7	6.9	4.6	8.9	5.9	12.2	8.1
Soudan grass	2.8	1.9	5.1	3.4	8.6	5.7	13.3	8.9
Tall fescue	3.9	2.6	5.8	3.9	8.6	5.7	13.3	8.9
Tall wheat grass	7.5	5.0	9.9	6.6	13.3	9.0	19.4	13.0
Vegetable crops								
Beans	1.0	0.7	1.5	1.0	2.3	1.5	3.6	2.4
Beets	4.0	2.7	5.1	3.4	6.8	4.5	9.6	6.4
Broccoli	2.8	1.9	3.9	2.6	5.5	3.7	8.2	5.5
Cabbage	1.8	1.2	2.8	1.9	4.4	2.9	7.0	4.6
Cantaloupe	2.2	1.5	3.6	2.4	5.7	3.8	9.1	6. 1
Carrot	1.0	0.7	1.7	1.1	2.8	1.9	4.6	3.1
Cucumber	2.5	1.7	3.3	2.2	4.4	2.9	6.3	4.2
Lettuce	1.3	0.9	2.1	1.4	3.2	2.1	5.2	3.4
Onion	1.2	0.8	1.8	1.2	2.8	1.8	4.3	2.9
Pepper	1.5	1.0	2.2	1.5	3.3	2.2	5.1	3.4
Potato	1.7	1.1	2.5	1.7	3.8	2.5	5.9	3.9
Radish	1.2	0.8	2.0	1.3	3.1	2.1	5.0	3.4
Spinach	2.0	1.3	3.3	2.2	5.3	3.5	8.6	5.7
Sweet corn	1.7	1.1	2.5	1.7	3.8	2.5	5.9	3.9
Sweet potato	1.5	1.0	2.4	1.6	3.8	2.5	6.0	4.0
Tomato	2.5	1.7	3.5	2.3	5.0	3.4	7.6	5.0

*Adapted from ref. [14].

tion water required) may be estimated by knowing the relationship between crop yield and EC_e. Assuming that drainage is not limiting (i.e., rate of field drainage below the crop root zone is equal to or greater than the rate of infiltration), the depth of irrigation water can be expressed as the product of the average infiltration rate I_{ave} and the

TABLE 6.25 Salt Tolerance Levels for Various Reductions in Crop Yield*

	100%		90%		75%		50%	
Crop	EC_e	EC_{iw}	EC_e	EC_{iw}	EC_e	EC_{iw}	EC_e	EC_{iw}
Fruit crops								
Almond	1.5	1.0	2.0	1.4	2.8	1.9	4.1	2.7
Apple, pear	1.7	1.0	2.3	1.6	3.3	2.2	4.8	3.2
Apricot	1.6	1.1	2.0	1.3	2.6	1.8	3.7	2.5
Avocado	1.3	0.9	1.8	1.2	2.5	1.7	3.7	2.4
Date palm	4.0	2.7	6.8	4.5	10.9	7.3	17.9	12.0
Grape	1.5	1.0	2.5	1.7	4.1	2.7	6.7	4.5
Grapefruit	1.8	1.2	2.4	1.6	3.4	2.2	4.9	3. 3
Lemon	1.7	1.1	2.3	1.6	3.3	2.2	4.8	3.2
Orange	1.7	1.1	2.3	1.6	3.2	2.2	4.8	3.2
Peach	1.7	1.1	2.2	1.4	2.9	1.9	4.1	2.7
Plum	1.5	1.0	2.1	1.4	2.9	1.9	4.3	2.8
Strawberry	1.0	0.7	1.3	0.9	1.8	1.2	2.5	1.7
Walnut	1.7	1.1	2.3	1.6	3.3	2.2	4.8	3.2

*Adapted from ref. [14].

time of irrigation t_i. The leaching fraction (LF) can then be estimated using Eq. (6.33):

$$LF = 1 - \frac{ET_c}{D_{iw}} = 1 - \frac{ET_c}{I_{\text{ave}}\, t_i} \tag{6.33}$$

where LF = leaching fraction (%)
D_{iw} = depth of irrigation water added (in)
ET_c = crop evapotranspiration (in)
I_{ave} = average infiltration rate (in/h)
t_i = irrigation time (h)

NOTE: It is assumed in Eq. (6.33) that irrigation water is applied at a rate equal to the average infiltration rate.

Example 6.16 illustrates the procedure for estimating the leaching fraction and volume of irrigation water required at a biosolids land-application site from the relationship between salt concentration and reduction in crop yield.

Example 6.16 The following field data were obtained for the Poole County biosolids land-application site. Estimate the leaching fraction, time of irrigation, and volume of irrigation water used per irrigation event.

EC_{iw}: 1.2 mmhos/cm
Crop: alfalfa
ET_c: 10 mm/day

Allowable crop yield reduction: 10 percent
Soil infiltration rate: 1.2 cm/h
Irrigation frequency: 10 days

solution

Step 1. From Table 6.24, soil having an EC_e of 3.4 mmhos/cm will result in a 10 percent reduction in alfalfa yield. Assuming that EC_{dw} is equal to EC_e, the leaching fraction can be estimated using Eq. (6.32):

$$LF = \frac{EC_{iw}}{EC_{dw}} = \frac{1.2 \text{ mmhos/cm}}{3.4 \text{ mmhos/cm}} = 0.35$$

Step 2. The moisture required to meet the crops consumptive need (EC_i) is estimated as follows:

ETc 5 10 mm/day?10 days 5 100 mm 5 10 cm.

Step 3. The time of irrigation can then be estimated by applying Eq. (6.33):

$$LF = 1 - \frac{ET_c}{I_{\text{ave}}\, t_i}$$

$$0.35 = 1 - \frac{10 \text{ cm}}{1.2 \text{ cm/h} \cdot t_i}$$

$$t_i = 12.9 \text{ h}$$

Therefore, 12.9 hours of irrigation at an application rate of 1.2 cm/h is required for each 10-day irrigation cycle.

Step 4. The volume of irrigation water required per irrigation event (10 days) is estimated as follows:

$$\begin{aligned}\text{Volume of irrigation water (cm)} &= 12.9 \text{ hours} \cdot 1.2 \text{ cm/h} \\ &= 15.5 \text{ cm (6.1 in) every 10 days}\end{aligned}$$

NOTE: The volume of irrigation water is sufficient to meet both the crop and salt-management demands.

By rearranging Eq. (6.33) and substitution of electrical conductivities, the depth of irrigation water (i.e., D_{iw}) may be estimated as illustrated by Eq. (6.34):

$$D_{iw} = \frac{ET_c}{1 - LF} = \frac{EC_{dw}}{EC_{dw} - EC_{iw}}\, ET_c \qquad (6.34)$$

where D_{iw} = depth of irrigation water added/required (in)
ET_c = crop evapotranspiration (in)
EC_{iw} = electrical conductivity of the irrigation water (mmhos/cm)
EC_{dw} = electrical conductivity of drainage water (mmhos/cm)
LF = leaching fraction (%)

When irrigation is used, percolating irrigation water may result in significant groundwater mounding below the biosolids land-application site. In most cases, groundwater mounding is not a concern because the maximum rise in the water table will be sufficiently below the crop root zone. However, in areas of seasonal shallow groundwater levels or in areas where shallow hardpans (e.g., clay lenses) are present, groundwater mounding can occur, resulting in saturation of the crop root zone. If the groundwater is also saline, damage to the crop can be substantial. It is critical for the drainage engineer to be aware of the soil horizons and the impacts of the irrigation system on the groundwater table. Example 6.17 illustrates the procedure for estimating the change in groundwater level due to irrigation practices.

Example 6.17 Estimate the rise in the groundwater table in an irrigated area if the electrical conductivity of the irrigation water (EC_{iw}) is 1 mmhos/cm, the electrical conductivity of the drainage water (EC_{dw}) is 8 mmhos/cm, and the crop evapotranspiration is 0.30 in/day.

solution

Step 1. Using Eq. (6.34), the required depth of the irrigation water to manage the salt accumulation can be estimated as follows:

$$D_{iw} = \frac{EC_{dw}}{EC_{dw} - EC_{iw}} ET_c$$

$$= \frac{8}{8 - 1} \cdot 0.30 = 0.34 \text{ in/day}$$

Therefore, 0.34 in/day of irrigation water is required to meet both the crop needs and salt-management demands.

Step 2. Subtracting the consumptive use of water from the volume of applied irrigation water provides an estimate of the rate of groundwater recharge:

$$\text{Rate of groundwater recharge} = D_{iw} - ET_c$$

$$= 0.34 \text{ in/day} - 0.30 \text{ in/day} = 0.04 \text{ in/day}$$

Therefore, to maintain a constant groundwater table, the drainage system must be designed to remove at least 0.04 in/day of saline ($EC = 8$ mmhos/cm) drainage water.

It should be noted that the total moisture in a root zone is not readily available to plants because of root distribution and the pattern of water use from the root zone. The water that is available to crops in a given root zone is called the *total readily available moisture* (TRAM). It is a physical characteristic of a given soil profile, specific crop root zone, and moisture extraction pattern.

In arid areas, the irrigation schedules will vary from year to year because of variations in crops, acreage, rainfall, solar radiation, and time of planting. Once the total readily available moisture, root zone depth, and crops have been determined, the irrigation scheduling process is a simple bookkeeping exercise [14].

6.11.7 Practical considerations in drainage design

Once the drainage investigation has been completed, the drainage requirements are divided into two principal categories: offsite and onsite drainage. Although the drainage principles are the same, the drainage objectives of these two categories are quite different.

6.11.7.1 Offsite drainage. To properly design and manage the biosolids land-application site, surface flows outside the immediate application area must be controlled. The specific objectives of offsite drainage system include the following:

1. Protecting the land-application area from erosive flows
2. Elimination of additional hydraulic loads through the site

Offsite drainage is focused solely on water management and is not used for salinity control. This type of drainage normally is accomplished through the use of open ditches. that serve as diversions for undesirable overland flow (Fig. 6.22). These drains are constructed across a slope or at the toe of a slope. Adequate capacity must be provided in the outlet to prevent any backup and overflow of the channel at design flow. The rational formula may be used for proper drain sizing.

6.11.7.2 Onsite drainage. The objectives of onsite drainage are more varied than offsite drainage and include the following:

1. Elimination of wet areas
2. Elimination of anaerobic conditions
3. Removal of salts
4. Protection from erosive flows

Onsite drainage may be accomplished through the use of surface or subsurface drains. In humid regions, the major source of water that must be drained from the site originates from precipitation. The sizes of the drainage channels are directly related to the rainfall and the soil infiltration rate. Regardless as to whether biosolids are being applied to land in an arid or humid region, the existence of a high groundwater table can be detrimental to crop yield. The drainage

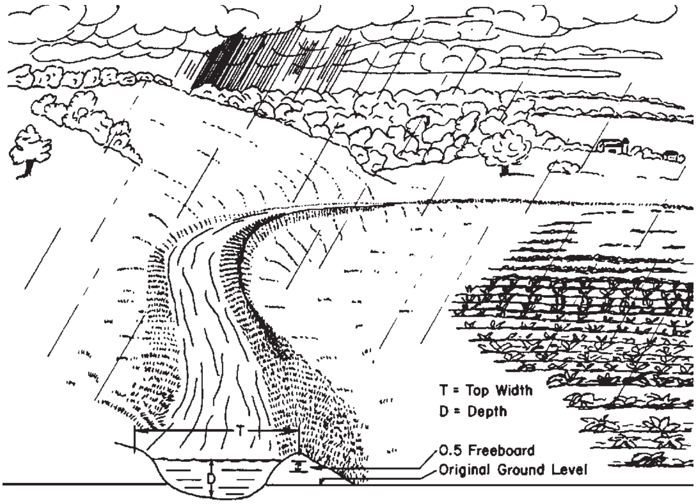


Figure 6.22 Typical open ditch drain for offsite drainage. (*Adapted by permission from ref. [29].*)

system must be designed to ensure that saturated conditions never occur within the crop root zone.

6.11.7.3 Drainage outlet. All drainage systems must have an outlet. Moreover, the drainage outlet must be of sufficient capacity to handle the anticipated peak-flow conditions. There are two principal types of outlets used for drainage systems: gravity and pump outlets. The topography of the land as well as the permeability of the soil dictates the choice of outlet [32] (Fig. 6.23).

6.11.8 Drainage system design

Once the moisture removal rate required to maintain proper site performance has been estimated, the size of the drains can be determined. Although there are many approaches to estimate drain sizes, the most universally accepted approach is based on the Manning formula. The Manning formula is used to estimate the flow velocity in a drain as a function of drain slope and area of flow. The Manning formula was first developed for flow in open channels but now is used for both open-channel and closed-conduit flows. The Manning formula is given by Eq. (6.35):

$$\text{Velocity (ft/s)} = \frac{1.486}{n} R^{2/3} S^{1/2} \qquad \text{(U.S. units)} \qquad (6.35a)$$

$$\text{Velocity (m/s)} = \frac{1.0}{n} R^{2/3} S^{1/2} \qquad \text{(SI units)} \qquad (6.35b)$$

Figure 6.23 Typical drainage outlet [32].

TABLE 6.26 Values of n to Be Used with the Manning Equation*

Type of drainage material	Average n value
Coated cast iron pipe	0.011
Uncoated cast iron pipe	0.012
Riveted and spiral steel pipe	0.013
Corrugated plastic drain pipe	0.012
Clay drainage tile	0.013
Concrete pipe	0.012
Concrete-lined channels	0.012
Straight and uniform earth channels	0.017
Smooth and uniform rock-lined channels	0.025
Earth channels with substantial vegetation (weeds)	0.035

*Data adapted from ref. [45].

where n = coefficient of roughness (Table 6.26)
R = hydraulic radius, ft (m), defined by Eq. (6.36)
S = slope of the energy grade line, ft/ft (m/m)

$$R = \frac{\text{cross-sectional area of flow, ft}^2\ (\text{m}^2)}{\text{wetted perimeter, ft (m)}} \tag{6.36}$$

For a circular drainage pipe flowing full, the hydraulic radius is given by Eq. (6.37):

$$R = \frac{(\pi/4)\,D^2}{\pi D} = \frac{D}{4} \tag{6.37}$$

where D is the pipe diameter (ft or m).

Examples 6.18 and 6.19 illustrate application of the Manning formula for both rectangular and circular drainage systems, respectively.

Example 6.18 Estimate the flow rate in a rectangular concrete drainage ditch with a width of 6 ft and a slope of 0.001 ft/ft when the depth of flow is 2 ft. Assume that n equals 0.014.

solution

Step 1. Determine the hydraulic radius R:

$$R = \frac{\text{area}}{\text{wetted perimeter}} = \frac{6\text{ ft} \cdot 2\text{ ft}}{6\text{ ft} + 2\,(2\text{ ft})} = 1.2\text{ ft}$$

Step 2. Estimate the flow rate Q:

$$Q = \text{area} \cdot \text{velocity} = (6\text{ ft} \cdot 2\text{ ft}) \cdot \frac{1.486}{n} R^{2/3} S^{1/2}$$

$$= 12\text{ ft}^2 \, \frac{1.486}{0.014} \, (1.2)^{2/3} \, (0.001)^{1/2}$$

$$= 45.5\text{ ft}^3/\text{s}$$

Example 6.19 Estimate the flow rate in a circular 12-in-diameter clay drain pipe flowing full. Assume that the drain has a slope of 0.0005 ft/ft and n equals 0.012.

solution

Step 1. Determine the hydraulic radius R for a circular pipe flowing full:

$$R = \frac{\text{area}}{\text{wetted perimeter}} = \frac{(\pi D^2)\,/4}{\pi D} = \frac{D}{4} = \frac{1\text{ ft}}{4} = 0.25\text{ ft}$$

Step 2. Estimate the flow rate Q:

$$Q = \text{area} \cdot \text{velocity} = \frac{\pi D^2}{4} \, \frac{1.486}{n} R^{2/3} S^{1/2}$$

$$= \frac{\pi\,(1)^2}{4} \, \frac{1.486}{0.012} \, (0.25)^{2/3} \, (0.0005)^{1/2}$$

$$= 0.86\text{ ft}^3/\text{s}$$

For circular drains flowing partially full, the relationship between various hydraulic elements to the depth of flow has been determined experimentally and is depicted graphically in Fig. 6.24. Example 6.20 illustrates the use of Fig. 6.24 to estimate the volumetric flow in partially full circular drains.

Example 6.20 A 0.75-m circular drain is placed on a slope of 0.002 m/m. If plastic drainage pipe is used (n equals 0.012), estimate the following:

1. Velocity and flow rate when the drain is flowing full

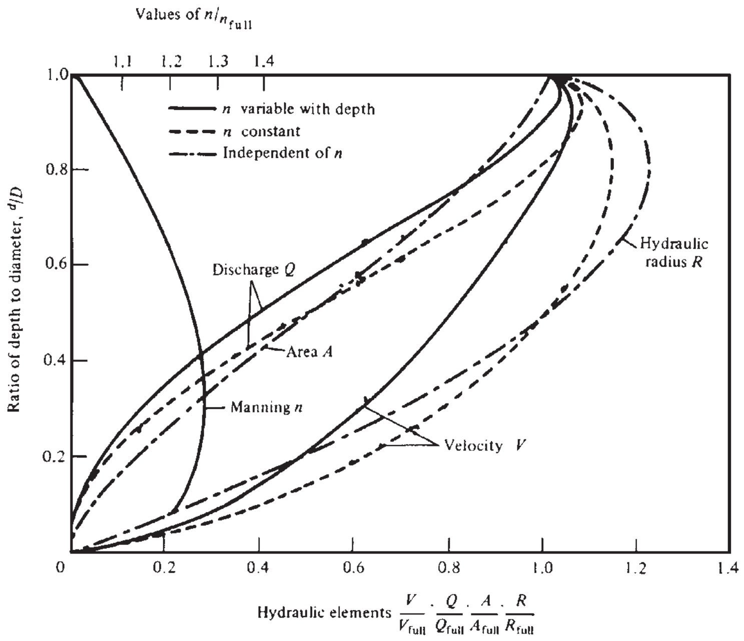


Figure 6.24 Hydraulic elements A/A_{full}, V/V_{full}, Q/Q_{full}, depth/diameter for circular drains and sewers. (*Adapted by permission from ref. [45].*)

2. Flow rate and velocity when the depth of flow is 0.2 m

solution

Step 1. Find velocity and flow rate when the drain is flowing full:

$$V_{\text{full}} = \frac{1}{n} R^{2/3} S^{1/2} = \frac{1}{0.012} \left(\frac{0.75}{4}\right)^{2/3} (0.002)^{1/2} = 1.22 \text{ m/s}$$

$$Q_{\text{full}} = \text{area} \cdot \text{velocity} = \frac{\pi}{4} (0.75)^2 (1.22 \text{ m/s}) = 0.54 \text{ m}^3\text{/s}$$

Step 2. Find velocity and flow rate when the depth of flow is 0.2 m. Determine the depth-to-diameter ratio:

$$\frac{d}{D} = \frac{0.2}{0.75} = 0.267$$

From Fig. 6.24, at this ratio

$$\frac{V}{V_{\text{full}}} \cong 0.73$$

$$\text{Flow velocity } (V) = 0.73\, V_{\text{full}} = 0.73 \cdot 1.22 \text{ m/s} = 0.89 \text{ m/s}$$

At the same depth-to-diameter ratio,

$$\frac{Q}{Q_{\text{full}}} = 0.16$$

$$\text{Volumetric flow rate } (Q) = 0.16 \cdot 0.54\ \text{m}^3/\text{s} = 0.09\ \text{m}^3/\text{s}$$

6.11.8.1 Maintenance of open-ditch drains. Open ditches are widely used for surface and subsurface drainage at biosolids land-application sites. They are used as individual field drains and for main drains. An important advantage of open ditches is their low initial cost and ready access for maintenance repair work. They are the least expensive type of drain where large amounts of surface runoff must be managed.

An important concern in the use of these types of drains is the difficulty they pose for some operations. Fields with a large number of open-ditch drains can be hazardous and normally require that bridges be constructed for mobility of large equipment. Another concern regarding use of these types of drains is their continual maintenance needs. Sedimentation and weed control must be part of the overall drainage maintenance plan. Weed control is especially important in open-ditch drains. The growth of weeds can seriously reduce the capacity of the ditch to carry water. Weeds can be removed by chemical or mechanical methods. Controlled burning of drainage ditches also has reduced the labor involved in the control of weeds.

6.11.8.2 Drain failure. The failure of a drainage system can result in the immediate closure of a biosolids land-application site. The anticipated performance of the application site depends on the proper management of moisture, which requires that the drainage system be designed and managed properly. There are a number of factors that contribute to the failure of a drain line. These include (1) improper alignment of sections, (2) large cracks or perforations in pipe, (3) grade reversal during construction, (4) failure or collapse because of excessive loads, (5) settlement of sections, (6) erosion of backfill soil into the drain line, (7) improper design of gravel envelopes designed to protect drain lines from entry of fines, and (8) improper locations of gravel envelope. Many of these can be prevented by quality control during drain line construction [2,32,50].

6.11.9 Drainage water quality criteria

As in all agricultural practices, drainage water quality must be monitored routinely before being discharged to surface waters. The applicable water-quality standards must be met and the required permits obtained before any discharge can take place. In some cases, treatment of drainage water before final disposal may be required [50].

6.12 Problems

6.1 The Gitega City Wastewater Treatment Plant is considering using biosolids to reclaim several mining sites in the area. Given the following chemical characteristics of an aqueous extract of soil from the disturbed site, classify the soil based on its electrical conductivity and sodium adsorption ratio.

Electrical conductivity EC_e: 18.0 mmhos/cm

[Ca^{2+}]: 62.0 mg/liter

[Mg^{2+}]: 30.0 mg/liter

[Na^{+}]: 420.0 mg/liter

6.2 The Durban County Sewer Improvement District is surface applying its thickened biosolids on forest land at a rate of 3.2 metric tons/ha (dry-mass basis). At this biosolids application rate, estimate the plant-available nitrogen for the first year of application. The biosolids chemical analysis indicated the following constituent values (dry-weight basis):

NO_3^{2-}: 1.6 percent

NH_4^{+}: 0.2 percent

Organic nitrogen: 3.8 percent

6.3 The Malakal City Wastewater Treatment Plant is currently subsurface injecting anaerobically digested biosolids on disturbed land to promote revegetation. If the biosolids are being applied once at the rate of 12.0 metric tons/ha (dry-mass basis), estimate the fraction of organic nitrogen that will be available to plants during years 1, 2, and 3. Assume that the organic matter content (N_o) of the biosolids is 3.5 percent (dry-mass basis).

6.4 The Tunduru Mining Company desires to apply 15.0 metric tons/ha (dry-mass basis) on its mine tailings impoundment. If the biosolids application rate is to occur only once, estimate the mass of organic nitrogen mineralized in years 1, 2, and 3 from the biosolids application using the mineralization factors K_m. Assume that the organic nitrogen content of the biosolids is 4.0 percent (dry-mass basis).

6.5 The Biera City Water Reclamation Plant is subsurface injecting 4 metric tons/year (dry weight) of thickened biosolids on agricultural land. If the biosolids have the following chemical composition, estimate the plant-available nitrogen during each of the first 3 years of application.

NO_3^{2-}: 0.1 percent

NH_4^{+}: 3.0 percent

Organic nitrogen: 2.2 percent

6.6 A 16-lb soil sample loses 4.5 lb of moisture on drying in an oven set at 105°C. What is the moisture content of this soil on a dry-weight and volumetric basis (inches of H_2O per inch of soil depth) if the initial soil sample has a volume of 0.19 ft^3?

6.7 A 200-g soil sample loses 52 g of moisture on drying in an oven set at 105°C. What is the moisture content of this soil on a dry-weight and volumetric basis (inches of H_2O per inch of soil depth) if the initial soil sample has a volume of 0.14 liter?

6.8 An 8-lb soil sample having a volume of 0.09 ft^3 is analyzed for water retention capacity using a pressure-plate apparatus. If the soil sample has a moisture content of 22 percent after being subjected to a -0.33-bar tension for 6 hours and a moisture content of 10 percent after being subjected to a -15-bar tension for 1 hour, estimate the soil field capacity (FC), permanent wilting point (PWP), and available water capacity (AWC).

6.9 A 500-g soil sample having a volume of 390 ml is analyzed for water retention capacity using a pressure-plate apparatus. If the soil sample has a moisture content of 28 percent after being subjected to a -0.33-bar tension for 6 hours and a moisture content of 9 percent after being subjected to a -15-bar tension for 1 hour, estimate the soil field capacity (FC), permanent wilting point (PWP), and available water capacity (AWC).

6.10 An 800-ml soil sample from a biosolids land-application site weighs 1130 g. If, after drying the sample at 105°C for 6 hours, the sample weighs 874 g, estimate the moisture content on a mass and volume basis, bulk density, and soil porosity. Assume that the soil particle density is 3.2 gm/cm^3.

6.11 A 0.10-ft^3 soil sample from a biosolids land-application site weighs 6.92 lb. If, after drying the sample at 105°C for 6 hours, the sample weighs 4.87 lb, estimate the moisture content on a mass and volume basis, bulk density, and soil porosity. Assume that the soil particle density is 85.0 lb/ft^3.

6.12 Two locations in a soil profile at a biosolids land-application site are being monitored for their moisture potential. If the pressure potentials at points A and B are -60 and -25 cmH_2O, respectively, determine (A) the total potential at points A and B (ignore osmotic potential) and (B) the direction of moisture flow.

6.13 At a biosolids land-application site, two locations in a soil profile are being monitored for their moisture potential. If the pressure potentials at points A and B are -50 and -85 cmH_2O, respectively, determine (A) the total potential at points A and B (ignore osmotic potential) and (B) the direction of moisture flow.

6.14 The Maputo City Water Reclamation Plant is considering land application of its thickened biosolids on adjacent pastureland. If two piezometers are installed in the same vicinity at the potential site but at different depths (as illustrated in the accompanying figure), estimate the hydraulic potentials of the two points and the vertical direction of water flow (ignore osmotic potential).

6.15 The Upington Sewer Improvement District has installed a groundwater well to increase the depth of the unsaturated zone at its biosolids

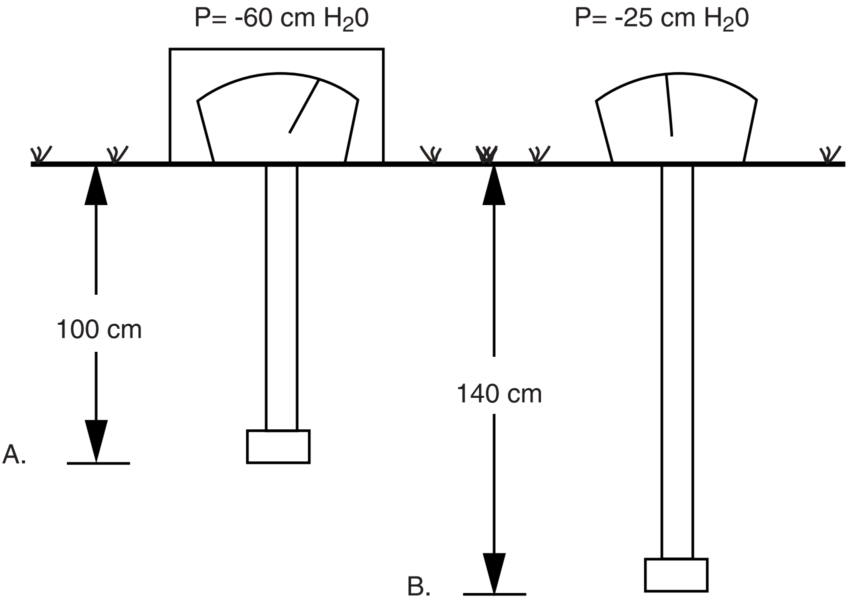


Figure Problem 6.12 Legend to come.

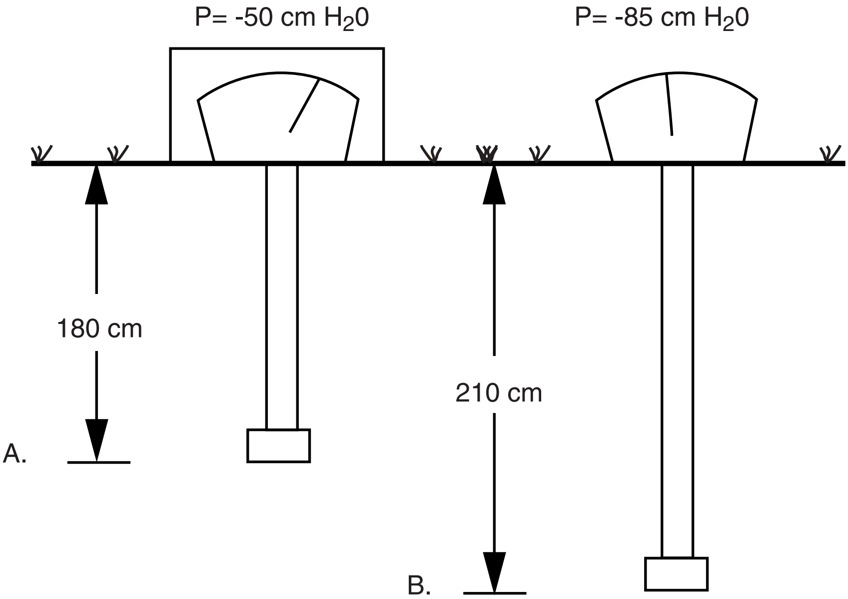


Figure Problem 6.13 Legend to come.

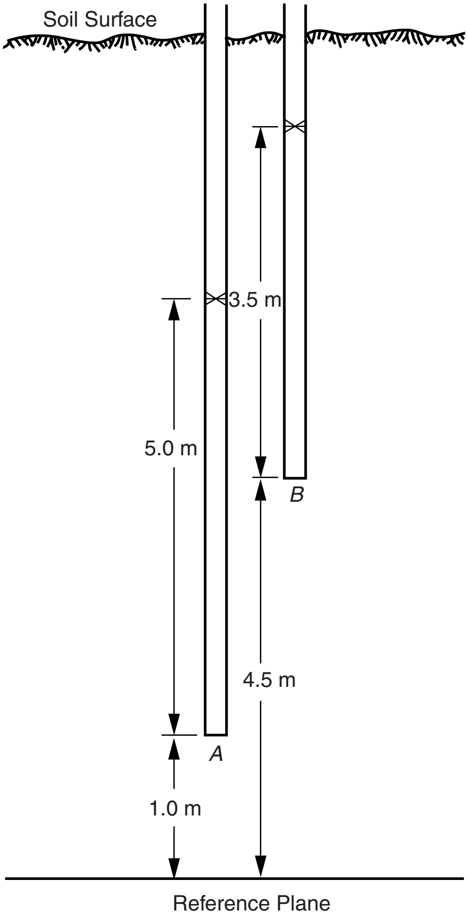


Figure Problem 6.14 Legend to come.

land-application site. If the steady-state draw-down from the well is given in the accompanying figure, estimate the hydraulic gradient between points *A* and *B*. Assume that the following data apply: $s_1 = 100$ ft; $s_2 = 60$ ft; $h_A = 68.1$ ft; and $h_B = 64.9$ ft.

6.16 The Ondangwa County Water Reclamation Plant is currently land applying its thickened biosolids on an agricultural site of 80 ha. If the depth to the water table is 2.0 m and the unsaturated soil has an average tension of -230 cmH$_2$O, estimate the vertical volumetric flow rate of groundwater through the land-treatment area. Assume that the hydraulic conductivity of the soil is approximately 8.5×10^{-6} cm/s and the depth to the water table remains constant. The effects of osmotic potential can be neglected.

6.17 The Dirico City Wastewater Treatment Plant is currently evaluating the suitability of several potential biosolids land-application sites. If the following

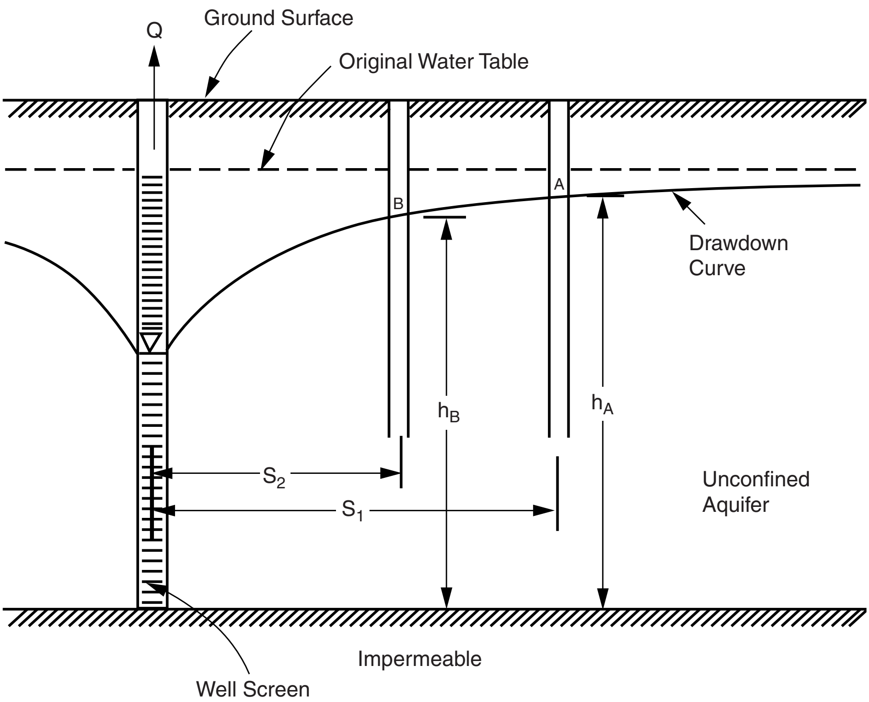


Figure Problem 6.15 Legend to come.

moisture infiltration data were obtained using a double-ring infiltrometer, estimate the coefficients of the Kostiakov equation and the infiltration rate (cm/h) for this soil after 8 hours.

Time (min)	Depth of infiltration (cm)
30	0.8
240	5.9

6.18 The Milanje City Water Reclamation Plant is currently applying biosolids to 450 acres of agricultural land. If the land is primarily pastureland with an average slope of 2 percent and stripcropping is being employed to reduce soil erosion, estimate the annual soil loss from the site in tons per year given the following conditions:

Rain and runoff factor *R:* 120

Soil erodibility factor *K:* 0.21

Topographic factor *LS:* 0.25

Cover and management factor *C:* 0.15

Support practice factor *P:* 0.30

6.19 The Lusaka County Sewer Improvement District has chosen to grow coffee on its biosolids beneficial-use site. If the crop is irrigated with water whose electrical conductivity is 0.60 mmhos/cm and a leaching fraction of 0.20 is maintained, estimate (A) the salinity of the leaching fraction and (B) the minimum leaching fraction necessary to maintain crop yield. Assume that coffee is known to suffer a significant loss in yield when the total dissolved solids (TDS) concentration of the soil water exceeds 3600 mg/liter.

6.20 The Sapele City Wastewater Treatment Plant is currently applying liquid biosolids to several fruit orchards located adjacent to the facility. If the crop requires periodic irrigation to meet moisture needs, estimate the leaching fraction, time of irrigation and the volume of irrigation water used per irrigation event if the following conditions apply:

Crop: Grapefruit
EC_{iw}: 1.6 mmhos/cm
ET_c: 7 mm/day
Allowable crop yield reduction: 10 percent
Soil infiltration rate: 1.4 cm/h
Irrigation frequency: 5 days

6.21 The Loudima County Sewer Improvement District is currently using local surface water to irrigate crops on its biosolids beneficial-use site. If the electrical conductivity of the irrigation water EC_{iw}, electrical conductivity of the drainage water EC_{dw}, and crop evapotranspiration are 2.6 and 8.2 mmhos/cm and 0.80 in/day, respectively, estimate the rise in the groundwater table during crop irrigation.

6.22 The Okoyo City Wastewater Treatment Plant has constructed concrete drainage ditches upgradient from its biosolids land-application site to divert overland moisture flow. If the drainage ditch is rectangular in cross-sectional area and has a width of 8 ft and a slope of 0.002 ft/ft, estimate the flow rate when the depth of flow is 1.5 ft. Assume that the Manning coefficient n equals 0.014.

6.23 The Bitam County Water Reclamation Plant has constructed rectangular earthen ditches to divert all overland flow from the biosolids land-application site to temporary storage lagoon. If the earthen ditches have a width of 6 ft, estimate the flow rate if the bottom slope is 0.003 ft/ft and the depth of flow is 1.8 ft. Assume that the Manning coefficient n for the earth ditch equals 0.023.

6.24 The Omondi County Sewer Improvement District has constructed subsurface drains beneath the biosolids land-application site to maintain a suitable depth of the unsaturated zone. If the clay drainpipe is 10 in in diameter and has a slope of 0.004 ft/ft, estimate the flow rate when the drainpipe is full. Assume that the Manning coefficient n for the subsurface drainpipe equals 0.013.

6.25 For the Omondi County Sewer Improvement District (see Prob. 6.24), if the depth of flow in the subsurface drainpipe is 3 in, estimate the flow velocity and volumetric flow rate. The clay drainpipe is 10 in in diameter and has a slope of 0.004 ft/ft. The Manning Coefficient *n* for the drainpipe equals 0.013.

6.13 References

1. Aguilar, R., J. Gosz, and T. Ward (1992), "Sewage Sludge Application in Semiarid Grasslands: Effects on Vegetation and Water Quality. Annual Report of Attainment," Project No.1423645, New Mexico Water Resources Research Institute (NMSU), Las Cruces, N.Mex.
2. American Society of Agronomy (1974), *Drainage for Agriculture,* Madison, Wis.
3. Bastian, R. K. (1997), "The Biosolids (Sludge) Treatment, Beneficial Use and Disposal Situation in the USA," *European Water Pollution Control,* pp. 62–79.
4. Beauchamp, E. G., G. E. Kidd, and G. Thurtell (1978), "Ammonia Volatilization from Sewage Sludge Applied in the Field," *Journal of Environmental Quality* 7:141–146.
5. Bohn, H. L., B. L. McNeal, and G. A. O'Connor (1979), *Soil Chemistry,* John Wiley, New York.
6. Brady, N. C. (1984), *The Nature and Properties of Soil,* 9th ed., Macmillan, New York.
7. Carter, M., ed. (1993), *Soil Sampling and Methods of Analysis,* Lewis Publishers, Boca Raton, Fla.
8. Chae, Y. M., and M. A. Tabatabai (1986), "Mineralization of Nitrogen in Soils Amended with Organic Wastes," *Journal of Environmental Quality* 15:193–198.
9. Chaney, R. L. (1983), "Plant Uptake of Inorganic Waste Constituents," in: J. F. Parr, P. B. Marsh, and J. M. Kla, (eds.), *Land Treatment of Hazardous Wastes,* Noyes Data Corporation, Park Ridge, N.J. pp. 50–76.
10. Chaney, R. L., J. B. Munns, and H. M. Cathey (1980), "Effectiveness of Digested Sewage Sludge Compost in Supplying Nutrients for Soil-less Potting Media," *Journal of the American Society for Horticulture Science* 105:485–492.
11. Coker, E. G. (1983), "The Use of Sewage Sludge in Agriculture," *Water Science Technology* 15:195–208.
12. Colorado State University (1995), "Effects of Land Application of Biosolid in Arid and Semi-Arid Environments: Proceedings," Metro Wastewater Reclamation District, Denver, Colo.
13. Crohn, D. M. (1996), "Planning Biosolids Land Application Rates for Agricultural Systems," *Journal of Environmental Engineering,* pp. 1058–1066.
14. Cuenca, R. H. (1989), *Irrigation System Design: An Engineering Approach,* Prentice-Hall, Englewood Cliffs, N.J.
15. Dowdy, R. H., and W. E. Larson (1975), "Metal Uptake by Barley Seedlings Grown on Soils Amended with Sewage Sludge," *Journal of Environmental Quality* 4:229–233.
16. Duncomb, D. R., W. E. Larson, C. E. Clapp, R. H. Dowdy, D. R. Linden, and W. K. Johnson (1982), "Effect of Liquid Wastewater Sludge Application on Crop Yield and Water Quality," *Journal of Water Pollution Control Federation* 54:1185–1193.
17. Fredlund, D. G., and H. Rahardjo (1993), *Soil Mechanics for Unsaturated Soils,* Wiley Interscience, New York.
18. Fresquez, P. R., R. E. Francis, and G. L. Dennis (1990), "Effects of Sewage Sludge on Soil and Plant Quality in a Degraded Semiarid Grassland," *Journal of Environmental Quality* 19:324–329.
19. Garvey, D., C. Guarino, and R. Davis (1993), "Sludge Disposal Trends Around the Globe," *Water/Engineering & Management,* pp. 17–20.
20. Gilmor, J. T., F. Roman, and M. D. Clark (1996) "Decomposition of Biosolids in a Disposal Site Soil," *Journal of Environmental Quality* 25:1083–1086.
21. Gilmour, J., and M. Clark (1988), "Nitrogen Release from Wastewater Sludge: A Site-Specific Approach," *Journal of Water Pollution Control Federation* 60:494–341.

22. Goldstein, N., W. A. Yanko, J. M. Walker, and W. Jakubowski (1988), "Determining Pathogen Levels in Sludge Products," *Biocycle* 29:44–47.
23. Gouin, F. R., and J. M. Walker (1977), "Deciduous Tree Seedling Response to Nursery Soil Amended with Composted Sewage Sludge," *Horticulture Science* 12:45–47.
24. Huddleston, J. H. and M. P. Ronayne (1990), "Guide to Soil Suitability and Site Selection for Beneficial Use of Sewage Sludge," Oregon State University Extension Service.
25. Jewell, W. J. (1980), "Use and Treatment of Municipal Wastewater and Sludge in Land Reclamation and Biomass Production Projects: An Engineering Assessment," pp. 448–480, in *Utilization of Municipal Wastewater and Sludge: An Engineering Assessment for Land Reclamation and Biomass Production.* EPA 430/9-81-012, Washington.
26. Keeney, D. (1982), "Nitrogen-Availability Indices," in Page, A. L., (ed.) *Methods of Soil Analysis,* Part 2, 2d ed. American Society of Agronomy, Madison, WI, pp. 711–733.
27. Keeney, D., K. Lee, and L. Walsh (1975), "Guidelines for the Application of Wastewater Sludge to Agricultural Land in Wisconsin," Technical Bulletin 88, Wisconsin Department of Natural Resources, Madison, Wis.
28. Kelling, K. A., A. E. Peterson, L. M. Walsh, J. A. Ryan, and D. R. Keeney (1977), "A Field Study of Agricultural Use of Sewage Sludge: Effect on Crop Yield and Uptake of N and P," *Journal of Environmental Quality* 6:339–345.
29. Loehr, R., W. Jewell, J. Novak, W. Clarkson, and G. Friedman (1979), *Land Application of Wastes,* Vol. 2, Van Nostrand Reinhold, New York.
30. Loehr, R. C. (1977), *Land as a Waste Management Alternative,* Ann Arbor Science, Ann Arbor, MI.
31. Logan, T. J., and R. L. Chaney (1983), "Utilization of Municipal Wastewater and Sludge on Land: Metals," pp. 235–326, in A. L. Page et al. (eds.), *Utilization of Municipal Wastewater and Sludge on Land,* Univ. of California Press, Riverside, Calif.
32. Luthin, J. N. (1973), *Drainage Engineering,* Krieger Publishing.
33. Mitchell, J. K. (1993), *Fundamentals of Soil Behavior,* 2d ed., Wiley Interscience, New York.
34. National Association of Conservation Districts (1982), "Sludge and the Land: The Role of Soil and Water Conservation Districts in Land Application of Sewage Sludge," U.S. Environmental Protection Agency, 430/9-82-007, Washington.
35. Parkin, W. P. (1993), *The Complete Guide to Environmental Law,* Specialty Technical Publishers.
36. Power, J. F., and J. Alessi (1971), "Nitrogen Fertilization of Semiarid Grasslands: Plant Growth and Soil Mineral Nitrogen Levels," *Journal of Agronomy* 63:277–280.
37. Ryan, J., and R. Chaney (1993), "Regulation of Municipal Sewage Sludge Under the Clean Water Act Section 503: A Model for Exposure and Risk Assessment for MSW-Compost," in *Science and Engineering of Composting,* Renaissance Publications, Worthington, OH.
38. Sander, D., D. Walthers, and K. Frank (1994), "Nitrogen Testing for Optimum Management," *Journal of Soil and Water Conservation* 49(2):46–52.
39. Soon, Y. K., T. E. Bates, and J. R. Moyer (1978), "Land Application of Chemically Treated Sewage Sludge: II. Effects on Plant and Soil Phosphorus, Potassium, Calcium, Magnesium and Soil pH," *Journal of Environmental Quality* 7:269–273.
40. Soon, Y. K., T. E. Bates, E. G. Beauchamp, and J. R. Moyer (1978), "Land Application of Chemically Treated Sewage Sludge: I. Effects on Crop Yield and Nitrogen Availability," *Journal of Environmental Quality* 7:264–269.
41. Sopper, W. (1993), *Municipal Sludge Use in Land Reclamation,* Lewis Publishers, Boca Raton, Fla.
42. South Carolina Department of Health (1987), *Land Application of Sludge Guidance Manual.*
43. Sposito, G. (1989), *The Chemistry of Soils,* Oxford University Press, New York.

44. Sposito, G. (1984), *The Surface Chemistry of Soils,* Oxford University Press, New York.
45. Tchobanoglous, G. (1981), *Wastewater Engineering: Collection and Pumping of Wastewater,* McGraw-Hill, New York.
46. Tchobanoglous, G. (1991), *Wastewater Engineering: Treatment Disposal and Reuse,* 3d ed., McGraw-Hill, New York.
47. U.S. Army Corps of Engineers (1987), *Wetlands Delineation Manual,* Technical Report Y-87-1, Waterways Experiment Station, Vicksburg, Miss.
48. U.S. Department of Agriculture (1989), "Peer Review, Standards for the Disposal of Sewage Sludge. USEPA Proposed Rule 40 CFR Parts 257 and 503 USDA-CSRS Technical Committee," Univ. of California, Riverside, Calif.
49. U.S. Department of Agriculture (1994), *Sewage Sludge: Land Utilization and the Environment,* American Society of Agronomy, Crop Science Society of America, Soil Science Society of America.
50. U.S. Department of the Interior (1993), *Drainage Manual: A Water Resources Technical Publication,* 3d ed., Washington.
51. U.S. Environmental Protection Agency (1983), "Process Design Manual: Land Application of Municipal Sludge," EPA/625/1-83/016, Washington.
52. U.S. Environmental Protection Agency (1973), "Park Development with Wet Digested Sludge," EPA-R2-73-143, Washington.
53. U.S. Environmental Protection Agency (1979), "Process Design Manual for Sludge Treatment and Disposal," EPA/625/1-79/011, Washington.
54. U.S. Environmental Protection Agency (1990), "Environmental Regulations and Technology: Autothermal Thermophilic Aerobic Digestion of Municipal Wastewater Sludge," EPA/625/10-90/007, Washington.
55. U.S. Environmental Protection Agency (1992), "Environmental Regulations and Technology: Control of Pathogens and Vector Attraction in Sewage Sludge," EPA/625/R-92/013, Washington.
56. U.S. Environmental Protection Agency (1993), "Domestic Septage Regulatory Guidance: A Guide to The EPA 503 Rule," EPA/832/B-92/005, Washington.
57. U.S. Environmental Protection Agency (1993), "Preparing Sewage Sludge for Land Application or Surface Disposal: A Guide for Preparers of Sewage Sludge on the Monitoring, Recordkeeping, and Reporting Requirements of the Federal Standards for the Use or Disposal of Sewage Sludge, 40 CFR Part 503," EPA/831/B-93/002a, Washington.
58. U.S. Environmental Protection Agency (1993), "Regulatory Impact Analysis of the Part 503 Sewage Sludge Regulation," EPA/821/R-93/006, Washington.
59. U.S. Environmental Protection Agency (1993), "Standards for Use or Disposal of Sewage Sludge. Final Rule, 40 CFR Part 503," *Federal Register* 58(32):9248-9415.
60. U.S. Environmental Protection Agency (1993), "Technical Support Document for Land Application of Sewage Sludge," Vol. I," PB93-110575; Vol. II, PB93110583, Washington.
61. U.S. Environmental Protection Agency (1993), "Standards for the Use or Disposal of Sewage Sludge," *Federal Register* 58(32):9248–9415.
62. U.S. Environmental Protection Agency (1994), "A Plain English Guide to the EPA Part 503 Biosolids Rule," EPA/832/R-93/003, Washington.
63. U.S. Environmental Protection Agency (1994), "Guide to Septage Treatment and Disposal," EPA/625/R-94/002, Washington.
64. U.S. Environmental Protection Agency (1994), "Land Application of Sewage Sludge: A Guide for Land Appliers on the Requirements of the Federal Standards for the Use or Disposal of Sewage Sludge, 40 CFR Part 503," EPA/831/B-93/002b, Washington.
65. U.S. Environmental Protection Agency (1995), "A Guide to the Biosolids Risk Assessments for the EPA Part 503 Rule," EPA/832/B-93/005, Washington.
66. U.S. Environmental Protection Agency (1995), "Ground-Water and Leachate Treatment Systems," EPA/625/R-94/005, Washington.
67. U.S. Environmental Protection Agency (1995), "Part 503 Implementation Guidance," EPA/833/R-95/001, Washington.

68. U.S. Environmental Protection Agency (1995), "Process Design Manual: Land Application of Sewage Sludge and Domestic Septage," EPA/625/R-95/001, Washington.
69. U.S. Environmental Protection Agency (1995), "Process Design Manual: Surface Disposal of Sewage Sludge and Domestic Septage," EPA/625/R-95/002, Washington.
70. U.S. Environmental Protection Agency (1999), "Environmental Regulations and Technology: Control of Pathogens and Vector Attraction in Sewage Sludge," EPA/625/R-92/013, Washington.
71. Vesilind, P. A., G. C. Hattman, and E. T. Shene (1986), *Sludge Management and Disposal for the Practicing Engineer,* Lewis Publishers.
72. Viessman, W., and M. J. Hammer (1985), *Water Supply and Pollution Control,* 4th ed., Harper Collins, New York.
73. Weber, W. (1972), *Physicochemical Processes for Water Quality Control,* Wiley, New York.
74. Wei, Q. F., B. Lowery, and A. E. Peterson (1985), "Effect of Sludge Application on Physical Properties of a Silty Clay Loam Soil," *Journal of Environmental Quality* 14:178–180.
75. Whitford, W. G., E. A. Aldon, D. W. Freckman, Y. Steinberger, and L. W. Parker (1989), "Effects of Organic Amendments on Soil Biota on a Degraded Rangeland," *Journal of Range Management* 42:56–60.

Chapter 7

Beneficial Use of Biosolids

7.0 Introduction

Biosolids (or sludge) disposal has always represented a substantial portion of the costs associated with municipal wastewater treatment. Over the past 25 years, legal restrictions placed on biosolids/sludge disposal practices (e.g., ban on ocean dumping) together with public opposition to certain types of biosolids disposal practices (e.g., incineration) have encouraged many public wastewater treatment utilities to regard biosolids beneficial use as an increasingly cost-effective biosolids management alternative.

Beneficial use of biosolids generally is defined as the application of biosolids to land in order to take advantage of their nutritional and/or soil-conditioning characteristics [4,22,49]. Throughout the world, public health and environmental concerns regarding biosolids management practices are accelerating biosolids beneficial use as an acceptable alternative to landfilling or incineration. For example, in the United States, the ocean dumping ban of 1988, together with the increasing costs of landfilling and incineration, has prompted many communities to invest in biosolids beneficial use (note that in 1996, approximately 36 percent of biosolids were being used beneficially in the United States). In Europe, the decision by the European Economic Community (EEC) to eliminate ocean dumping of biosolids by 1998 has led to a substantial increase in the number of biosolids management programs with a focus on beneficial use, whereas in Japan, where over 80 percent of biosolids currently are either incinerated or landfilled, it is anticipated that over 3 million metric tons per year of biosolids will be used beneficially by the year 2015 [4,5,33].

Although most countries have their own national guidelines for biosolids reuse, the risk-based approach adopted by the United States is becoming a standard model for biosolids beneficial-use program development in both Europe and Asia [4,33]. Because of the likelihood that many countries will adopt at least part of the risk-based biosolids beneficial-use approach developed by the United States, this chapter will focus on the design of biosolids beneficial-use systems that comply with the 40 CFR Part 503 rule [97,100].

The process of designing a biosolids beneficial-use program begins with a preliminary planning step that includes the collection and assessment of biosolids quality and generation rates. This information is used to estimate the anticipated biosolids land-application rates and total land requirements. Once preliminary biosolids characteristics and generation rate data are obtained, the biosolids quality and land requirements should be compared with applicable federal, state, and local regulations to determine if unacceptable levels of toxic compounds or insufficient available land will eliminate biosolids beneficial use as a practical biosolids management alternative.

In addition to biosolids quality, biosolids generation rates, and land area requirements, the mode of biosolids transport to the application site, as well as public opinion regarding biosolids beneficial use, should be evaluated during the preliminary planning stage. If, after evaluating the results from the preliminary planning stage, biosolids land application remains a feasible option, a detailed Phase I site-screening investigation is initiated. The U.S. Environmental Protection Agency (USEPA) has provided technical guidelines for effectively conducting the planning and design of a biosolids beneficial-use system (Fig. 7.1). As a result of the Phase I effort, one or more biosolids land-treatment options should be identified as technically, socially, and economically feasible for the design process to continue. The final engineering design and costs for each beneficial-use option is evaluated during a Phase II site evaluation. The result of the Phase II site evaluation will be a final ranking of each of the viable options, with the most cost-effective option being selected as the preferred design. Details of the preliminary planning, Phase I, and Phase II design steps are described in the following sections.

7.1 Preliminary Planning Process

7.1.1 Public participation

Public participation is critical during the initial stages of planning a biosolids beneficial-use project. Public participation may include town meetings, community workshops, and television/radio talk shows [100]. Early community involvement helps the design engineer gauge the limits to public (and political) acceptability of the project. Strong

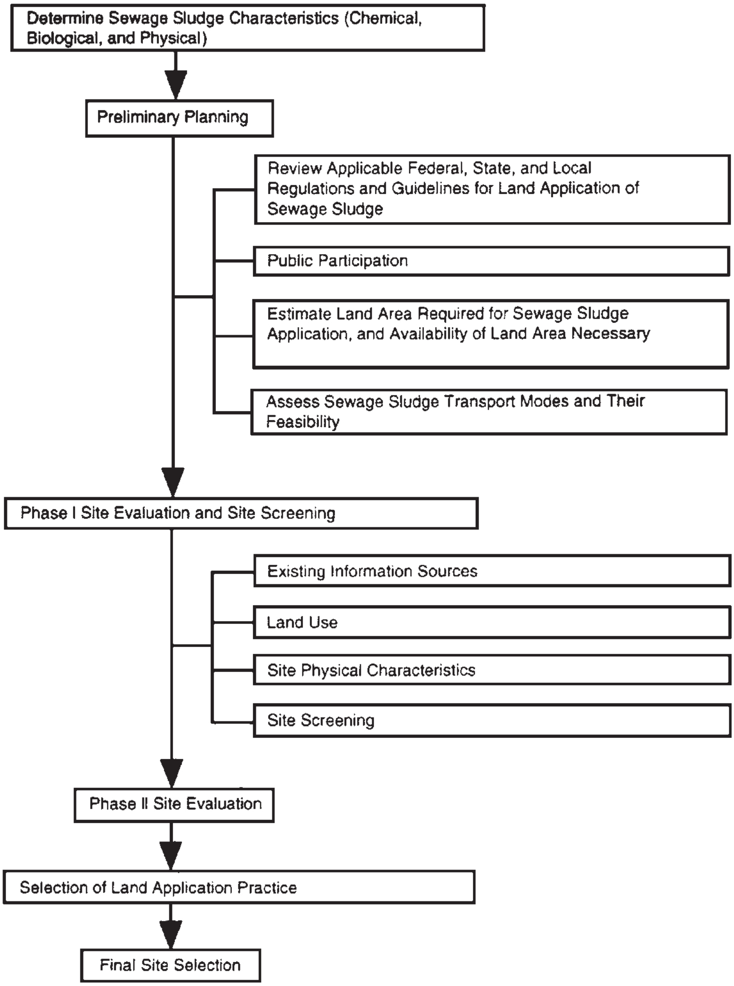


Figure 7.1 Technical guidelines for biosolids beneficial-use planning and design.

local support for a biosolids beneficial-use program may be achieved by developing and implementing a public educational program that emphasizes both the economic and societal benefits of biosolids beneficial use as well as the inherent level of protection for public health and the environment that underlies the legal requirements established in the 40 CFR Part 503 rule.

7.1.2 Land area requirements

Although a precise determination of land area requirements should be based on detailed design calculations, for preliminary planning pur-

TABLE 7.1 Preliminary Estimates of Biosolids Application for Various Land Types*

Land type	Time period of application	Typical rates	
		Metric tons/ha†	Ton/acre‡
Agricultural land	Once or twice annually	10 (Range, 2–70)	5 (Range, 1–30)
Forest land	Annually (or at 3- to 5-year intervals)	18 (Range, 10–220)	8 (Range, 4–100)
Land reclamation	One time	112 (Range, 7–450)	50 (Range, 3–200)

*Adapted from ref. [100].
†Metric tons/ha, metric tons (1000 kg) per hectare.
‡Ton/acre, U.S. ton (2000 lb) per acre.

poses, an estimate of the land area requirements can be obtained from the data contained in Table 7.1. For example, according to Table 7.1, to land apply 1000 tons (U.S. units, 2000 lb/ton) of biosolids (dry weight) per year, the required land area for agricultural biosolids land application would be approximately 200 acres (1000 tons/year ÷ 5 tons/acre) plus an additional area (normally 10 percent) for buffer zones and biosolids storage. On the other hand, for a one-time land application of 1000 tons of biosolids at a land reclamation site, a 20-acre area (1000 tons ÷ 50 tons/acre) would be required. It should be noted that the land area estimate based on data from Table 7.1 represents the minimum land area requirement and should only be used as a screening measure. If no sites are identified with at least the minimum land area requirements, the biosolids beneficial-use design process should be discontinued.

7.1.3 Biosolids transport

Transport of biosolids can represent a major cost to a biosolids beneficial-use project. Moreover, the cost of biosolids transport will have a direct effect on the size of the geographic search area being surveyed for potential beneficial-use sites. Although biosolids can be transported directly to the beneficial-use site by pipeline, truck, or rail, in many circumstances, combined transport methods (e.g., rail-truck) must be used (Chap. 5). The choice of the biosolids transportation method depends on the (1) biosolids volume and solids content, (2) number and distance to each destination point, and (3) type of land selected (e.g., agricultural, forest, reclamation, public access). The impact of solids content on the available transportation options is summarized in Table 7.2.

Although cost prohibitive for the transport of large volumes of dilute biosolids, truck conveyance allows greater flexibility than any other

TABLE 7.2 Impact of Solids Content on Biosolids Transport Options*

Biosolids type	Typical solids content	Transport options
Liquid	1–10	Pipeline, tank truck, gravity flow, rail
Cake	10–50	Conveyor, auger truck transport
Dried	50–95	Conveyor, front end loader, truck transport

*Adapted from ref. [80].

transport option. With truck transport of biosolids, destinations can be changed with little advanced notice, and the biosolids can be distributed to many different locations. Many biosolids beneficial-use programs use truck transport either alone or after biosolids are transported by pipeline or rail to an intermediate storage facility [80].

At many publicly owned treatment works (POTWs), biosolids are dewatered prior to being transported to beneficial-use sites. In evaluating biosolids transportation options, the costs of dewatering must be weighed against the cost savings associated with transporting drier biosolids. Table 7.3 provides a comparison of the limitations associated with transporting liquid and dried biosolids.

Once the preliminary planning process is completed, the biosolids beneficial-use program should be characterized as either feasible or not. Reasons for canceling a biosolids beneficial-use program are varied but can include technical flaws as well as economic and/or political

TABLE 7.3 Comparison of Transport Options for Biosolids*

Biosolid type	Transport option considerations
Liquid	
Tank truck	Maximum load capacity for roads is approximately 6600 gal.
Pipeline	Requires minimum velocity of 1 ft/s to keep solids in suspension. Pipeline construction normally have high capital costs.
Rail tank car	Normally, 24,000-gal capacity limit per tank car. Solids normally will settle in transit.
Cake or dried biosolids	
Truck	Commercial equipment available to unload and spread on soil.
Farm manure spreader	Appropriate for small systems where nearby farmland is accessible.
Rail hopper car	Although, in many cases, conventional unloading equipment can be used, many need special accessories for handling biosolids.

*Adapted from refs. [80,100].

factors. If no obvious reason exists for terminating the biosolids beneficial-use design process, the preliminary list of suitable land-application sites is then evaluated in further detail during the Phase I site-screening investigation.

7.2 Phase I Site-Screening Investigation

The Phase I site-screening investigation uses the land area and transportation assessment from the preliminary planning effort to evaluate potential beneficial-use land-application sites. Land use and availability should be addressed immediately by the design engineer/planner to avoid the unnecessary expense of evaluating unprocurable sites. Site screening also allows elimination of unsuitable areas due to physical, environmental, or political factors.

Public sources of information may be used initially to evaluate site characteristics. Sources of information on land characteristics, cropping patterns, and other relevant data can be found by contacting government agencies, including (1) the U.S. Department of Agriculture, (2) the U.S. Environmental Protection Agency, and (3) the U.S. Geological Survey. Other sources include state universities and local planning and/or health departments.

7.2.1 Land use and availability

Projected land use will influence site selection as well as acceptance (or rejection) of a particular biosolids beneficial-use program. Zoning and land-use planning are closely related, and zoning ordinances generally reflect future land-use planning. Applicable zoning laws that may affect potential biosolids land-application sites should be reviewed concurrently with land-use evaluations. Since it is unlikely that a community will have a specific area zoned for biosolids land application, project proponents may need to petition for a zoning change [100].

7.2.2 Aesthetics

Selection of a biosolids beneficial-use site can be influenced by community concerns over such issues as noise, fugitive dusts, and odors. In addition to the beneficial-use site, biosolids transport routes should be mapped to avoid residential areas, bridge load limitations, etc. Disruption of the local scenic character and/or recreational activities may generate strong local opposition to a biosolids beneficial-use program. Therefore, buffer zones usually are required to separate biosolids land-application sites from residences, roads, parks, etc.

7.2.3 Site acquisition

Land application of biosolids often can be accomplished without direct purchase or lease of land. Educational and public participation programs can be used early in the planning stages to identify landowners willing to participate in a biosolids land-application program. This type of arrangement often is more acceptable to the public than direct land purchase. However, in cases where landowners cannot be identified to accept biosolids, direct purchase or lease of land may be necessary. Under these circumstances, it is desirable to purchase land as close to the wastewater treatment plant as possible [48,100].

7.2.4 Physical factors

When evaluating the technical aspects of a biosolids beneficial-use site, the site physical factors will have a significant impact on required capital improvements and operational costs. The physical factors of concern at a biosolids land-application site include the following:

- Topography
- Soil permeability, infiltration, and drainage patterns
- Proximity to surface water
- Depth to groundwater
- Climate

Each of these factors is described in more detail in the following sections.

7.2.4.1 Topography. Topography influences surface water movement, which, in turn, affects soil erosion and surface runoff. Uncontrolled runoff can transfer biosolids to surface waters, leading to water quality deterioration. Federal, state, and local laws often stipulate the maximum slopes allowable for biosolids land-application sites (Table 7.4).

Although steep slopes (e.g., 30 to 50 percent) normally are unacceptable for biosolids land application, in some cases biosolids may be applied to steep slopes in forested areas because of their high permeability and infiltration capacity [48,49]. Moreover, earth excavation (e.g., cut and fill) may be used to transform steep slopes to acceptable grades. The U.S. Geological Survey publishes quadrangle maps that can be used to estimate terrain slope, local depressions or wet areas, rock outcrops, regional drainage patterns, and water table elevations [77].

7.2.4.2 Soil permeability, infiltration, and drainage patterns. *Soil permeability* refers to the ease with which water and air may be transmitted

TABLE 7.4 Typical Slope Limitations for Land Application of Biosolids*

Slope	Considerations
0–3%	Ideal—no concern of soil erosion or runoff of liquid biosolids.
3–6%	Slight risk of soil erosion; however, acceptable for surface application of liquid or dewatered biosolids.
6–12%	Surface application of dewatered biosolids is acceptable; liquid biosolids must be injected under most circumstances.
12–15%	No liquid biosolids should be applied without runoff control; surface application of dewatered biosolids is acceptable, but immediate incorporation is recommended.
Over 15%	Only suitable for sites with high permeability (e.g., forest soils) or where the slope length is long with adequate buffer zone downslope.

*Adapted from ref. [100].

through soil [23]. Fine-textured soils (e.g., clay) are characterized by low permeability, whereas the permeability of coarse-textured soils (e.g., sand) ranges from moderate to extremely rapid [10]. In addition to soil permeability, soil texture also has an impact on moisture infiltration rates, with finer-textured soils exhibiting lower infiltration rates. Moisture infiltration rates are also affected by the soil's salt content, with high salt concentrations causing the disintegration of the soil structure, which, in turn, results in a significant decrease in moisture infiltration rate [10].

To protect surface water quality, drainage patterns must be identified at each potential biosolids land-application site. Excessive overland flow velocities and short-circuiting should be eliminated by engineering controls, if necessary [48,51]. With proper design and operation, biosolids can be applied successfully to almost any type of soil. However, poorly drained soils may present special design challenges. Sites with such conditions generally are given a lower priority during the phase I site-screening investigation. Table 7.5 summarizes the soil factors of importance for biosolids application to agricultural land.

7.2.4.3 Proximity to surface water. The number, size, and type of surface water bodies on or near a potential biosolids land-application site are significant factors in site selection due to the potential contamination from site runoff and/or flood events. Areas that are subjected to frequent flooding should be eliminated from the list of potential biosolids land-application sites. However, engineered flood-control structures can be constructed to protect a biosolids land-application site against flooding if limitations on land availability exist [48,100].

7.2.4.4 Depth to groundwater. In general, the greater the water table depth, the more desirable a site is for biosolids land application (Table 7.6). In addition to the soil depth to groundwater, consideration of the

TABLE 7.5 Soil Factors That Limit Biosolids Application to Agricultural Land*

Soil factors	Degree of limitation		
	Slight	Moderate	Severe
Slope	<6%	6–12%	>12%
Depth to seasonal groundwater	>1.2 m	0.6–1.2 m	< 0.6 m
Depth to bedrock	>1.2 m	0.6–1.2 m	<0.6 m
Permeability of the most restricting layer above one meter (cm/h)	0.24–0.8	0.08–0.24	<0.08 or >2.40
Available water capacity (cm/m)	>2.4 cm	1.2–2.4 cm	<1.2 cm

*Adapted from ref. [100].

TABLE 7.6 Recommended Minimum Depth to Groundwater*

Type of site	Drinking water aquifer†	Nonpotable aquifers‡
Agricultural	1–2 m (3.3–6.6 ft)	0.5 m (1.6 ft)
Forest	2 m (6.6 ft)	0.7 m (2.3 ft)
Reclamation	1–2 m (3.3–6.6 ft)	0.5 m (1.6 ft)

*Adapted from ref. [100].
†Individual states may have other depth to groundwater requirements.
‡Depths are recommended to ensure trafficability of surface, not for groundwater protection.

physical condition of consolidated material above the water table is important for sites where high rates of biosolids application are desirable. For example, the existence of extensive layers of fractured bedrock may allow leachate to move rapidly into groundwater, whereas unfractured bedrock (particularly at shallow depths) will restrict water movement, which may result in groundwater mounding, subsurface lateral flow, or poor drainage [48,98,100]. Therefore, land areas that are located in the vicinity of potable groundwater supplies and are characterized by shallow fractured or unfractured bedrock should be eliminated from further consideration as potential biosolids land-application sites.

7.2.4.5 Climate. Rainfall, temperature, evapotranspiration, and wind speed and direction are important climatic factors affecting biosolids land-application systems. Table 7.7 characterizes the relative impact of humidity and temperature on biosolids land-application system design and operations.

In the United States, regional climatic data can be obtained from the National Oceanic and Atmospheric Administration (NOAA). In addition to regional weather data, airports, universities, military installations, and agricultural and forestry extension services can provide detailed climatic information specific for local areas [48].

TABLE 7.7 Relative Impact of Climatic Regions on Land Application of Biosolids*

	Climatic region		
Impact	Warm/arid	Warm/humid	Cold/humid
Operation time	Year round	Seasonal	Seasonal
Operation cost	Lower	Higher	Higher
Storage requirement	Less	More	More
Salt accumulation	High	Low	Moderate
Leaching potential	Low	High	Moderate
Runoff potential	Low	High	High

*Adapted from ref. [100].

7.2.5 Contact with owners of prospective sites

Once potential biosolids land-application sites are identified through the Phase I site-screening investigation, land ownership should be determined. Contacting landowners prematurely without adequate preparation may result in an initial negative reaction that is difficult to reverse. The individuals involved in making the initial land owner contacts should be knowledgeable about potential program benefits and constraints. Once land availability has been determined, a public information program should be prepared and local political support secured. Ideally, the Phase I site-screening investigation will identify one or more sites that merit a more detailed Phase II site evaluation.

7.3 Phase II Site Evaluation

The Phase II site evaluation involves a more detailed field investigation of those sites identified in the Phase I site-screening investigation as being potentially suitable for biosolids land application. The extent and type of information required for a Phase II site evaluation will vary depending on the following factors:

- Land-application practice being considered (e.g., agricultural, forest, land reclamation, or public access)
- Regulatory requirements
- Completeness and suitability of information on soils, topography, and hydrogeology

Table 7.8 provides a summary of the general and site-specific information required for the various land-application practices.

TABLE 7.8 Site-Specific Information for Biosolids Land Application*

General information	Forest application	Reclamation
a. Property ownership	a. Age of trees	a. Existing vegetation
b. Physical site dimensions	b. Species of trees	b. Type of disturbance
c. Current land use	c. Irrigation	c. Need for grading
d. Future land use	d. Vehicular access	d. Vehicular access
Agricultural application	**Surface/groundwater**	
a. Cropping patterns	a. Location and depth of wells	
b. Typical crop yields	b. Location of surface water	
c. Methods of tillage	c. Drainage patterns	
d. Final use of crops	d. History of flooding	
e. Vehicular access	e. Quality and users of groundwater	

*Adapted from refs. [48,49,100].

Prior to conducting a detailed site evaluation, a preliminary review of site characteristics should be finalized from existing site survey maps and site visits. In the United States, site survey maps can be obtained from the following agencies:

- U.S. Geological Survey
- U.S. Soil Conservation Service
- U.S. Federal Emergency Management Agency (FEMA)
- U.S. Fish and Wildlife Service

A general site base map should be developed from the information drawn from these sources for each site to be evaluated. Because of their various levels of specificity, map information should be verified through site visits. Important map features that should be verified include the following:

- Location of surface waters (including intermittent surface waters)
- Location of groundwater wells
- Location of residences, other buildings, public roads, fence lines, etc.

Once the preliminary review is finalized and the base map has been constructed, the field data necessary to complete the Phase II site evaluation should be determined. Moreover, the required analytical and field procedures (including the quality-assurance/quality-control procedures) should be documented and independently reviewed prior to collection of field data [48,70,100].

The type of data required to complete the Phase II site evaluation will vary depending on the complexity of the site as well as the type of biosolids land-application practice chosen. However, in general, the following site characteristics should be evaluated: (1) topographic limitations, (2) soil characteristics (e.g., texture, pH, CEC, etc.), (3) delineation of flood plains/wetlands, and (4) site hydrogeology. The approach for evaluating each of these site characteristics is described in the following sections.

7.3.1 Topographic limitations

State regulations normally define slope grade limits for biosolids land application (Table 7.4). Permissible slopes will range according to both the type of biosolids application practice (e.g., agricultural, forest, reclamation, or public access) and the method of application (e.g., surface spreading, injection, etc.). Soil slopes that exceed the regulatory limitations should be marked as potentially unsuitable areas before conducting any field verification.

7.3.2 Soil characteristics

Soil survey maps published by the U.S. Soil Conservation Service are the best source of information on soil characteristics. These maps contain important information on the subsurface geologic and hydrogeologic conditions at the site as well as the soil physical and chemical characteristics [10,23,78].

Although published soil surveys provide useful information, they may not be completely adequate for a site-specific evaluation. In many cases, only ranges of values for important soil parameters are provided. Direct field sampling and testing are the only methods of accurately defining soil properties at a potential biosolids land-application site [14].

7.3.3 Delineation of flood plains and wetlands

Land application of biosolids on or near flood plains and wetlands is strongly discouraged because of the potential adverse impact on water quality. Flood plains can be identified easily as low-lying areas adjacent to streams. Accurate delineation of flood plain boundaries requires detailed engineering and hydrologic studies. *Wetlands* are defined as areas that are inundated or saturated by groundwater or surface water at a frequency and duration necessary to promote and sustain the growth of wetland vegetation and typically are characterized as swamps, marshes, or bogs. Accurate wetland delineation is based on the following parameters: (1) site hydrology, (2) types and relative abundance of vegetation, and (3) number and types of rare, endangered, or otherwise protected species and their habitats. In the United States, the standard approach for wetland delineation is described in the *U.S. Army Corps of Engineers Wetlands Delineation Manual* [76].

7.3.4 Site hydrogeology

During the Phase II site evaluation, the field soil survey should evaluate the range of soil permeability for each major soil horizon. In addition to soil permeability, the following hydrogeologic factors should be characterized:

- Depth to groundwater (including seasonal variations)
- Quality of existing groundwater
- Present and future groundwater use
- Existence of perched water tables
- Direction of groundwater flow

Soils with a high permeability (i.e., greater than $1.5 \cdot 10^{-2}$ cm/s) or low permeability (i.e., less than $5.0 \cdot 10^{-5}$ cm/s) should be eliminated from the list of potential areas for biosolids land application. If elimination of such areas places an excessive restriction on the number of potential biosolids land-application sites, it may be necessary to conduct field infiltration and permeability tests to determine whether engineering controls can be implemented to render these soils suitable for use [22,48,59,100].

After completion of the Phase II site-evaluation field survey, the base map should be revised to include any new information concerning topographic limitations, soil characteristics, presence and extent of flood plains/wetlands, and site hydrogeology. Also included on the revised base map should be information describing existing vegetation and cropping patterns in the area as well as available access roads to the site [100].

Once the suitability of the sites is established, it is necessary to estimate the permissible rate of biosolids application. The rate of biosolids land application will affect the design and operation of the biosolids land-application system as well as the total land area requirement.

7.3.5 Biosolids land-application rates

Except for land reclamation, the 40 CFR Part 503 rule requires that biosolids be applied to land at a rate that is equal to or less than the agronomic rate [100]. Determining the agronomic rate for land application of biosolids is one of the key elements for ensuring that groundwater quality is not compromised. In addition to the agronomic rate, it should be noted that when the metal loading at a site approaches the cumulative pollutant loading rate (CPLR) limit, the 40 CFR Part 503 rule requires that the biosolids application rate be modified so that the CPLR is not exceeded [88].

Under most circumstances, estimating the agronomic rate requires conducting a nitrogen mass balance that accounts for (1) the nitrogen levels in soil, (2) the nitrogen needs of the vegetation or crop, (3) the availability of nitrogen in the biosolids, and (4) nitrogen losses (e.g., volatilization). The approach used to estimate the biosolids agronomic rate will depend on the type of land to receive biosolids (i.e., agricultural, forest, reclamation, or public access). Under some circumstances, a phosphorus balance is used in lieu of a nitrogen balance for estimating the agronomic rate. Details describing each approach for estimating the agronomic rate including the use of a phosphorus balance are presented in the following sections.

7.3.6 Biosolids land-application practices

There are basically four types of land-application practices available for biosolids beneficial-use programs. These practices include the application of biosolids to land designated as (1) agricultural, (2) forest, (3) land reclamation, or (4) public access. Potential social and environmental impacts resulting from each biosolids land-application practice should be evaluated in terms of the following factors:

- Estimated cost
- Reliability
- Flexibility
- Land area requirements and availability
- Land-use effects
- Public acceptance
- Regulatory requirements

Comparing the results from each evaluation will reveal the most suitable biosolids land-application practice that fits both the needs of the wastewater treatment facility and local conditions. The flowchart in Fig. 7.2 summarizes the procedure for selecting a biosolids land-application practice. Once the biosolids land-application practice is chosen, the engineer or program planner should conduct a preliminary cost analysis.

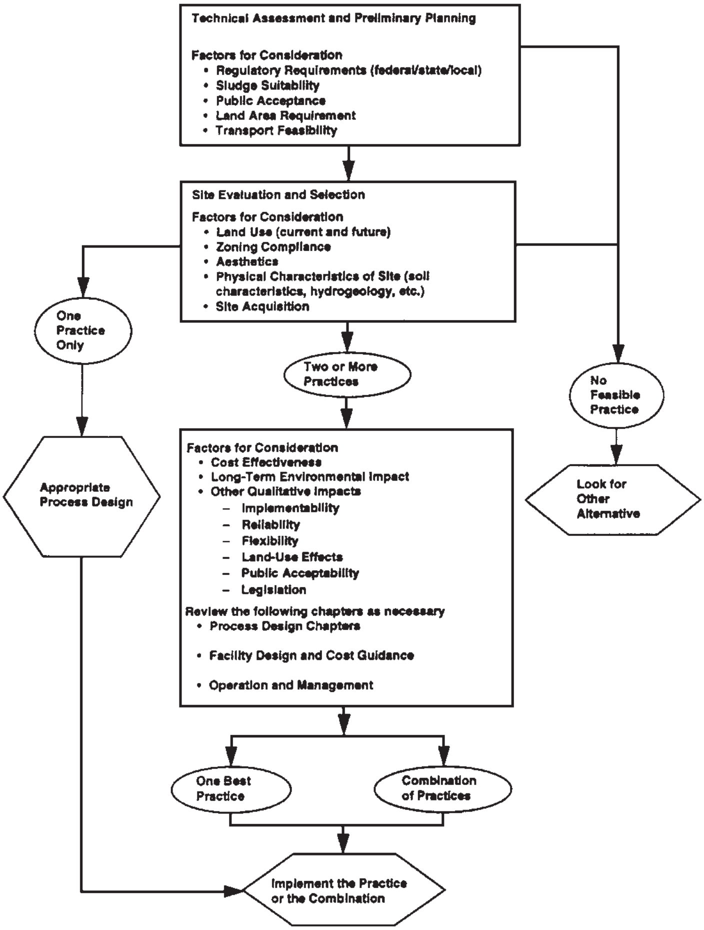


Figure 7.2 Procedures for selection of the land-application treatment practice.

7.3.7 Preliminary cost analysis

A preliminary estimate of relative costs should be made as part of the Phase II site evaluation. Capital costs should be amortized over the design life of the site, whereas operating costs should be estimated annually [83]. Cost factors that must be accounted for during site selection are summarized in Table 7.9.

7.3.8 Final site selection

The final selection of the biosolids land-application site(s) is based on the biosolids land-application practice (agricultural, forest, land reclamation,

TABLE 7.9 Cost Factors Involved in Site Selection*

Capital costs
Land acquisition
Site preparation—drainage, grading, roads, buildings, etc.
Equipment (e.g., transport)
Biosolids storage
Operating and maintenance costs
Fuel
Labor
Equipment repair
Monitoring
Materials and miscellaneous supplies

*Adapted from refs. [78,100].

or public access) and the availability of the best site(s). This approach for identifying a biosolids land-application site is easily applied in the case of a small community with a few potential sites. However, if the site-selection process is complex and involves many potential sites, a weighted scoring system is used [100]. In addition to those technical factors previously described, the following items should be considered in a weighted scoring system:

- Compatibility of biosolids quantity and quality with the specific land-application practice selected
- Public acceptance of both the practice(s) and site(s) selected
- Anticipated design life based on assumed application rate, land availability, metal loadings rate, and soil properties
- Cost analysis

The following sections illustrate the approach used in completing the Phase II site evaluation for each of the biosolids land-application practices. Because of the importance of the biosolids land-application rate on system operation and site life, it is imperative that qualified technical personnel be employed to verify the numerical calculations used to establish the final design specifications [48,100].

7.4 Agricultural Land Application of Biosolids

Agricultural land application is the most widely used biosolids beneficial-use practice (Fig. 7.3). The design approach for agricultural land

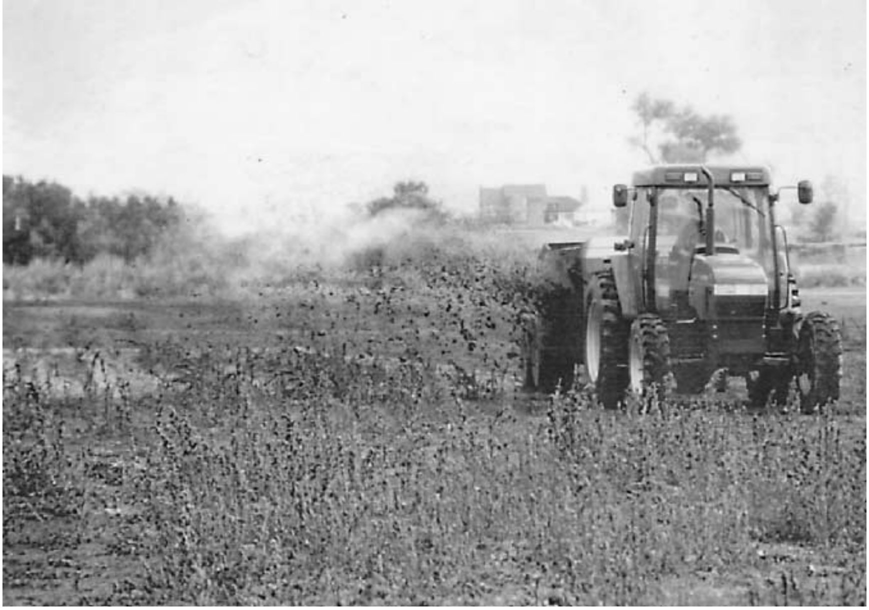


Figure 7.3 Photograph of agricultural crop production using dewatered biosolids. (*Note:* This surface spreading of biosolids is normally followed by incorporation, i.e., plowing.) (*Courtesy of Central Davis Wastewater Treatment Plant, Kaysville, Utah.*)

application emphasizes the fertilizer and soil-conditioning value of biosolids, with the final biosolids land-application rates (tons/acre) being limited by either (1) the nutrient crop requirements or (2) the pollutant concentrations. Before initiating the biosolids land-application design, it is assumed that the engineer/planner has completed the following tasks:

- Received information on biosolids quality and rate of generation from the POTW
- Completed preliminary planning
- Chosen a transportation system to convey the biosolids
- Selected the agricultural land-application method (e.g., injection, spreading, or incorporation) to be used
- Obtained results from soil testing indicating (1) soil texture, (2) nutrient levels, and (3) pH

Once the basic data on biosolids and soil characteristics are known, information regarding crop selection, yields, and fertilizer requirements should be obtained from local agricultural extension agents.

Despite the fact that agricultural land application of biosolids is the most prevalent beneficial-use practice applied today, the net effect of biosolids beneficial use in agriculture is small. For example, if the estimated 5.3 million dry metric tons of biosolids that are currently generated in the United States were to be applied to agricultural land at agronomic rates, they would only be able to satisfy the nitrogen needs of approximately 1.6 percent of the 1250 million hectares (309 million acres) of U.S. cropland (assuming that the average concentration of nitrogen in biosolids is 4 percent and nitrogen is being applied at 100 kg per hectare per year). Moreover, since only about one-quarter of this land is used to grow food for human consumption, of which only 2 percent produces crops that may be consumed fresh, the overall impact of biosolids beneficial use on U.S. agriculture is minimal.

Nevertheless, the local availability of agricultural land, combined with other regional and local concerns, is an important factor to be considered in biosolids management decisions. While many western and midwestern states have ample agricultural land relative to the amount of biosolids generated, land is less available in other regions. For example, in New Jersey, over half the state's cropland would need to receive biosolids application to avoid other forms of disposal or out-of-state transfers, whereas Rhode Island would need essentially all its cropland to satisfy its biosolids disposal needs. The biosolids management decisions for the states of New Jersey and Rhode Island stand in sharp contrast to North Dakota, which would require only 0.05 percent of its agricultural land to receive all the biosolids generated in the state.

In some cases, biosolids are transported considerable distances to reach a suitable land-application site. For example, contractors are currently transporting some of New York City's biosolids to northeast Texas and eastern Colorado for cropland application, while Boston, Massachusetts, transports a significant fraction of its biosolids in the form of heat-dried pellets to Florida for application to cropland and pasture. Some of the biosolids from the Los Angeles Basin are being transported by truck for cropland application in Yuma, Arizona.

Biosolids have value to the farmer for their nutrient content and as a soil conditioner. The USEPA has estimated that the nutritional value of land-applied biosolids is worth approximately \$100 to \$140 per acre [19,22,93,100]. In addition to nitrogen and phosphorus, biosolids contain all other nutrients essential for crop growth, including calcium, iron, magnesium, manganese, potassium, sodium, and zinc. When biosolids are applied according to agronomic rates for nitrogen, all these essential nutrients are present in amounts adequate to meet crop needs.

Soil conditioning refers to the beneficial effects of supplemental organic matter on the structural and/or chemical properties of soil. The land application of biosolids increases the level of organic matter, which, in turn, enhances the structural properties of a soil by binding together soil particles into aggregates and creating large (noncapillary) pores through which air and water can move. In addition to improving soil structural characteristics such as (1) water and air transmission capacity, (2) water infiltration rates, and (3) soil erosion, organic matter addition to soil through biosolids application results in increasing the soil's net negative charge and its cation exchange capacity (CEC). A high CEC in soil is desirable because it lessens or prevents essential nutrient loss by leaching [10,23].

The future market demand for biosolids will depend on the marginal productivity of biosolids, the cost of alternative sources of nutrients or soil amendments, and regulatory/permitting costs. Other economic considerations include the cost of applying biosolids and the additional monitoring, recordkeeping, and management required by federal, state, and local regulations. In most cases, the biosolids land-application contractor and/or POTW typically incur some or all of these costs. To offset some of the monitoring, recordkeeping, and management costs associated with the operations of the biosolids land-application program, the POTW may choose a higher level of biosolids treatment. However, an increase in POTW biosolids treatment costs requires additional financial resources, which are typically provided for by the public in the form of higher sewage rates or, preferably, by an increase in the sale price of biosolids.

7.4.1 Crop selection, yields, and nutrient requirements

To minimize operational difficulties, it is normally advantageous to maintain the same cropping patterns at biosolids agricultural land-application sites as those found in the local community. Understanding these cropping patterns is essential for POTW biosolids management personnel because cropping patterns will affect the scheduling, storage requirements, and methods of biosolids application. Since biosolids applications typically are limited by the plant nutrient requirements, selecting crops with high nutrient demands (e.g., forages, corn, soybeans, etc.) will minimize the amount of land required for agricultural land application.

For all crops, yield potential (e.g., tons per acre) and soil fertility are affected by (1) the amount and distribution of rainfall (or irrigation), (2) soil physical properties (drainage, water-holding capacity, and compaction), and (3) the length of the growing season and incidence of weed, insect, or disease damage [2,27,45,48]. To properly design an agricultural biosolids land-application system, it is essential to obtain reliable local crop yield information. In the United States, an excellent source for this information will be the local agricultural extension agency.

Fertilizer recommendations for crops are based on the nutrient requirements necessary to achieve the desired yield. Fertilizer recommendations for a particular locality are developed using a combination of (1) guidelines based on historical experience with crop yields on different soil types, (2) soil test data, and (3) estimates of residual nutrients from previous application of biosolids, manure, and/or nitrogen-fixing crops [26,46,100]. Typical nitrogen, phosphorus, and potassium requirements for various yields of corn and grain sorghum are provided in Table 7.10. Data similar to those contained in Table 7.10 may be used to estimate the adjusted nutrient requirement of the crop at a given location. The adjusted nutrient requirement of the crop, which is the difference between the crop's nutritional requirements for a given yield and the level of plant-available nitrogen, phosphorus, and potassium remaining in the soil from previous nutrient applications, is an important design parameter for biosolids land-application systems. Example 7.1 illustrates use of crop fertilizer requirements and soil analyses to estimate the nutrient levels that must be supplied by biosolids or commercial fertilizers to achieve a desired crop yield.

Example 7.1 The Walcott Water Reclamation Plant is land applying biosolids to grow corn at its agricultural biosolids beneficial-use site. If the expected crop yield is 10 metric tons of corn per hectare, what level of nutrients must be supplied by the biosolids if the soil analyses indicated the following nutrient levels.

Plant-available nitrogen: 100 kg/ha

Phosphorus: 25 kg/ha

Potassium: 150 kg/ha

solution Estimate the nutrient requirements from Table 7.10: From Table 7.10, a corn yield of 10 metric tons/ha requires 190 kg/ha of plant-available nitrogen. Moreover, a phosphorus level of 25 kg/ha represents a *medium* level of available phosphorus, while 150 kg/ha of potassium represents a *low* potassium level. Therefore, the nutrient levels that must be supplied from land-applied biosolids can be estimated as follows:

Plant-available nitrogen: 90 kg/ha (190 kg/ha − 100 kg/ha)

Phosphorus (P): 67 kg/ha as P_2O_5 (Table 7.10)

Potassium (K): 135 kg/ha as K_2O (Table 7.10)

NOTE: If biosolids do not provide these nutrients at the desired quantity, supplemental nutrients (in the form of chemical fertilizers) must be added. Conversely, if biosolids provide an excess of any of these nutrients, the residual nutrients may be available for the next growing season.

Although Example 7.1 demonstrates the process for estimating supplemental fertilizer requirements from crop nutrient needs and soil testing, in actual biosolids land-application operations, soil testing can be an expensive operational cost [44,77,100]. Therefore, once the initial soil testing of a site is completed, the residual soil nutrient levels after crop harvesting should be estimated to ensure that excessive nutrient levels are not permitted to accumulate within the soil.

TABLE 7.10 Fertilizer Recommendations for Corn and Grain Sorghum*

Crop yield (metric tons/ha)‡	Nitrogen applied (kg/ha)	Recommended phosphorus (as P_2O_5) and potassium (as K_2O) levels (kg/ha)†					
		Fertilizer	Very low§	Low§	Medium§	High§	Very high§
7.4–7.7	134	P_2O_5	113	80	56	33	0
		K_2O	112	78	57	34	0
7.4–8.4	157	P_2O_5	123	90	67	33	0
		K_2O	135	101	67	34	0
8.4–10.1	190	P_2O_5	136	103	67	46	10
		K_2O	169	135	78	45	0
10.1–11.8	224	P_2O_5	146	113	80	56	10
		K_2O	201	157	101	67	0
11.8–13.4	258	P_2O_5	169	136	90	56	10
		K_2O	224	179	135	89	0

*Adapted from ref. [78].

†1.0 kg/ha = 0.89 lb/acre.

‡Metric tons/ha = 1000 kg/ha.

§Soil test levels. Very low, 0–11 kg/ha P, 0–88 kg/ha K; low, 12–22 kg/ha P, 89–165 kg/ha K; medium, 23–33 kg/ha P, 166–230 kg/ha K; high, 34–77 kg/ha P, 231–330 kg/ha K; very high, 78+ kg/ha P, 331+ kg/ha K.

7.4.2 Estimation of residual nutrient levels

When biosolids are applied to soils each year, the nitrogen, phosphorus, and potassium added in previous years that are not utilized by crops will be partially available during the current cropping season. The contribution of residual nitrogen to plant-available nitrogen (PAN) can be significant when biosolids are applied each year. Although the largest percentage of mineralizable organic nitrogen is converted to inorganic nitrogen during the first year that biosolids are applied, the continued decomposition of organic nitrogen in succeeding years can provide a significant portion of the crop's nitrogen requirement. The amount of organic nitrogen mineralized (i.e., conversion of organic nitrogen to inorganic nitrogen) depends on the type of biosolids treatment processes used at the POTW, the ratio of inorganic to organic nitrogen in the biosolids, and the amount of organic nitrogen applied in previous years (Table 7.11). Once the residual nutrient levels are estimated, the biosolids application rate required for achieving a desired crop yield can be estimated.

7.4.3 Biosolids application rates at agricultural sites

The approach for estimating the agricultural biosolids land application rate (i.e., agronomic rate) is based on the nutrient requirements of the specific crop grown at the site, the nutrient levels in the

TABLE 7.11 Estimated Percentages of Organic Nitrogen Mineralized after Biosolids Are Applied to Soil*

	Type of biosolids applied to soil			
	Unstabilized primary and waste activated sludge	Aerobically digested	Anaerobically digested	Composted
Time after biosolids application (years)	K_{min}† (% of N_o)‡	K_{min}† (% of N_o)‡	K_{min}† (% of N_o)‡	K_{min}† (% of N_o)‡
0–1	40	30	20	10
1–2	20	15	10	5
2–3	10	8	5	3
3–4	5	4	3	3
4–5	3	3	3	3
5–6	3	3	3	3
6–7	3	3	3	3
7–8	3	3	3	3
8–9	3	3	3	3
9–10	3	3	3	3

*Adapted from ref. [77].

†K_{min}, annual mineralization rate (percentage).

‡Percentage of organic nitrogen (N_o) mineralized during the given time interval.

biosolids, and the residual nutrient levels in the soil. The only exception to this approach occurs when the metal concentration at the land-application site is approaching the cumulative pollutant loading rate (CPLR) limit. In this case, the biosolids application rate must be modified to ensure that the CPLR is not exceeded [46,100].

Under normal conditions, nitrogen is the nutrient that limits the biosolids application rate. In order to determine the biosolid application rate, the biosolids nitrogen content, which is determined by the nitrate, ammonia, and organic nitrogen concentration, is used to estimate the plant available nitrogen (PAN). PAN, which represents the nitrogen level in soil that is available to crops during the current growing season, is expressed in units of kilograms of nitrogen per hectare (or pounds of nitrogen per acre).

To properly evaluate PAN, both the concentration of the specific nitrogen species in biosolids must be known and their fate in the soil environment. In general, it is assumed that all the ammonia and nitrate present in soil after biosolids land application are available for plant uptake in the present crop-growing year. However, because of ammonia volatility, the amount of ammonia in biosolids is normally significantly greater than what remains in the soil after land application [7,100]. Factors that affect the rate of ammonia volatilization at a biosolids land-application site include (1) ambient temperature, (2) method of biosolids application, (3) length of time biosolids remain on the soil before incorporation, and (4) biosolids/ soil pH [7,8].

Because of its effect on the vapor pressure of ammonia, the higher the ambient temperature, the greater the rate and extent of ammonia (NH_3) volatilization from land-applied biosolids. In areas that are both warm and dry, most of the ammonia in liquid biosolids will be lost to the atmosphere if the biosolids are not incorporated immediately into soil (e.g., tilling). In addition to high ambient temperatures, land application of biosolids to highly alkaline soils (i.e., those with a $\text{pH} > 8.0$) will result in a significant loss of nitrogen through ammonia volatilization [7,8].

In the absence of field data, ammonia losses should be assumed to be 50 percent if liquid biosolids are surface applied and not incorporated immediately into soil and zero if the liquid biosolids are incorporated immedately into soil (i.e., injected or plowed). If dewatered biosolids are applied to the land, it is assumed that there is no loss of ammonia through volatilization. For purposes of calculating PAN, Table 7.12 can serve as a guide for estimating typical ammonia losses during the land application of biosolids.

In contrast to nitrogen loss through ammonia volatilization, mineralization of organic nitrogen to ammonia and/or nitrate increases the PAN during the present growing season as well as in future years [77,100]. Because of its impact on the chemical structure of the

TABLE 7.12 Ammonia Volatilization Losses*

Biosolids and application method	Volatilization factor (K_v)
Liquid, surface applied	0.5 (50% ammonia loss)
Liquid, soil injected	1.0 (no loss)
Dewatered, surface applied	1.0 (no loss)

*Adapted from ref. [100].

organic nitrogen compounds found in biosolids, the type(s) of treatment to which the biosolids are subjected at the POTW affects the rate and extent of organic nitrogen mineralization in soil. In general, the greater the extent of biosolids processing within the wastewater treatment plants, the slower is the nitrogen mineralization rate in soil. Since there is a significant variation in nitrogen mineralization rates of biosolids receiving similar treatment, the USEPA recommends that laboratory-scale incubation studies be conducted to estimate site-specific soil mineralization rates (Fig. 7.4).

Once the nitrate, ammonia, and organic nitrogen content in biosolids are known, the PAN per metric ton of biosolids may be estimated using Eq. (7.1). It should be noted that if the biosolids application rate S (metric ton/ha) is known, the PAN for the first year may be estimated using Eq. (7.2).

PAN per metric ton of biosolids (kg/metric ton)

$$= [\,(\mathrm{NO_3}) + K_v\,(\mathrm{NH_4}) + K_{\min}\,(\mathrm{N_o})\,]\,10 \qquad (7.1)$$

where PAN = plant-available nitrogen from the first year's biosolids application (kg/ha)

$\mathrm{NO_3}$ = percent nitrate nitrogen in the biosolids (as a percent dry solids)

K_v = volatilization factor (normally assumed to be 0.5 for surface-applied liquid biosolids or 1.0 for incorporated liquid biosolids and dewatered biosolids applied in any manner)

$\mathrm{NH_4}$ = percent ammonia nitrogen (as a percent dry solids)

$K_{\min}$ = mineralization factor for organic nitrogen in the biosolids from the first-year application (Table 7.11)

$\mathrm{N_o}$ = percent organic nitrogen in the biosolids (as a percent dry solids)

10 = conversion factor [(1000 kg biosolids/metric ton) · (1 kg nitrogen/100 kg biosolids)]

Although all the nitrate and ammonia in the soil is available for plant uptake in the present growing season, the organic nitrogen can

Figure 7.4 Photograph of a laboratory-scale soil incubation study. (*Courtesy of Cache Environmental Laboratory, P.C.*)

contribute to the nitrogen requirements in subsequent years. Example 7.2 illustrates the use of Eqs. (7.1) and (7.2) in estimating the PAN from the first year's biosolids application as well as for subsequent years.

$$\text{PAN (kg/ha)} = S\,[\,(\text{NO}_3) + K_v\,(\text{NH}_4) + K_{\min}\,(\text{N}_o)\,]\,10 \qquad (7.2)$$

where PAN = plant-available nitrogen from the first year's biosolids application (kg/ha)

S = biosolids application rate (metric ton/ha)

NO_3 = percent nitrate nitrogen in the biosolids (as a percent dry solids)

K_v = volatilization factor (normally assumed to be 0.5 for surface-applied liquid biosolids or 1.0 for incorporated liquid biosolids and dewatered biosolids applied in any manner)

NH_4 = percent ammonia nitrogen (as a percent dry solids)

$K_{\min}$ = mineralization factor for organic nitrogen in the biosolids from the first-year application (Table 7.11)

N_o = percent organic nitrogen in the biosolids (as a percent dry solids)

10 = conversion factor [(1000 kg biosolids/metric ton) · (1 kg nitrogen/100 kg biosolids)]

Example 7.2 The Tororo County Sewer Improvement District is currently land applying 10 metric tons (dry weight) per hectare of anaerobically digested liquid biosolids annually to grow rye grass. Chemical analysis of the biosolids reveals no nitrate, an ammonia concentration of 2.5 percent, and an organic nitrogen content of 1.5 percent (all on a dry-weight basis). Assuming that the liquid biosolids are surface applied, estimate the PAN (kg/ha) from the first-year biosolids application and for an additional 3 years from this one-time application. Note that $K_{\min}$ for the first year is equal to 0.2.

solution

Step 1. The PAN for the first year can be calculated using Eq. (7.2) as follows:

$$\begin{aligned}\text{PAN (kg/ha)} &= S\,[\,(\text{NO}_3) + K_v\,(\text{NH}_4) + K_{\min}\,(\text{N}_o)\,]\,10\\ &= 10\,[\,(0) + 0.5\,(2.5) + 0.2\,(1.5)\,]\,10\\ &= 155.0\text{ kg/ha}\end{aligned}$$

Step 2. To estimate the PAN for subsequent years, it is assumed that the PAN will equal the amount of organic nitrogen mineralized. The mass of organic nitrogen from the one-time biosolids application (N_o) is estimated as follows:

$$\text{N}_o = (0.015)\,(10\text{ metric tons/ha})\,(1000\text{ kg/metric ton}) = 150\text{ kg/ha}$$

The PAN in subsequent years can be estimated using the $K_{\min}$ factors for anaerobically digested biosolids. The $K_{\min}$ factors from Table 7.11 are provided in the following table:

Year	$K_{\min}$
0–1	0.2
1–2	0.1
2–3	0.05

If it is assumed that all organic nitrogen mineralized in 1 year is consumed in that same year, only the remaining organic nitrogen would contribute to the PAN in subsequent years. Therefore, the PAN can be calculated as follows:

$$\begin{aligned}\text{N}_o\text{ mineralized in year 0–1} &= (0.2)\,(150) &&= 30\text{ kg/ha (PAN from N}_o\text{ in year 1)}\\ \text{N}_o\text{ remaining in year 1–2} &= 150 - 30 &&= 120\text{ kg/ha}\\ \text{N}_o\text{ mineralized in year 1–2} &= (0.1)\,(120) &&= 12\text{ kg/ha (PAN available in year 2)}\\ \text{N}_o\text{ remaining in year 2–3} &= 120 - 12 &&= 108\text{ kg/ha}\\ \text{N}_o\text{ mineralized in year 2–3} &= (0.05)\,(108) &&= 5.4\text{ kg/ha (PAN available in year 3)}\end{aligned}$$

NOTE: The PAN for year 1 was estimated to be 155.0 kg/ha, of which 30 kg/ha was contributed by the mineralization of organic nitrogen.

TABLE 7.13 Approach for Estimating the Biosolids Agronomic Rate*

1. Determine the total crop nitrogen requirements for the desired crop at the yield level anticipated.
2. Subtract the residual soil nitrogen from the total nitrogen requirement. Residual nitrogen can include the following sources:
    - Nitrogen left by a previous leguminous crops (e.g., alfalfa)
    - Any nitrogen that has been applied previously or will be applied during the growth of the crop by fertilizers, manure, or other sources
    - Any nitrogen that is anticipated to be added by irrigation water that will be applied during the growth of the crop
3. Determine the adjusted nitrogen requirement by adding any anticipated nitrogen losses due to volatilization, denitrification, immobilization, or chemical fixation of NH_4^+ by clay minerals.
4. Determine the PAN per dry ton of biosolids for the first year of application using Eq. (7.2).
5. Divide the adjusted nitrogen requirement ANR (kg/ha) by the PAN per dry ton (kg/metric ton) of biosolids [Eq. (7.2)] to obtain the agronomic rate (metric ton/ha).

*Adapted from ref. [100].

The biosolids application rate (metric ton/ha) S is referred to as the *agronomic rate* when the nutrient demands of the crop limit the annual application. Table 7.13 summarizes the steps required to estimate the biosolids agronomic rate based on a nitrogen mass balance.

The steps outlined in Table 7.13 for estimation of the biosolids agronomic rate are quantitatively summarized by Eq. (7.3). Use of Eq. (7.3) to estimate the biosolids agronomic rate is illustrated in Example 7.3.

Agronomic rate S (metric tons/ha)

$$= \frac{\text{adjusted nitrogen requirement (kg/ha)}}{\text{PAN per metric ton of biosolids (kg/metric ton)}}$$

$$= \frac{\text{ANR (kg/ha)}}{[\,(NO_3) + K_v\,(NH_4) + K_{\min}\,(N_o)\,]\,10} \tag{7.3}$$

where ANR = adjusted nitrogen requirement (kg/ha); nitrogen demand of crop minus the available nitrogen present in soil

NO_3 = percent nitrate nitrogen in the biosolids (as a percent dry solids)

K_v = volatilization factor (normally assumed to be 0.5 for surface-applied liquid biosolids or 1.0 for incorporated liquid biosolids and dewatered biosolids applied in any manner)

NH_4 = percent ammonia nitrogen (as a percent dry solids)

$K_{\min}$ = mineralization factor for organic nitrogen in the biosolids from the first year application (Table 7.11)
N_o = percent organic nitrogen in the biosolids (as a percent dry solids)
10 = conversion factor [(1000 kg biosolids/metric ton) · (1 kg nitrogen/100 kg biosolids)]

Example 7.3 The Perry County Wastewater Treatment Plant is considering land application of aerobically digested liquid biosolids by direct soil injection (i.e., $K_v = 1.0$) into agricultural cropland. If the adjusted nitrogen fertilizer requirement for the crop is 225 kg/ha, estimate the biosolids agronomic rate S in (1) dry metric tons/ha, (2) wet metric tons/ha, and (3) m^3/ha. The biosolids, which are being stored temporarily in a lagoon located adjacent to the site, were found to have the following average concentrations:

NO_3-N: 0.8 percent (dry basis)
NH_4-N: 1.1 percent (dry basis)
Total nitrogen: 4.0 percent (dry basis)
Percent dry solids: 4.6 percent

Assume that the density of the liquid biosolids is 1000 kg/m^3.

solution

Step 1. Estimate the percentage of organic nitrogen in the generated biosolids:

$$\begin{aligned}\text{Percent organic nitrogen } (N_o) &= \text{total nitrogen} - (\text{nitrate} + \text{ammonia})\\ &= 4.0 \text{ percent} - (0.8 \text{ percent} + 1.1 \text{ percent})\\ &= 2.1 \text{ percent}\end{aligned}$$

Step 2. Estimate the PAN per metric ton of biosolids using Eq. (7.1) given that the mineralization rate ($K_{\min}$) from Table 7.11 is 0.3 for the first year:

$$\begin{aligned}\text{PAN per metric ton (kg/metric ton)} &= [\,(NO_3) + K_v\,(NH_4) + K_{\min}\,(N_o)\,]\,10\\ &= [\,(0.8) + 1.0\,(1.1) + 0.3\,(2.1)\,]\,10\\ &= 25.3 \text{ kg/metric ton}\end{aligned}$$

Step 3. Estimate the agronomic rate S using Eq. (7.3):

Agronomic rate (metric ton/ha)

$$= \frac{\text{adjusted nitrogen requirement (kg/ha)}}{\text{plant-available nitrogen per metric ton (kg/metric ton)}}$$

$$= (225 \text{ kg/ha})/(25.3 \text{ kg/metric ton})$$

$$= 8.9 \text{ metric tons/ha (3.27 tons/acre)}$$

Step 4. Estimate the biosolids application rate in wet metric tons per hectare given a solids content of 4.6 percent:

$$\text{Wet metric tons per hectare} = \frac{8.9 \text{ metric tons (dry) /ha}}{4.6 \text{ metric tons (dry) /100 metric tons (wet)}}$$

$$= 193 \text{ metric tons (wet)/ha (or 70.9 tons/acre)}$$

Step 5. Estimate the biosolids application rate in (m^3/ha).

Application rate (m^3/ha)

$$= 193 \text{ metric tons (wet)/ha} \cdot 1000 \text{ kg/metric ton (wet)} \cdot 1 \text{ m}^3/1000 \text{ kg}$$

$$= 193 \text{ m}^3/\text{ha (20,638 gal/acre)}$$

Although Example 7.3 illustrates the general approach for estimation of the biosolids agronomic rate, all the design equations were presented in SI or metric units. In the United States as well as in other parts of the world, the regulatory authorities often require that the design specifications be presented in standard U.S. or English units. Equations (7.4) and (7.5) provide the basic design relationships for both the PAN and the PAN per ton of biosolids applied in standard U.S. units:

$$\text{PAN per ton (lb N/ton)} = [\,(\text{NO}_3) + K_v\,(\text{NH}_4) + K_{\text{min}}\,(\text{N}_\text{o})\,] \qquad (7.4)$$

$$\text{PAN (lb N/acre)} = S\,[\,(\text{NO}_3) + K_v\,(\text{NH}_4) + K_{\text{min}}\,(\text{N}_\text{o})\,] \qquad (7.5)$$

where PAN = plant available nitrogen from the first year's application (lb/acre)

S = biosolids application (ton/acre)

NO_3 = nitrate content in biosolids (lb/ton) on dry-mass basis

K_v = volatilization factor (normally assumed to be 0.5 for surface-applied liquid biosolids or 1.0 for incorporated liquid biosolids and dewatered biosolids applied in any manner)

NH_4 = ammonia content in biosolids (lb/ton) on dry-mass basis

K_{min} = mineralization factor for organic nitrogen in the biosolids from the first-year application (see Table 7.11)

N_o = organic nitrogen content in biosolids (lb/ton) on dry-mass basis

The biosolids agronomic rate S in standard U.S. units (i.e., tons/acre) can then be determined by dividing the adjusted nitrogen fertilizer requirement by the PAN per ton of biosolids (lb/ton), as described by Eq. (7.6). Use of Eq. (7.6) in estimating the biosolids agronomic rate is illustrated in Example 7.4.

$$\text{Agronomic rate (tons/acre)} = \frac{\text{adjusted nitrogen fertilizer requirement (lb/acre)}}{\text{PAN per ton of biosolids (lb/ton)}}$$

$$= \frac{\text{ANR (lb/acre)}}{(\text{NO}_3) + K_v\ (\text{NH}_4) + K_{\min}\ (\text{N}_o)} \qquad (7.6)$$

Example 7.4 The Poole County Water Renovation plant is presently stabilizing its biosolids through anaerobic digestion. If the liquid biosolids from the anaerobic digester are to be surface applied to agricultural cropland without immediate incorporation into soil (i.e., $K_v = 0.5$), estimate the biosolids agronomic rate in (1) dry tons per acre, (2) wet tons per acre, and (3) gallons per acre. Assume that the adjusted nitrogen fertilizer requirement for the crop (e.g., corn) was estimated to be 140 lb/acre and that the density of the liquid biosolids is 8.34 lb/gal. Chemical analysis of the liquid biosolids provided the following results:

NO_3-N: 1.0 percent (dry basis)

NH_4-N: 1.1 percent (dry basis)

Total nitrogen: 3.4 percent (dry basis)

Total solids: 5.2 percent

solution

Step 1. Estimate the percentage of organic nitrogen in the biosolids:

$$\begin{aligned}\text{Percent organic nitrogen } (\text{N}_o) &= \text{total nitrogen} - (\text{nitrate} + \text{ammonia})\\ &= 3.4 \text{ percent} - (1.0 \text{ percent} + 1.1 \text{ percent})\\ &= 1.3 \text{ percent}\end{aligned}$$

Step 2. Estimate the nitrate, ammonia, and organic nitrogen content in biosolids in units of pounds per dry ton (lb/ton):

$$\text{NO}_3 = 1.0 \text{ lb NO}_3\text{-N/100 lb biosolids)} \cdot 2000 \text{ lb/ton} = 20 \text{ lb/ton}$$

$$\text{NH}_4 = 1.1 \text{ lb NH}_4\text{-N/100 lb biosolids)} \cdot 2000 \text{ lb/ton} = 22 \text{ lb/ton}$$

$$\text{N}_o = 1.3 \text{ lb N}_o\text{/100 lb biosolids)} \cdot 2000 \text{ lb/ton} = 26 \text{ lb/ton}$$

Step 3. Calculate PAN per ton of applied biosolids (dry weight) using Eq. (7.4). Note that the mineralization rate $K_{\min}$ from Table 7.11 is 0.2 for the first year.

$$\begin{aligned}\text{PAN per ton (lb N/ton)} &= (\text{NO}_3) + K_v\ (\text{NH}_4) + K_{\min}\ (\text{N}_o)\\ &= 20 \text{ lb/ton} + 0.5\ (22 \text{ lb/ton}) + 0.2\ (26 \text{ lb/ton})\\ &= 36.2 \text{ lb N/ton}\end{aligned}$$

Step 4. Estimate the agronomic rate (ton/acre) using Eq. (7.6):

$$\text{Agronomic rate (tons/acre)} = \frac{\text{ANR (lb/acre)}}{(\text{NO}_3) + K_v\,(\text{NH}_4) + K_{\text{min}}\,(\text{N}_\text{o})}$$

$$= \frac{140 \text{ lb N/acre}}{36.2 \text{ lb N/ton}} = 3.9 \text{ tons (dry basis)/acre}$$

Step 5. Estimate the agronomic rate in wet tons per acre by noting the fact that the biosolids have a solids content of 5.2 percent:

$$\text{Wet tons per acre} = \frac{3.9 \text{ dry tons/acre}}{5.2 \text{ dry tons/100 wet tons}}$$

$$= 74 \text{ tons (wet)/acre}$$

Step 6. Estimate the agronomic rate in gallons per acre assuming that the density of the liquid biosolids is 8.34 lb/gal:

$$\text{Gallons per acre} = 74 \text{ wet tons/acre} \cdot 2000 \text{ lb/wet ton} \cdot 1 \text{ gal/8.34 lb}$$

$$= 17{,}835 \text{ gal/acre}$$

It should be noted that the agronomic rate S is based exclusively on the nutrient content of biosolids and the nutrient demand of the crop. Although in typical agricultural applications there is normally sufficient soil permeability to readily absorb liquid biosolids, in all cases where liquid biosolids are being land applied, the agronomic rate should be compared with the minimum infiltration rate of the soil to ensure that ponding does not occur. For example, in Example 7.4, the agronomic rate of 17,835 gal/acre is equivalent to approximately 0.66 in of liquid. In the absence of field data, this agronomic rate normally would be considered too large to apply in one application without creating runoff of liquid biosolids from the site. Under normal conditions, 8000 to 10,000 gal/acre (0.30–0.37 in) is the maximum range over which liquid biosolids can be applied safely to a site at one time. To apply liquid biosolids safely at an agronomic rate greater than this requires the use of split applications. For example, to minimize the potential of biosolids runoff in the system described by Example 7.4, 50 percent of the agronomic rate could be applied on one day (i.e., 8918 gal/acre), and the other 50 percent could be applied approximately 5 to 7 days later.

7.4.4 Agronomic rate based on phosphorus

Because of the increased concern over eutrophication, many communities have opted to use the crop's adjusted phosphorus requirements rather than the crop's nitrogen needs to estimate the biosolids agronomic rate [77,100]. Like nitrogen, the adjusted crop phosphorus requirements are determined from soil fertility tests and the anticipated crop yield. In the absence of field data, it is normally assumed that 50 percent of the inorganic phosphorus content of biosolids is available for plant uptake [77]. Given this assumption, the phosphorus-based biosolids agronomic rate may then be determined using Eq. (7.7). Example 7.5 illustrates use of Eq. (7.7) in estimating the biosolids agronomic rate based on the biosolids phosphorus content.

$$\text{Agronomic rate (ton/acre)} = \frac{P_{req}}{\text{crop-available } P_2O_5 \text{ per ton biosolids (dry basis)}} \tag{7.7}$$

where P_{req} = adjusted crop phosphorus fertilizer requirement (lb/acre)

Available P_2O_5 = 0.5 · total lb of P_2O_5 per ton of biosolids (dry basis)

Total lb of P_2O_5 per ton biosolids = lb of phosphorus in biosolids · 2.3 (dry basis)

NOTE: 2.3 is the factor used to convert pounds of phosphorus to pounds of P_2O_5 (mass weight ratio P_2O_5:P = 142:62).

Example 7.5 Due to deterioration of water quality, the Poole County Health Department has required that all biosolids land-application operations base their application rates on the crop's phosphorus nutrient requirements. The Poole County Water Renovation Plant (Example 7.4) is presently employing anaerobic digestion to stabilize its biosolids. If the liquid biosolids from the anaerobic digester contain 2.0 percent phosphorus (dry basis) and the municipality would like to apply the biosolids to agricultural crop land to grow corn (P_{req} = 22 lb P/acre), estimate the agronomic rate in tons (dry) per acre.

solution

Step 1. Estimate the total phosphorus per dry tons of biosolids:

$$\text{lb P/ton (dry)} = 2.0 \text{ lb P/100 lb biosolids} \cdot 2000 \text{ lb/ton} = 40 \text{ lb P/ton (dry)}$$

Step 2. Estimate the total pounds of P_2O_5 per dry ton of biosolids assuming that all phosphorus contained in biosolids is inorganic phosphorus:

$$\text{lb } P_2O_5\text{/ton (dry)} = 40 \text{ lb P/ton (dry)} \cdot 2.3 = 92 \text{ lb } P_2O_5\text{/ton (dry)}$$

Step 3. Estimate plant-available P_2O_5 per dry ton assuming that 50 percent of the inorganic phosphorus is available for plant uptake:

$$\text{Available } P_2O_5 \text{ per dry ton} = 92 \cdot 0.5 = 46 \text{ lb } P_2O_5\text{/dry ton}$$

Step 4. Estimate the agronomic rate S using Eq. (7.7):

$$\text{Agronomic rate (tons/acre)} = \frac{P_{req}}{\text{available } P_2O_5 \text{ per dry ton}}$$

$$= \frac{22 \text{ lb/acre}}{47 \text{ lb } P_2O_5\text{/ton}} = 0.47 \text{ ton/acre}$$

NOTE: In comparing the phosphorus-based agronomic rate with the nitrogen-based agronomic rate (see Example 7.4), it is clear that for nearly all biosolids land-application systems, supplemental nitrogen fertilization will be required to optimize crop yields when application rates are based on crop phosphorus needs (the only exception being legumes).

7.4.5 Biosolids application rate based on pollutant limitations

If proper attention is paid to industrial pretreatment, most municipal biosolids will contain heavy metals at concentrations that do not exceed the 40 CFR Part 503 pollutant concentration limits. In these circumstances, heavy metal concentrations will not be a factor in determining the biosolids application rate. However, when the concentration of heavy metals in biosolids is less than the ceiling limits but greater than the pollutant concentration limits, the cumulative pollutant loading rates (i.e., CPLRs) must be met if the biosolids are disposed in bulk [93,94,99]. To estimate the maximum total quantity of biosolids based on the CPLR, Eq. (7.8) should be used.

$$\text{Maximum biosolids application rate (tons/acre)} = \frac{\text{lb/acre (CPLR)}}{0.002 \cdot \text{ppm of pollutant}} \tag{7.8}$$

where ppm of pollutant = milligrams of heavy metal per kilogram of dry biosolids

CPLR = cumulative pollutant loading rate for each of the nine regulated pollutants (lb/acre; Table 1.4)

0.002 = (lb metal/10^6 lb biosolids) · (2000 lb biosolids/ton biosolids)

After estimating the total quantity of land-applied biosolids based on each of the nine heavy metals regulated by the 40 CFR Part 503 rule, the lowest value should be identified as the maximum quantity of biosolids to be applied on that particular site. To monitor the rate at which metals are being applied to a site when the biosolids application rate is based on the agronomic rate S, Eq. (7.9) can be used:

$$\text{Heavy metal loading (lb/acre)} = \frac{\text{biosolids agronomic rate } S \text{ (tons/acre)}}{0.002 \cdot \text{ppm of pollutant}} \qquad (7.9)$$

where S = biosolids agronomic rate (tons per acre), dry basis
ppm of pollutant = milligrams of heavy metal per kilogram of dry biosolids
0.002 = (lb metal/10^6 lb biosolids) · (2000 lb biosolids/ton biosolids)

A cumulative record of individual biosolids applications as well as metal loadings must be kept for each site receiving biosolids. When the cumulative amount of any one of the nine regulated toxic metals reaches its CPLR, no additional biosolids containing that particular metal can be applied to that site. Example 7.6 illustrates the use of the CPLR in estimating the permissible biosolids application rate.

Example 7.6 Biosolids generated at the Perry County Water Reclamation Facility have the following heavy metal concentrations (dry-weight basis). Given these metal concentrations, estimate the maximum biosolids application rate in tons per acre if it is assumed that the biosolids quality will remain constant for the active life of the land-application site.

Metal	Concentration (dry basis)* (mg/kg)	CPLR† (lb/acre)
Arsenic	21.0	37.0
Cadmium	29.0	35.0
Copper	1100.0	1335.0
Lead	112.0	268.0
Mercury	6.0	15.0
Nickel	169.0	375.0
Selenium	40.0	89.0
Zinc	1890.0	2500.0

*Molybdenum is the ninth toxic metal to be regulated. At present, the USEPA is negotiating the CPLR value for molybdenum.

†Adapted from the 40 CFR Part 503 rule.

solution

Step 1. Estimate the maximum biosolids application rate (tons per acre) for each regulated metal. The following calculation, which is based on the arsenic concentration, illustrates the approach to be used for each of the regulated metals. Note that the CPLR value for each metal was obtained from Table 1.4.

$$\text{Maximum biosolids application (tons/acre)} = \frac{\text{CPLR (lb/acre)}}{0.002 \cdot \text{ppm}}$$

$$= \frac{36.5 \text{ (lb/acre)}}{0.002 \cdot 21 \text{ (mg/kg)}}$$

$$= 881 \text{ tons/acre}$$

Step 2. For each regulated metal, similar calculations as those shown for arsenic are made with the resulting data presented in tabular form:

Metal	Maximum biosolids application rate (tons/acre)
Arsenic	881
Cadmium	603
Copper	607
Lead	1192
Mercury	1250
Nickel	1109
Selenium	1113
Zinc	659

Step 3. From the results in step 2, the limiting pollutant is identified as cadmium. Given the present quality of biosolids, the maximum lifetime application of these biosolids will be 603 tons/acre. Once this biosolids loading has been reached, the CPLR for cadmium will have been attained and no more biosolids can be applied at this site.

7.4.6 Biosolids land-application equipment

The choice of equipment used in agricultural biosolids land application depends on the physical characteristics of the biosolids and soil as well as the types of crops grown. For example, liquid biosolids can be applied by surface spreading or subsurface injection using the following equipment:

- Farm tractors
- Tank wagons
- Tank trucks
- Portable or fixed irrigation systems
- Ridge and furrow irrigation

Surface spreading by tank trucks and applicator vehicles is the most common method used for applying liquid biosolids to agricultural croplands. After biosolids have been applied and allowed to partially dry, they are normally incorporated into the soil by plowing prior to planting (unless minimum or no-till systems are employed).

Although surface spreading of liquid biosolids from a truck-mounted directional application system is sometimes used, traditional spray irrigation systems (e.g., big gun) generally should not be employed to surface apply liquid biosolids. The adherence of biosolids to plant vegetation can have a detrimental effect on crop yields by reducing photosynthesis [22,48,100]. Spray irrigation also tends to increase odors and reduces the aesthetics at the application site.

In addition to surface spreading, liquid biosolids also can be injected below the soil surface (called *subsurface injection*). Available equipment includes tractor-drawn tank wagons with injection shanks and tank trucks fitted with flotation tires and injection shanks (Fig. 7.5). Both types of equipment minimize odor problems and reduce ammonia volatilization by rapid mixing biosolids and soil. Injection can be used either before planting or after harvesting of most crops. Subsurface injection normally is unacceptable for forage or sod production because shanks can damage stands and leave deep ruts in the field [100].

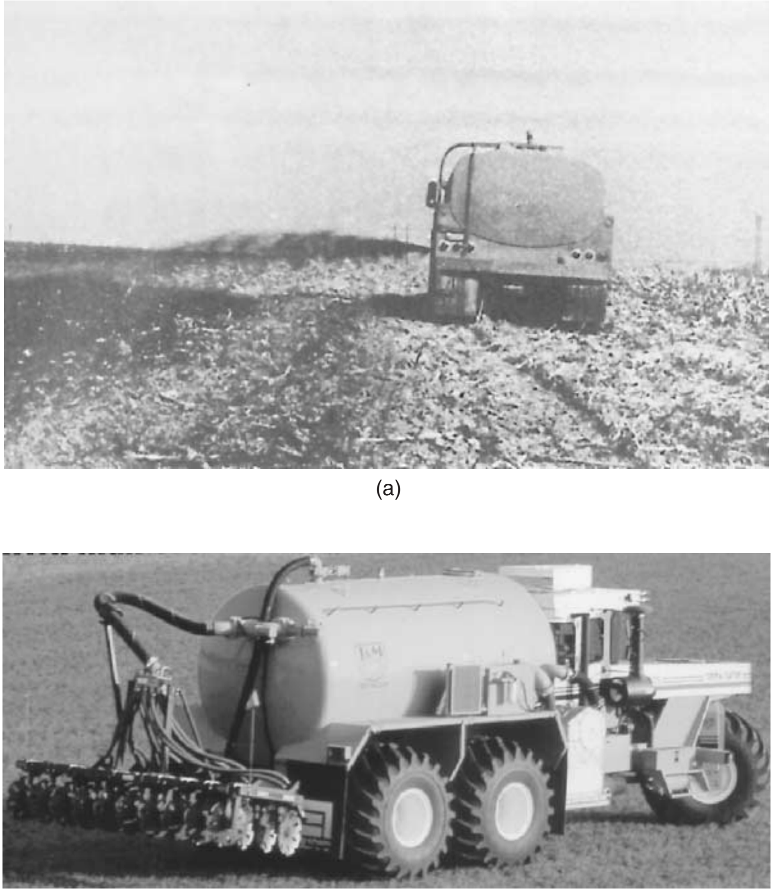
Regardless as to the type of biosolids land-application system, the volumetric application rate of liquid biosolids always should be compared with the soil's infiltration rate. To minimize the potential of surface runoff of biosolids, the application rate should be significantly less than the minimum infiltration rate [100].

Dewatered biosolids may be applied to cropland with the same equipment used to apply animal manures (Fig. 7.6). Typically, the dewatered biosolids will be applied to the surface and then incorporated by plowing or another form of tillage. Incorporation is not practiced where dewatered biosolids are applied to minimal- or no-till land [19].

7.4.7 Land application of biosolids to arid land

In arid regions, evapotranspiration exceeds precipitation, and therefore, scheduled irrigation normally is required for maintaining desired crop yields. As in all irrigation practices, the soluble salt concentration should be monitored. Excessive salt concentrations in soil can (1) lower crop yield, (2) reduce water infiltration rates, and (3) cause soil structural changes that make tilling more difficult [1,13,30–32].

Biosolids application to arid land not only can serve as a source of crop nutrients but also can offset some of the detrimental effects of high salt concentrations. The application of biosolids increases the organic matter content of soil, which, in turn, enhances the soil physical properties such as moisture holding capacity, infiltration, and aeration rates [48,100].


(a)

(b)

Figure 7.5 Equipment used for surface and subsurface injection of liquid biosolids: (*a*) surface spreading equipment; (*b*) subsurface injection. (*Courtesy of Ag-Chem Equipment Company.*)

7.4.8 Land application of biosolids to rangeland

The application of biosolids to rangeland enhances its productivity, improves forage quality, and increases its moisture-holding capacity. An added benefit of land application of biosolids to arid rangelands is that the remoteness of most rangeland sites minimizes public concerns about odors, vectors, and traffic. Various field studies have confirmed the benefits of biosolids application to rangeland (Table 7.14).

A key to successful biosolids application on rangeland is minimizing the disturbance of soil and vegetation. Once the plant cover is dis-

Figure 7.6 Surface application of dewatered biosolids. (*Courtesy of Leland Myers, Central Davis Wastewater Treatment Plant, Kaysville, Utah.*)

turbed, recovery is slow, leaving the rangeland vulnerable to erosion and weed invasion [1,13,57,100].

7.4.9 Scheduling of biosolids land application

The scheduling of biosolids land application must consider the tillage, planting, and harvesting operations for the specific crops grown, climate, and soil properties [19,45,73]. While biosolids should not be land applied during periods of inclement weather (e.g., storms), the 40 CFR Part 503 rule does not prohibit the application of biosolids to flooded, frozen, or snow-covered land. However, biosolids land application under these conditions must not threaten wetlands or surface water quality [100].

Split application of biosolids may be required depending on their moisture content and the minimum infiltration rate of the soil. Unless field data indicate otherwise, agronomic rates above 0.37 acre·in (i.e., 10,000 gal/acre) should be applied using split application. A single application of liquid biosolids at a rate significantly greater than this level could lead to runoff and contamination of surface waters [48].

7.4.10 Biosolids storage

Biosolids storage facilities are required to hold biosolids during periods of inclement weather, equipment breakdown, or when land is unavailable due to crop growth. Liquid biosolids can be stored in digesters, tanks, lagoons, or drying beds, whereas dewatered biosolids can be

TABLE 7.14 Field Studies of Biosolids Application to Rangeland*

Geographic location	Plant community	Results
Wolcott, Colo.	Western wheat grass, alkali blue grass, indian rice grass profile	Increase in species diversity and nitrogen contained in soil
Fort Collins, Colo.	Fringed sage, buffalo grass, western wheat grass	Rapid increase in vegetative growth
Servilleta National Wildlife Refuge, N.Mex.	Blue gamma, hairy gamma	Reduction in runoff due to increased water adsorption and surface roughness
Rio Puerco Watershed, N.Mex.	Blue gamma, snake weed	Twofold increase in forage production

*Adapted from ref. [100].

stockpiled [75]. The minimum storage volume requirements will depend on individual land-application systems and climate [48,100]. In general, volume requirements can be estimated from the following data:

- Biosolids volume and physical characteristics
- Climatic conditions
- Cropping patterns and schedule

Except for forage crops, biosolids application to agricultural land is limited to those months of the year when a crop is not present.

7.4.11 Monitoring requirements

Compliance with the risk-based regulations described in the 40 CFR Part 503 rule for biosolids application to agricultural land reduces the need for monitoring of soils, crops, surface water, and groundwater. Since the basic rationale is to use biosolids as a substitute for commercial fertilizers, monitoring of groundwater at agricultural sites is not usually required provided that the biosolids are not applied at rates greater than the agronomic rate. However, state and local regulatory agencies should be contacted to obtain site-specific monitoring requirements for each individual project.

7.5 Forest Land Biosolids Application

Since nutrients, particularly nitrogen and phosphorus, frequently are limited in forest soils, biosolids application to forests can greatly increase forest productivity. Forest soils are well suited for biosolids application because they have high rates of infiltration, large amounts

Figure 7.7 Biosolids application to forest lands.

of organic material, and perennial root systems (Fig. 7.7). A major advantage of biosolids application to forests over agricultural biosolids application is that forest products have an insignificant impact on the human food chain. In addition, in many regions of the United States, forest land is extensive and provides a low-cost alternative to the use of agricultural cropland.

The primary environmental and public health concern with forest application is contamination of water supplies. Contamination of water supplies by nitrates can be prevented by limiting biosolids application rates to the agronomic rate for desired yields of forest crops. Application of biosolids to forestland is feasible on commercial timber lands, government forests, and privately owned wood lots [36,38–40].

7.5.1 Effect of biosolids application on tree growth and wood properties

The growth response of forests to biosolids applications can vary from 2 to 100 percent for existing stands to over 100 percent for trees planted in soils amended with heavy applications of biosolids (e.g., reclamation site). The magnitude of the response depends on site characteristics and tree stand ages. Some of the most important site characteristics affecting tree growth response to biosolids application include (1) nutrient levels of the site, (2) thinned versus unthinned stands, and (3) specific tree species [78,100]. When applying biosolids to forest sites, seedling survival, wood quality, and pathogen persistence are major concerns for the biosolids land-application design engineer/planner. Because of their importance to the overall biosolids management program, each of these concerns is addressed in the following sections.

7.5.1.1 Seedling survival. Seedling mortality is normally not a concern at forest sites if biosolids are applied at agronomic rates. Moreover, the planting of seedlings can begin as soon as possible after biosolids application to forest soils. Preliminary studies of biosolids application to forest soils have shown a greater seedling growth response to biosolids compared with some commercial fertilizers [36,100]. The magnitude of seedling growth response depends on various site characteristics, including (1) the age of the tree stands, (2) the degree of tree thinning that has occurred, and (3) the identity and concentration of individual seedling species.

7.5.1.2 Wood quality. Accelerated tree growth resulting from biosolids application has the potential to alter basic wood characteristics, including specific gravity, shrinkage, and mechanical properties [77]. Although some research studies have indicated that accelerated tree growth can have beneficial effects on wood quality, other studies have not supported this claim [77]. For example, while recent studies have shown a 10 to 15 percent reduction in density and in the modulus of elasticity of wood from trees grown in soils amended with biosolids, other studies have reported no difference in wood strength properties from trees grown in soil amended with biosolids compared with wood from trees grown with commercial fertilizers [100].

7.5.1.3 Pathogens. Predation by microorganisms present in forest soils is responsible for the rapid reduction of pathogen concentrations following biosolids land application. Pathogen-related concerns involving wind-borne contamination may arise when spray application of liquid biosolids on forest lands is used. Precautions can be taken to minimize human exposure to pathogens during and after spray application such as restricting public access. Generally, aerosols will not travel far in an established forest because of interception by leaves, branches, and trunks of trees.

7.5.2 Effect of biosolids on forest ecosystems

Immediately after the land application of biosolids, a forest site is altered in appearance. However, within 6 months, understory growth is often much more vigorous than before biosolids application. Moreover, the increased understory growth typically is higher in nutrients and can provide a better habitat for wildlife.

The biosolids land-application design engineer/planner typically will have the option of selecting among the following types of forest sites for applying biosolids:

- Sites recently cleared prior to replanting
- Young plantations
- Established forests

The choice of which type of forest site on which to land apply biosolids will depend on various technical and economic factors. The advantages and disadvantages associated with each type of forest site are described in Table 7.15.

7.5.2.2 Biosolids application to clear-cut forests. Recently clear-cut forests are the easiest and most economical site on which to land apply biosolids to forest soils [48]. Because biosolids application takes place prior to tree planting, many of the methods used to apply biosolids in agricultural applications can be employed. Vehicles delivering biosolids from wastewater treatment plants can discharge semisolid biosolids (i.e., biosolids having a moisture content less than 85 percent) directly on the land. The biosolids can then be spread by a bulldozer, followed by incorporation into soil (e.g., disking). Ease of biosolids delivery to forest soils depends on the amount of site preparation (stump removal, residual debris burning, etc.), slope of terrain, soil conditions, and weather [100]. Site preparation and biosolids characteristics are also major factors in choosing an appropriate application method (e.g., spray irrigation, injection, manual spreading, etc.).

While biosolids application is easier to perform on clear-cut forests, these sites may require additional management practices (e.g., weed and rodent control). If application to a clear-cut forest is planned, a program of periodic disking and pesticide application should be considered, while soil injection of liquid biosolids should be employed to minimize rodent problems.

7.5.2.3 Biosolids application to young stands. Land application of biosolids to existing forest stands normally is accomplished by a tanker truck with a mounted spray system. This biosolids land-application method reduces costs by allowing biosolids to be spread over tree tops. Spraying biosolids over tree tops reduces the number and length of transportation trails required to achieve an even dispersion of biosolids within the forest. Because of their extended root system, trees over 5 years old are the best candidates for this type of biosolids application. Finally, timing is critical with this type of biosolids application because adherence of biosolids to foliage can retard tree growth [77,100].

TABLE 7.15 Advantages and Disadvantages of Biosolids Application to Forests*

Biosolids Application to Recently Cleared Forest Sites

Advantages

- Better access for biosolids application equipment.
- Option of incorporating the biosolids into the soil if the site is sufficiently cleared.
- Option of establishing ridge and furrow or boarder flooding biosolids application system.
- Option to select tree species that demonstrate good growth on biosolids-amended sites.
- Often easier to control public access.

Disadvantages

- Seedlings of some tree species show poor survival when planted directly in freshly applied biosolids.
- Seedlings have low nitrogen uptake rates.
- An intensive program of weed control is necessary.
- Intensive browsing by deer and damage by pests (e.g., voles) may require special control measures.

Biosolids Application to Young Forest Plantations (Over 2 Years Old)

Advantages

- Seedlings are established and more tolerant of fresh biosolids applications.
- Weed control is less of a problem than with cleared sites because of established trees and vegetation.
- Nitrogen uptake by trees is rapid.
- Access for biosolids application equipment is usually good.
- Rapid growth response from most deciduous and many coniferous tree seedlings can be expected.

Disadvantages

- Biosolids application by spraying over the canopy is restricted to those periods when trees are dormant.
- Some weed control may be necessary.
- Plant nitrogen uptake rate is less than that of a well-established forest cover.

Biosolids Application to Closed Established Forests (Over 10 Years Old)

Advantages

- Established forest land is often more readily available in sufficient size to sewage treatment plants.
- Established forests are less susceptible to biosolids-induced changes in vegetation (e.g., weed growth).
- Excellent growth response can be expected to result from increased nutrients.
- Biosolids application by spraying can be done under the tree foliage.
- During precipitation, rapid runoff of storm water contaminated with biosolids constituents is unlikely.
- Forest soils usually have high C/N ratios resulting in an excellent capability to immobilize nitrogen.

Disadvantages

- Access to biosolids application vehicles into a mature forests is often difficult.
- Control of public access is more difficult.
- In publicly owned forests, acceleration of growth may not be desirable. In contrast, commercial forest operations desire faster growth of trees.

---

*Adapted from refs. [77,100].

Figure 7.8 Photograph of a traveling-gun biosolids sprayer.

Although liquid biosolids can be applied to forest soils using a sprinkle irrigation system, clogging of nozzles has been the major drawback to this method. Manure spreaders are capable of applying dewatered biosolids that cannot be sprayed. However, a sufficient number of suitable trails must be available to achieve an even distribution of biosolids when a manure spreader or equivalent device is used to land apply dewatered biosolids to forest sites.

7.5.2.4 Biosolids application to mature stands. Application to older stands has the advantage that biosolids can be applied year round. Moreover, since spraying of liquid biosolids takes place under the tree foliage, foliage is unaffected. The types of biosolids application methods for mature stands are similar to those described for young stands. Figure 7.8 depicts a traveling-gun sprayer typically used in liquid biosolids application to forests.

7.5.3 Equipment for biosolids application at forest sites

There are four types of methods for applying biosolids to forests: (1) direct spreading, (2) spray irrigation with either a set system or a traveling gun, (3) spray application by an application vehicle with a spray cannon, and (4) application by a manure-type spreader. A major factor affecting the choice of biosolids land-application equipment or

TABLE 7.16 Comparison of Biosolids Application Methods Used for Forest Soil*

Method	Advantages	Disadvantages
Biosolids spreading and incorporation (range, 10 ft)	Simple to operate Low O&M costs	Site must be cleared
Spray irrigation		
Set irrigation system (range, 30–200 ft)	Simple to operate Low O&M costs	Frequent clogging Can only use with low-percent solids High capital costs
Traveling gun (range 200 ft)	Low O&M costs Simple to operate	Moderate capital costs Frequent clogging
Vehicle-mounted cannon (range, 125 ft)	Can use on any terrain	High O&M costs Moderate capital costs Need special trails
Manure spreader (range, 50–200 ft)	Low capital costs Low O&M costs Only method to spread high-solids material	Trails need to be close together

*Adapted from refs. [77,100].

method is the liquid content of the biosolids. Table 7.16 lists the most prevalent biosolids land-application methods, including their specific advantages and disadvantages.

7.5.4 Determining biosolids application rates for forests

The biosolids application rates for forest sites are based on tree nitrogen requirements. There are significant differences between tree species in their rate of nitrogen uptake. In addition to the difference in nitrogen uptake rates that exists between various species of trees of the same age, there is a large difference between the nitrogen uptake rates of seedlings, vigorously growing trees, and mature trees within the same species. Table 7.17 provides estimates of annual nitrogen uptake by the overstory and understory vegetation of fully established and vigorously growing forest ecosystems. The USEPA recommends that local agricultural extension agents or U.S. Forest Service personnel be consulted for information on forest nutrient requirements in specific geographic regions.

To properly estimate the biosolids application rates to forests, the design engineer/planner must account for the various nitrogen transformation and partitioning mechanisms that affect the availability of nitrogen to vegetation, including (1) nitrogen mineralization, (2) ammonia volatilization, (3) denitrification, (4) uptake by understory, (5) soil immobilization, and (6) nitrogen leaching

TABLE 7.17 Annual Nitrogen Uptake by Forests*

	Tree age (years)	Average annual nitrogen uptake (kg/ha)
Eastern forests		
Mixed hardwoods	40–60	200
Red pine	25	100
Old field with white spruce plantation	15	200
Pioneer succession	5–15	200
Southern forests		
Mixed hardwoods	40–60	280
Southern pine with no understory	20	200
Southern pine with understory	20	260
Lake state forests		
Mixed hardwoods	50	100
Hybrid poplar	5	150
Western forests		
Hybrid poplar	4–5	300
Douglas fir plantation	15–25	200

*Adapted from refs. [77,100].

[40,77,100]. Each of these factors is discussed briefly in the following sections.

7.5.4.1 Nitrogen mineralization. Mineralization occurs when the organic matter in biosolids decomposes releasing ammonia. Table 7.18 gives typical values of organic nitrogen mineralization as well as ammonia volatilization, denitrification, and nitrogen immobilization from biosolids applied to forest soils. Since nitrogen mineralization rates vary significantly depending on the degree of processing at the wastewater treatment plant, the USEPA recommends that laboratory studies be conducted to define the rate and extent of nitrogen mineralization at a particular site [10,77].

7.5.4.2 Ammonia volatilization. The rate of ammonia volatilization in forest soils is significantly less than that observed in biosolids application to agricultural soils because of various factors, including (1) the low pH of forest soils, (2) the low wind speed in forest stands, and (3) the reduced level of radiation that reaches the forest floor. Typical ammonia volatilization rates in forests are provided in Table 7.18. In addition to the factors described previously, the biosolids application method also will affect ammonia volatilization, with soil injection resulting in minimal ammonia losses [7,8].

7.5.4.3 Denitrification. Ammonia not utilized by vegetation or immobilized by soil will be microbially transformed to nitrate. When oxygen

TABLE 7.18 Annual Nitrogen Mineralization and Loss Rates in Forest Soils*

	Range	Typical value
Organic nitrogen mineralization		
Anaerobically digested	20–65%	
Short detention time		40%
Long detention time		20%
Lagooned	10–20%	
Short detention time		20%
Long detention time		10%
Composted	5–50%	
Short detention time		20%
Long detention time (fully cured)		10%
Ammonia volatilization	0–25%	
Open stand		10%
Closed stand		0%
Denitrification	0–25%	
Moist soils		10%
Dry soils		0%
Nitrogen immobilization	0–1000 lb/acre	
First application		
Young stand		100 lb/acre
Old stand		0 lb/acre
Reapplications		0 lb/acre

*Adapted from refs. [77,100].

becomes limiting, nitrate is reduced to elemental nitrogen (N_2) or nitrous oxide (N_2O) and lost to the atmosphere. The rate and extent of this reduction process, called *denitrification,* will depend on soil conditions, including soil moisture content, temperature, and pH [10,44,49]. In warm, moist soils, the denitrification process can result in the removal of approximately 25 percent of the total nitrogen applied [100]. Table 7.18 provides typical ranges of values for estimating nitrogen losses through denitrification.

7.5.4.4 Nitrogen uptake by understory. In general, the uptake of nitrogen by forests can be as large as that of agricultural crops if the system is managed correctly and species are selected that respond to biosolids applications (Fig. 7.9). The amount of vegetative understory will affect the uptake of nitrogen by forests. A dense understory will have large nitrogen uptake rates. In general, the annual nitrogen uptake in forests will vary from 106 to 300 kg/ha (89–267 lb/acre) depending on tree species, tree age, etc.

7.5.4.5 Soil immobilization. *Immobilization* is defined as the transformation of ammonia and nitrate into organic nitrogen by soil microbes or vegetation [10,52,100]. Forest soils are characterized by

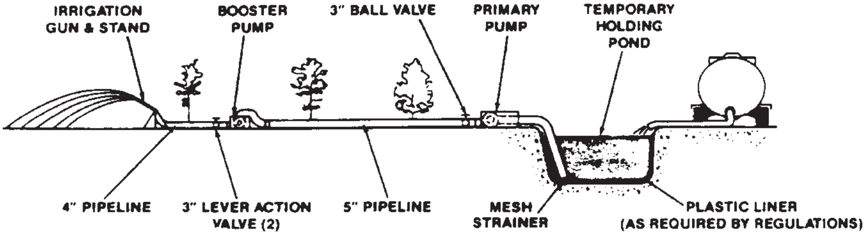


Figure 7.9 Schematic diagram of biosolids application to forest soils.

an extensive organic layer containing decaying litterfall, twigs, and branches. With an abundance of organic carbon, applied nitrogen is used rapidly by soil microorganisms for cellular growth (i.e., microbial protein formation). Nitrogen immobilization through cellular growth represents long-term soil storage of nitrogen that will be released at a very slow rate when the microorganism dies. The biodegradable carbon-to-nitrogen ratio (C:N) of the forest floor and surface soil horizons serves as an indicator of the potential for nitrogen immobilization. Generally, a soil with a C/N ratio greater than 25:1 will result in significant nitrogen immobilization when biosolids are applied.

Overestimation of nitrogen immobilization at forest sites can result in biosolids application rates that significantly exceed tree nitrogen requirements. Consequently, estimates of nitrogen immobilization either should be set conservatively or should be based on documented results from field studies [100].

7.5.4.6 Nitrogen leaching. Nitrogen is normally the limiting constituent in estimating the biosolids land-application rate to forest soils. However, despite the rapid uptake of nitrogen in forest soils, care must be taken not to apply biosolids at a level that exceeds the agronomic rate, since excess applied nitrogen often results in nitrate leaching. Several studies have shown that nitrogen applications substantially above the agronomic rate have resulted in significant deterioration of groundwater quality in the vicinity of biosolids application to forest soils [44,77,100].

7.5.5 Biosolids agronomic rate for forests

The first step in determining the agronomic rate for forests requires that the annual nitrogen requirement for forests N_{req} be estimated. The annual nitrogen requirement of forests N_{req} is the sum of three factors, including (1) nitrogen uptake by trees, (2) nitrogen uptake by understory, and (3) soil immobilization of nitrogen. The sum of these factors is described mathematically by Eq. (7.10).

The nitrogen requirement N_{req} can be met by essentially two nutrient sources, including (1) the nitrogen mineralized from previous biosolids, fertilizer, or manure applications and/or (2) the nitrogen supplied by the current biosolids application.

$$N_{req} = U_{trees} + U_{us} + NI \tag{7.10}$$

where N_{req} = annual nitrogen requirement of forest (kg/ha)
U_{trees} = annual nitrogen uptake by trees (kg/ha)
U_{us} = annual nitrogen uptake by understory (kg/ha)
NI = annual nitrogen immobilization (kg/ha)

Organic nitrogen is mineralized rapidly from biosolids during the first year of forest application. In future years, the rate of mineralization for the remaining organic matter becomes progressively slower [77]. In the absence of field data on mineralization rates, the contribution of organic nitrogen to the plant-available nitrogen (PAN) reserves should be ignored beyond 3 years after application [100]. The PAN supplied from previous years biosolids applications may be estimated using Eq. (7.11).

$$\begin{aligned} N_{prev} = [&(S_1)(N_1)(1 - K_o)(K_1) + (S_2)(N_2)(1 - K_o)(1 - K_1)(K_2)] \\ &+ (S_3)(N_3)(1 - K_o)(1 - K_1)(1 - K_2)(1 - K_3)\,] \cdot 1000 \end{aligned} \tag{7.11}$$

where N_{prev} = PAN from previous biosolids applications (kg/ha)
$S_{1,2,3}$ = biosolids application rate 1, 2, and 3 years ago (metric tons/ha)
$N_{1,2,3}$ = percent organic nitrogen in biosolids applied 1, 2, and 3 years ago (fraction)
K_o = mineralization rate of organic nitrogen during the current year application (Table 7.11)
$K_{1,2,3}$ = mineralization rate of organic nitrogen 1, 2, and 3 years *after* the year of application (Table 7.11)
1000 = 1000 kg/metric ton

The difference between the forest nitrogen requirement N_{req} and the nitrogen available from organic nitrogen mineralized from previous biosolids application N_{prev} is the biosolids nitrogen demand N_{bio}. The biosolids nitrogen demand N_{bio} is described mathematically by Eq. (7.12):

$$N_{bio} = N_{req} - N_{prev} \tag{7.12}$$

where N_{bio} = biosolids nitrogen demand (kg/ha)
N_{req} = forest nitrogen requirement (kg/ha)
N_{prev} = mineralized organic nitrogen available (kg/ha)

In order to calculate the biosolids agronomic rate (metric tons/ha), it is necessary to estimate the PAN per metric ton of biosolids (kg/metric tons). The PAN per metric ton of biosolids can be determined by modifying Eq. (7.2) to account for that portion of nitrogen which is lost through denitrification K_{den}, as illustrated by Eq. (7.13):

$$\text{PAN per ton (kg/metric ton)} = [(\text{NO}_3) + K_v\,(\text{NH}_4) + K_{\text{min}}\,(\text{N}_o)\,]\,(1 - K_{\text{den}})\,10 \qquad (7.13)$$

where NO_3 = percent nitrate nitrogen in biosolids (percent dry solids)
K_v = volatilization factor (normally assumed to be 0.5 for surface-applied liquid biosolids or 1.0 for incorporated liquid biosolids and dewatered biosolids applied in any manner)
NH_4 = percent ammonia nitrogen in biosolids (percent dry solids)
K_{min} = mineralization factor for organic nitrogen in the biosolids from the first-year application (see Table 7.11)
N_o = percent organic nitrogen in the biosolids (as a percent dry solids)
K_{den} = loss of nitrogen by denitrification (fraction)
10 = conversion factor [(1000 kg biosolids/metric ton) · (1 kg nitrogen/100 kg biosolids)]

The agronomic application rate is then determined by dividing the biosolids nitrogen demand N_{bio} by the PAN per metric ton, as illustrated by Eq. (7.14):

$$\text{Agronomic rate (metric ton/ha)} = \frac{\text{N}_{\text{bio}}\ (\text{kg/ha})}{\text{PAN per metric ton (kg/metric ton)}} = \frac{\text{N}_{\text{bio}}\ (\text{kg/ha})}{[\,(\text{NO}_3) + K_v\,(\text{NH}_4) + K_{\text{min}}\,(\text{N}_o)\,]\,(1 - K_{\text{den}})\,10} \qquad (7.14)$$

The biosolids agronomic rate for any given year requires conducting a nitrogen mass balance. Results of the annual nitrogen mass balance will vary depending on whether it is the initial biosolids application and whether subsequent applications are planned. Example 7.7 illustrates the use of the nitrogen mass balance in estimating the agronomic rate of biosolids for land application on forest soils.

Example 7.7 The Poole County Water Renovation Plant is considering land application of anaerobically digested biosolids to forest lands located adjacent to the facility. Because of limited resources, the municipality would like to surface apply liquid biosolids with no subsequent soil incorporation. Assuming that no previous biosolids have been applied at the site and that the forest contains primarily Douglas fir trees with an estimated annual nitrogen requirement N_{req} of 200 kg/ha, estimate the biosolids application rate in metric tons per hectare for the first 3 years of application. Assume that the biosolids will be applied each year and that the denitrification rate (i.e., K_{den}) was determined through laboratory tests to be 0.25. The biosolids chemical analysis (dry-mass basis) is given as follows:

NO_3: 0.5 percent

NH_4: 1.5 percent

Total nitrogen: 5.0 percent

Percent dry solids: 4.2 percent

solution

Step 1. Estimate the percentage of organic nitrogen mineralized during the first 3 years by finding the appropriate mineralization rates. From Table 7.11, K_{min} has the following values for the first 3 years of biosolids application:

Year	K_{min}
0–1	30
1–2	15
2–3	8

Step 2. Estimate the fraction of organic nitrogen (N_o) in the biosolids:

$$\begin{aligned} N_o &= \text{total nitrogen} - (\text{nitrate} + \text{ammonia}) \\ &= 5.0 - (0.5 + 1.5) \\ &= 3.0 \text{ percent (or 0.03)} \end{aligned}$$

Step 3. Estimate the PAN per metric ton of biosolids for the first year using Eq. (7.13). Note that K_v equals 0.5 because the biosolids are not immediately incorporated into soil.

$$\begin{aligned} \text{PAN per ton (kg/metric ton)} &= [\,(NO_3) + K_v\,(NH_4) + K_{min}\,(N_o)\,]\,(\,1 - K_{den})\,10 \\ &= [0.5 + 0.5\,(1.5) + 0.3\,(3.0)\,]\,(1 - 0.25)\,10 \\ &= 16.13 \text{ kg/metric ton} \end{aligned}$$

Step 4. Estimate the biosolids nitrogen demand N_{bio} for the first year using Eq. (7.12):

$$\begin{aligned} N_{bio} &= N_{req} - N_{prev} \\ &= 200 \text{ kg/ha} - 0 \\ &= 200 \text{ kg/ha} \end{aligned}$$

Step 5. Estimate the agronomic rate for the first-year application using Eq. (7.14):

$$\text{Agronomic rate (metric tons/ha)} = \frac{\text{N}_{\text{bio}}\text{ (kg/ha)}}{\text{PAN per metric ton (kg/metric ton)}}$$

$$= \frac{200\text{ kg/ha}}{16.13\text{ kg/metric ton}}$$

$$= 12.4\text{ metric tons/ha}$$

Step 6. Estimate the biosolids application rate for year 2 by accounting for the organic nitrogen that is mineralized during year 1 using Eq. (7.11):

$$\text{N}_{\text{prev}} = [\,(S_1)\,(\text{N}_1)\,(1 - K_{\text{o}})\,(K_1)\,]\,1000$$

$$= [\,(12.4\text{ metric tons/ha})\,(0.03)\,(1 - 0.3)\,(0.15)\,]\,1000$$

$$= 37.1\text{ kg/ha}$$

Step 7. Assuming that the forest nitrogen requirement remains fixed at 200 kg/ha, estimate the biosolids nitrogen demand for the second year using Eq. (7.12):

$$\text{N}_{\text{bio}} = \text{N}_{\text{req}} - \text{N}_{\text{prev}}$$

$$= 200\text{ kg/ha} - 37.1\text{ kg/ha}$$

$$= 160.9\text{ kg/ha}$$

Step 8. Estimate the second-year agronomic rate using Eq. (7.14):

$$\text{Agronomic rate (metric tons/ha)} = \frac{\text{N}_{\text{bio}}\text{ (kg/ha)}}{\text{PAN per metric ton (kg/metric ton)}}$$

$$= \frac{160.9\text{ kg/ha}}{16.13\text{ kg/metric ton}}$$

$$= 10.0\text{ metric tons/ha}$$

Step 9. Estimate the biosolids application rate for year 3 by accounting for the organic nitrogen mineralized from the applications during year 1 and year 2 N_{prev} using Eq. (7.11):

$$\text{N}_{\text{prev}} = [\,(S_1)\,(\text{N}_1)\,(1 - K_{\text{o}})\,(K_1) + (S_2)\,(\text{N}_2)\,(1 - K_{\text{o}})\,(1 - K_1)\,(K_2)\,]\,1000$$

$$= [\,(12.4\text{ metric tons/ha})\,(0.03)\,(1 - 0.3)\,(0.15)$$

$$+ (10\text{ metric tons/ha})\,(0.03)\,(1 - 0.3)\,(1 - 0.15)\,(0.08)\,]\,1000$$

$$= 53.3\text{ kg/ha}$$

Step 10. Assuming that the forest nitrogen requirement remains fixed at 200 kg/ha, estimate the biosolids nitrogen demand for the third year using Eq. (7.12):

$$\begin{aligned} N_{bio} &= N_{req} - N_{prev} \\ &= 220 \text{ kg/ha} - 53.3 \text{ kg/ha} \\ &= 147.7 \text{ kg/ha} \end{aligned}$$

Step 11. Estimate the third-year agronomic rate using Eq. (7.14):

$$\begin{aligned} \text{Agronomic rate (metric tons/ha)} &= \frac{N_{bio} \text{ (kg/ha)}}{\text{PAN per metric ton (kg/metric ton)}} \\ &= \frac{147.7 \text{ kg/ha}}{16.13 \text{ kg/metric ton}} \\ &= 9.2 \text{ metric tons/ha} \end{aligned}$$

Step 12. The agronomic rates for the first 3 years of biosolids application are summarized in the following table:

Year	Agronomic rate (metric tons/ha)
1	12.4
2	10.1
3	9.2

7.5.6 Scheduling

Biosolids application to forests can be made either annually or once every several years. Annual applications are designed to provide nitrogen only for the annual uptake requirements of the trees, considering ammonia volatilization and denitrification losses as well as nitrogen mineralization from current and prior application years. A periodic application (i.e., once every 3 to 5 years) is larger and takes advantage of the capacity of forest soils to temporarily immobilize nitrogen that will become available in later years.

Scheduling of biosolids application to forests requires consideration of climatic conditions and the age of the forest. High rainfall and/or freezing conditions can limit biosolids application. For application of liquid biosolids, if the total depth of an application is greater than approximately 0.25 acre·in (i.e., 6800 gal/acre), a series of three or more split applications should be made rather than one heavy application [48]. This biosolids land-application scheduling practice allows more time for stabilization to occur and is important for maintaining soil infiltration rate and controlling runoff [100]. The period between split applications typically will range from 2 to 14 days depending on weather conditions.

7.6 Biosolids Use for Land Reclamation

Land reclamation refers to the revegetation of disturbed or marginal lands for the purpose of meeting regulatory compliance and/or improve-

Figure 7.10 Land reclamation of a strip mine site. (*Courtesy of Dr. William Sopper.*)

ment of aesthetics and property values. Extensive areas of disturbed land that can benefit from reclamation exist throughout the United States as a result of mining clay, gravel, sand, stone, phosphate, coal, and other minerals (Fig. 7.10). In addition, construction areas (e.g., roadway cuts, borrow pits) and areas where dredge spoils or fly ash have been deposited are potential sites suitable for land reclamation. Other potential reclamation sites include clear-cut and burned forests, shifting sand dunes, landfills, and sites devastated by toxic fumes [70].

In general, disturbed lands are difficult to revegetate. Most disturbed lands provide a harsh environment for seed germination and subsequent plant growth. Soils at disturbed sites vary in quality and often are characterized by a lack of nutrients and organic matter, low pH, low water-holding capacity, low rates of water infiltration and permeability, poor physical properties, and the presence of toxic levels of trace metals. To correct these conditions, large amounts of lime, soil conditioners, and chemical fertilizers normally are applied on these sites at considerable expense to the landowner. Land application of biosolids to such sites offers a low-cost alternative to the purchase and application of commercial chemicals.

Research has shown that extensive plant cover can be established on many types of disturbed lands using biosolids as a fertilizer and/or soil conditioner [61,70]. The advantages and disadvantages of using biosolids as opposed to chemical fertilizers to revegetate disturbed and marginal land are summarized in Table 7.19.

7.6.1 Biosolids application rates at reclamation sites

The 40 CFR Part 503 rule states that, in general, the rate at which biosolids are applied to land should not exceed the agronomic rate. However, at reclamation sites, higher biosolids application rates are

TABLE 7.19 Advantages and Disadvantages of Using Biosolids in Land Reclamation*

Advantages

1. Vegetation will improve soil drainage.
2. Vegetation will enhance water removal through evapotranspiration.
3. Vegetation will reduce surface runoff volume from precipitation.
4. Vegetation will take up a portion of the nutrients and other biosolids constituents.
5. pH buffering capacity of biosolids is beneficial for reclaiming highly acidic or alkaline sites.

Disadvantages

1. Planting, cultivation, and harvesting of vegetation can be labor- and equipment-intensive.
2. Vegetated areas attract animals that could become a nuisance or serve as vectors.
3. Vegetated areas may result in more unauthorized public entry.

*Adapted from ref. [100].

allowed if approved by the permitting authority [70,77,100]. When determining the appropriate biosolids application rate, a general distinction must be drawn between *reconstructed* and *abandoned* reclamation sites.

Reconstructed reclamation sites generally include coal mine reclamation sites that have been or are being reclaimed according to the requirements of the 1977 Federal Surface Mining Reclamation and Control Act (PL 95-87) and surface mine reclamation sites involving no-coal minerals (e.g., iron and copper) that require a measure of soil reconstruction after the mineral has been removed [70]. Specific soil reconstruction measures required under the 1977 Federal Surface Mining Reclamation and Control Act include (1) grading to reestablish the approximate contour of the land, (2) saving and replacement of topsoil on all areas affected by mining, and (3) additional soil reconstruction of prime farmlands [53,100].

Abandoned reclamation sites typically are abandoned coal mine sites where disturbance occurred prior to the enactment of PL 95-87 and where natural revegetation has been sparse. Other mine sites where mining practices or unfavorable overburden chemistry has resulted in poor vegetation establishment can be considered abandoned reclamation sites [67,70].

Generally, application of biosolids at rates in excess of the agronomic rate is not justified at reconstructed reclamation sites. The approach for determining the biosolids application rates at reconstruction sites should be identical to that described for biosolids application to agricultural or forested sites [100]. An exception to this approach may be used in situations in which the topsoil is thin or missing prior to the initiation of mining activities.

A large one-time biosolids application that exceeds the agronomic rate normally is justified at abandoned sites, such as abandoned acid stripmine spoils. At such sites, the groundwater quality usually is severely degraded, and therefore, the long-term benefits of the large addition of biosolids to the mine spoils for establishing an improved vegetative cover exceed the short-term effects of leaching nitrate to groundwater.

Finally, lands to be reclaimed can differ significantly in their physical, hydrological, biological, and chemical characteristics. These differences are the result of variations in specific mining operations, climate, soil and geologic factors, etc. The first step in evaluating a biosolids land reclamation project is to establish the background environmental conditions at the site (including both the soil and groundwater characteristics) through a detailed site investigation.

7.6.2 Site investigation

Prior to choosing land application of biosolids to reclaim disturbed or marginal land, it is presumed that a reasonable level of preliminary planning has been conducted, a biosolids transportation system has been selected, and potential reclamation sites have been identified that are available within a reasonable distance from the POTW [53,100]. Once the results from the preliminary planning stage indicate that land reclamation is a technically feasible biosolids management option, a detailed site investigation should be conducted. The site investigation should document specific site characteristics, including (1) topography, (2) surface and groundwater flow characteristics and quality, and (3) soil chemical, biological, and physical properties. Each of these site characteristics is described briefly in the following sections.

7.6.2.1 Topography. There may be areas at a disturbed or marginal land reclamation site that, due to physical, hydrological, or chemical characteristics, are unsuitable for biosolids land application. Only areas found suitable for biosolids application should be surveyed and boundaries staked. An accurate topographic contour map of the site area should be developed to provide the basis for (1) delineating the areas with slopes that are too steep for biosolids application, (2) regrading, and (3) designing surface runoff water improvements (e.g., ditches, terraces, berms, etc.). Since available soil survey and topographic maps (e.g., soil conservation maps) only provide general details regarding site topography, it is imperative that onsite physical characteristics be measured. Site-specific data should be used to develop new site survey maps or to upgrade existing ones.

7.6.2.2 Groundwater and surface water characteristics. In addition to the surface contour evaluation, the site investigation should provide information regarding site-specific groundwater characteristics, including

- Depth to groundwater, including seasonal variations
- Quality of existing groundwater
- Present and potential future use of groundwater
- Existence of perched water
- Direction of groundwater flow

It should be noted that, in many instances, the groundwater quality at disturbed sites is so poor that it can be exempted from nondegradation regulations [77,100]. Since water pollution concerns, such as acid drainage, have been associated with mining activity, it is necessary to document both the surface water and groundwater quality prior to biosolids application.

7.6.2.3 Soil characteristics. To minimize any adverse environmental impact due to biosolids land application, the soil characteristics at potential reclamation sites should be determined prior to initiating reclamation activities. Soil characteristics are evaluated through the implementation of a soil-sampling program. Although the level of complexity of the soil-sampling program will vary depending on site conditions and state regulatory requirements, at a minimum, the sampling program should establish the following site characteristics:

- Soil pH, metal concentrations, and nutrient levels
- Quantity of supplemental fertilizer, lime, or other required soil amendment
- Infiltration and permeability characteristics of the soil

When considering the impact of soil conditions on vegetation survival, it should be noted that most grasses and legumes, as well as many shrubs and deciduous trees, grow better in soils with pH values that range from 5.5 to 7.5. Therefore, for sites that have been altered through extensive industrial operations (e.g., mining), pH adjustments usually are necessary prior to biosolids application. In fact, several states have adopted regulations that require that the soil pH be adjusted to at least 6.0 during the first year of biosolids application at reclamation sites [70]. The goal of these local regulations is to minimize the mobility of heavy metals prior to and after biosolids application [100].

Lime is often used for pH adjustment at reclamation sites, but other materials also can be employed. Recommendations of materials to use for pH adjustment may be obtained from a qualified environmental laboratory. However, it should be noted that in some cases standard soil tests significantly underestimate the lime requirement for sulfide-containing disturbed lands. In addition to the problems associated with sulfide-containing soils, the application of biosolids to disturbed lands may cause further acidification if managed improperly. Because of the potential mobility of toxic metals as a result of biosolids application, certified professional soil chemists and/or licensed professional engineers should be employed to analyze results of soil tests.

In addition to documenting soil characteristics, a survey of mining and other industrial debris left at the site should be conducted. For effective reclamation of disturbed land, the site may require the removal of debris remaining from mining, construction, or other operations previously conducted at the site. The extent to which debris must be removed depends on the postreclamation use. For example, if agricultural activities are planned, the top 24 in (60 cm) should be relatively free from any foreign material. On the other hand, if the site is to be revegetated for erosion control, debris should be removed from at least the top 12 in (30 cm) of soil [100].

Finally, surface runoff and soil erosion from the reclamation site should be controlled. Materials that often are used to minimize erosion include specially designed erosion-control blankets, filter fences, straw bales, and mulch. In addition to the use of these materials, it also may be necessary to construct diversion terraces and/or sedimentation ponds to minimize the potential impact of surface runoff/soil erosion on the environment.

7.6.3 Vegetation selection and management

Many species of plants are suitable for use at land reclamation sites. However, each site should be considered unique, and therefore, plant species should be chosen carefully. In general, plant species should be selected for their ability to grow under drought conditions and their tolerance for either acidic or alkaline soil conditions as well as high salt concentrations [70,71].

An important consideration in selecting the type of vegetation to be established at the reclamation site is the postreclamation land use. For example, if the site is to be managed for agriculture or forest production, the vegetation grown will be harvested and removed from the site. Under these conditions, supplemental nitrogen applications may be required to maintain adequate productivity. However, if the vegetation is not harvested, most of the nitrogen will remain on the site and be recycled through natural biotransformation processes [48]. At reclamation sites where the vegetation is not harvested, a one-time biosolids application normally is adequate. Agricultural extension agencies should be consulted for recommendations of appropriate plant varieties and establishment techniques [100].

When developing a suitable mixture of plant species for use at a reclamation site, it is important to consider species that are not only compatible but also grow well when biosolids are used as a fertilizer/soil conditioner. For example, if the goal of the reclamation effort is to establish a vegetative cover sufficient to prevent erosion, a perennial grass and legume mixture is preferred. A combined grass and legume seeding mixture allows the grass species to germinate rapidly, thus providing a complete protective cover within the first year after biosolids land application. Moreover, minimizing soil erosion with grass permits the legume species to gradually develop into the final vegetative cover [62,74]. An added benefit of the grass-legume mixture is that the grass will assimilate a large amount of the nitrogen from the biosolids, preventing it from leaching into the groundwater, whereas the legume species can fix nitrogen, making additional biosolids applications unnecessary [100].

If the reclamation site is to be reforested, it is still generally desirable to seed it initially with a mixture of grasses and legumes. The initial grass and legume cover protects the site from erosion and surface runoff while tree species are becoming established [48]. Planting slow-growing tree species at a reclamation site is not recommended because competition from the herbaceous vegetation hinders the growth of young tree stands. In contrast, fast-growing hardwoods seem to survive and grow well at reclamation sites because they usually can compete successfully with other vegetation. Suitable hardwood species for use at land reclamation sites include black locust, hybrid poplar, European alder, and European larch [100].

7.6.3.1 Seeding and mulching. Herbaceous species can be seeded at reclamation sites by broadcast, hydro, or aerial seeding [77]. Although other seeding methods (e.g., drill seeding) can be used at agricultural sites, disturbed land is often too rocky and irregular for these seeding methods [70]. Of all the practical seeding methods available, broadcast seeding is generally preferred at reclamation sites. Broadcast seeding normally results in a vegetative stand that is more (1) natural in appearance, (2) uniform and complete, and (3) effective in erosion control and site stabilization. Moreover, with broadcast seeding, it is generally unnecessary to cover the seed, since the first rainfall will push the seed into the loosened surface soil and result in adequate vegetative coverage. On sites that have adequate topsoil, agricultural seeding rates can be used at the reclamation site. However, on sites with limited topsoil, it may be necessary to apply seeds at a significantly greater rate [70,100].

Mulching involves applying organic or inorganic materials to the soil surface to protect the seed, reduce erosion, modify extremes in surface soil temperatures, and reduce evaporation. Materials used for mulching include straw, hay, peanut hulls, corncobs, sawdust, bagasse, bark, leaves, and woodchips. Mulching generally is advisable on steep slopes and on black anthracite refuse or fly-ash banks to protect germinating vegetation [70,100].

7.6.4 Grading

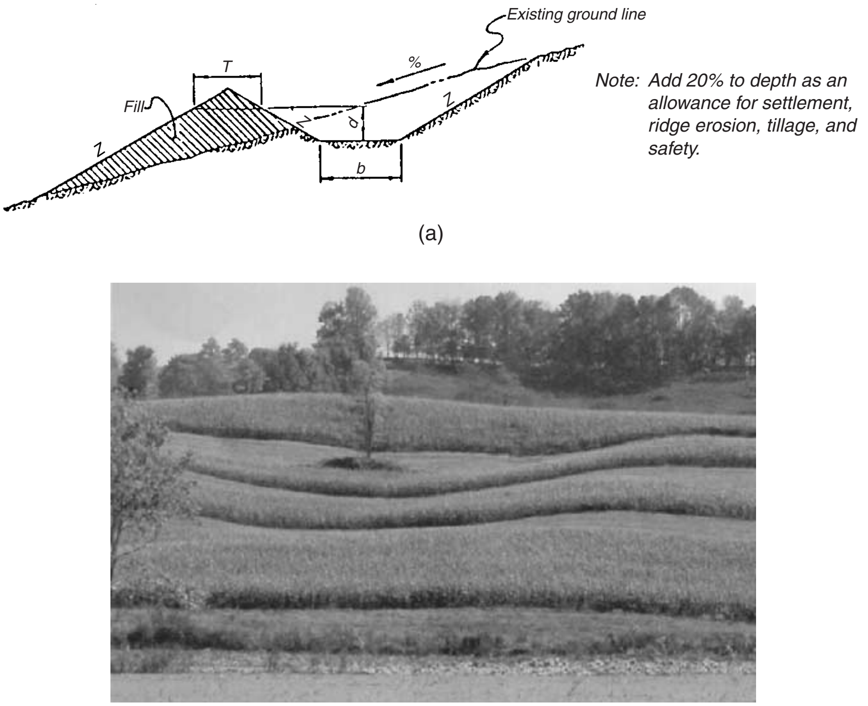
The purpose of establishing surface grades is to ensure that runoff water and/or liquid biosolids do not pond. The initial emphasis in establishing a suitable grade involves filling depressions with soil obtained from adjoining ridges and mounds. If an excessive amount of filling is required for low places, or if sufficient soil is not readily available, surface drains (e.g., ditches) can be constructed. In areas with negligible slopes, grades can be established by increasing slopes between parallel ditch drains with cuts from the edge of one ditch drain and fills from the next [70,100]. Terraces may be needed to protect lower lands from surface flows. Terraces generally are prepared across a slope or at the toe of a slope with the borrow material diked on the lower side for efficient use of the material [100]. Diversion terraces generally are graded and grass covered to minimize erosion and to control flow to the discharge point (Fig. 7.11).

Prior to biosolids application, the soil surface should be roughened to offset compaction caused during the leveling or grading operation (a process called *scarification*). Scarification of surface soil improves both surface water infiltration and permeability while slowing the movement of any surface runoff and erosion. A heavy mining disk or chisel plow typically is necessary to roughen the soil surface.

7.6.5 Biosolids storage

Although liquid biosolids can be stored adequately at the treatment plant in digesters, holding tanks, or lagoons, biosolids storage is sometimes required at reclamation sites. For example, at sites where large volumes of liquid biosolids are to be applied, storage lagoons normally are constructed at the site [48,75]. Similarly, in certain circumstances it may be necessary to store (i.e., stockpile) dewatered biosolids at the reclamation site prior to land application. Stockpiling of dewatered biosolids allows larger quantities to be applied in a relatively short period of time while permitting more efficient use of personnel and equipment [48].

A simple approach to determining the minimum biosolids storage volume necessary for a land reclamation program involves estimating



Note: Add 20% to depth as an allowance for settlement, ridge erosion, tillage, and safety.

Figure 7.11 (*a*) Schematic diagram of diversion terrace construction. (*b*) Photograph of agricultural production using diversion terrace soil conservation practices.

the biosolids generation rate and number of storage days necessary. The minimum number of storage days required, in turn, is based on climate and scheduling considerations plus a safety factor. It should be noted that the responsible regulatory agency often will stipulate the minimum number of storage days that must be provided. Example 7.8 illustrates a simplified approach to determining the minimum biosolids storage volume.

Example 7.8 The Kisumu County Wastewater Treatment Plant desires to land apply liquid biosolids from its anaerobic digesters to reclaim marginal rangeland for agricultural production. If the liquid biosolids will be stored initially in a lagoon constructed adjacent to the agricultural fields, estimate the minimum storage volume (in cubic meters) required for the lagoon if the following design data have been furnished by the facility:

1. Average biosolids generation rate is 589 kg/day (1300 lb/day), dry-mass basis.
2. Liquid biosolids contain 5 percent solids (on average).
3. One hundred days' liquid storage is required by the regulatory agency.

4. Safety factor is 30 percent (1.3).
5. Density of biosolids is 1000 kg/m^3.

solution

Step 1. Estimate the biosolids generation rate in kilograms per day (wet basis):

Biosolids generation (kg/day, wet basis)

$$= \frac{589 \text{ kg dry solids/day}}{0.05 \text{ kg dry solids/kg biosolids, wet basis}}$$

$$= 11{,}778 \text{ kg biosolids/day (wet basis)}$$

Step 2. Estimate the biosolids volumetric generation rate in cubic meters per day:

Biosolids generation rate (m^3/day)

$$= \frac{\text{biosolids generation rate (kg/day), wet basis}}{\text{liquid density (kg/m}^3\text{)}} = \frac{11{,}778 \text{ kg biosolids/day}}{1000 \text{ kg/m}^3}$$

$$= 11.778 \text{ m}^3\text{/day}$$

Step 3. Estimate the minimum biosolids storage volume required by multiplying the required storage day capacity by the given safety factor:

Minimum storage capacity (m^3)

$$= \text{biosolids generation rate (m}^3\text{/day)} \cdot \text{storage days required} \cdot \text{safety factor}$$

$$= 11.778 \text{ m}^3\text{/day} \cdot 100 \text{ days} \cdot 1.3$$

$$= 1531.1 \text{ m}^3 \text{ (use a design capacity of 1600 m}^3\text{)}$$

A more sophisticated method of calculating the minimum biosolids storage capacity involves preparation of a mass flow diagram of cumulative biosolids generation and projected biosolid application to the land-application site [75]. Regardless of the approach used to estimate the biosolids storage volume requirements, the project design engineer should increase the minimum storage required by a safety factor of 20 to 50 percent to cover years with unusual weather and other contingency factors. Finally, the following factors should be considered when siting a biosolids storage facility:

- Maximize the use of storage in the existing sewage treatment plant unit operations. It is often possible to obtain several weeks of storage capacity if the plant has an aerobic or anaerobic digestion unit.

- If possible, locate long-term biosolids storage facilities at the POTW because of the proximity of operating personnel, ease of vandalism control, and possibility of biosolids volume reduction to occur during storage.
- Since occasional odor problems should be anticipated, the biosolids storage facility should be located as far as possible from residential and other public-access areas.
- To reduce operational costs, minimize the number of times the biosolids must be handled, e.g., transferred, stored, etc.

7.6.6 Scheduling

To take advantage of the biosolids nutrient value, land application of biosolids should be scheduled to accommodate the growing season of the selected plant species. If the area to receive biosolids is regulated under federal or state mining statutes, the biosolids application must be scheduled to comply with the revegetation regulations. For example, in Pennsylvania, mined land can be seeded in the spring as soon as the ground is workable (normally early March), but seeding must terminate by May 15 [70,100].

Regardless of the specific reclamation approach employed, biosolids should never be applied to land during periods of heavy rainfall or in periods of prolonged extreme heat or dry conditions. If soil conditions are too wet, the soil structure may be damaged and infiltration decreased due to vehicle traffic. Similarly, if land application of biosolids occurs during extremely dry or hot conditions, substantial losses of nitrogen should be anticipated because nutrient uptake by vegetation under these circumstances will be minimal.

7.6.7 Reclamation of mining land

The reclamation of inactive mining land presents unique challenges to the biosolids land-application design engineer/planner. Specifically, reclaiming mining land requires compliance with federal mining regulations, including 40 CFR Parts 816 and 817 and the 1977 Federal Surface Mining Control and Reclamation Act (PL 95-87) and its subsequent amendments.

To comply with the federal mining regulations, mining companies must submit a postmining land reclamation plan to the appropriate federal agency prior to initiation of any mining activities [70]. The land reclamation plan must contain a detailed description of the postmining use of the land, including the approach for establishing a diverse, effective, and permanent vegetative cover.

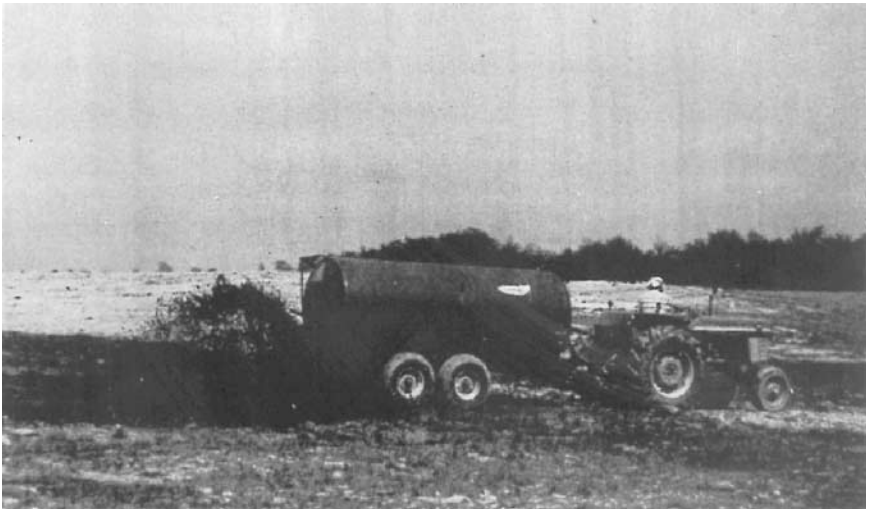


Figure 7.12 Application of liquid biosolids to bituminous coal strip mine spoil bank.

Under the 1977 Surface Mining Control and Reclamation Act (e.g., PL 95-87), the postmining land use is required to be at a level equal to or greater than the premining land use. Typical postmining land uses include the following:

- Wilderness or unimproved use
- Limited agricultural or recreation (e.g., forests land, grazing, hunting, and fishing)
- Developed agriculture or recreation (e.g., cropland, vacation sports, etc.)
- Suburban dwellings or light commercial industry
- Urban dwellings or heavy commercial industry

The 1982 and 1988 amendments to the 1977 Surface Mining Control and Reclamation Act (54 CFR Part 23) set forth the following additional requirements for reclaiming mining sites:

1. The permanent vegetative cover must be at least equal in extent to the natural vegetation of the area and must achieve productivity levels comparable with those of unmined lands for the approved postmining land use.
2. The period of responsibility for reclaiming the site begins after the last year of augmented seeding, fertilization, irrigation, or other work that ensures relative success.

3. In areas that receive more than 26 in of average annual precipitation, the period of legal responsibility will continue for not less than 5 years. In areas with 26 in of precipitation or less, the period of legal responsibility will continue for no less than 10 years.
4. Normal husbandry practices essential for plant establishment are permitted during the period of responsibility as long as they can reasonably be expected to continue after bond release.
5. In areas receiving more than 26 in of precipitation, the vegetative cover and production of pasture, grazing land, and cropland must be equal to or exceed the success standard during any 2 years of the responsibility period except the first year. Areas approved for other uses shall equal or exceed success standards during the growing season of the last year of the responsibility period. In areas receiving less than 26 in of precipitation, the vegetative cover must be equal to the success standard for at least the last 2 years of the responsibility period.
6. The ground cover, productivity, or tree stocking of the revegetation area shall be considered equal to the success standards approved by the regulatory authority when they are not less than 90 percent of the success standard with 90 percent statistical confidence.

7.6.8 Biosolids application rates

The basic approach for estimating the biosolids application rate for reclamation sites depends on whether a maximum one-time biosolids application is desired or if biosolids are to be applied routinely to the site. If a maximum one-time biosolids application is desired, the goal of the biosolids land-application approach is to create a large pool of nutrients for vegetation so that additional biosolids applications are unnecessary. To conserve nutrients under this biosolids land-application approach, vegetation on these sites is not harvested. Nutrients are recycled through the decay of leaf litter and through the planting of nitrogen-fixing vegetation (e.g., legumes). It should be noted that because of the large amount of biosolids applied, the one-time biosolids application approach for land reclamation typically results in applying nitrogen in excess of vegetative requirements (i.e., agronomic rate), which potentially can lead to nitrate leaching into groundwater.

If biosolids are to be applied on a regularly scheduled basis, management of the site normally includes periodic harvesting of vegetation. The biosolids application rate for reclamation sites on which the vegetation is harvested is based on either the agricultural or forested agronomic rate design. Since the approach for estimating the agronomic rate for agriculture and forested land was described previously, the following section will focus on the technical approach for estimating the maximum one-time biosolids application rate.

The approach for determining the maximum acceptable one-time application of biosolids to a reclamation site is based on both the environmental impacts of the applied nitrogen and the resulting heavy metal loadings. The following four steps summarize the methodology for estimating the maximum one-time biosolid application rate:

1. Determine the maximum allowable biosolids application rate $S_{\max}$ based on the 40 CFR Part 503 rule cumulative pollutant loading rate (CPLR) limit for heavy metals (Table 1.4).
2. Perform a nitrogen mass balance around the reclamation site to determine the available nitrogen in excess of plant needs.
3. Estimate the groundwater nitrogen concentration that may result from applying biosolids at the rate of $S_{\max}$.
4. If the groundwater nitrate concentration resulting from the land application of $S_{\max}$ is not acceptable to the regulatory authority, a lower application rate should be established that will not exceed a defined acceptable level.

If the results of the soil characterization sampling program indicate that the soil conditions (pH, moisture content, etc.) are adequate to minimize the leaching of metals, Eq. (7.15) may be used to estimate the maximum biosolids application rate $S_{\max}$ for each of the nine regulated heavy metals identified in the 40 CFR Part 503 rule [93,99]. It should be noted that at the time that this text was prepared, the USEPA was in the process of reevaluating the biosolids concentration limits for molybdenum [100]. After evaluating $S_{\max}$ for each of the nine regulated metals, the lowest biosolids application rate determined through the use of Eq. (7.15) would represent the maximum one-time (or cumulative) biosolids application.

Maximum biosolids application rate $S_{\max}$ (metric tons/ha)

$$= \frac{\text{cumulative pollutant loading rate (kg/ha)}}{\text{toxic metal concentration in biosolids (mg/kg)} \cdot 0.001} \tag{7.15}$$

where $S_{\max}$ = maximum biosolids application rate (metric tons/ha)
CPLR = cumulative pollutant loading rate (kg/ha); Table 1.4
0.001 = $(\text{kg}/10^6\ \text{mg}) \cdot (10^3\ \text{kg/metric ton})$

To determine if this one-time biosolids application will result in nitrogen levels in excess of vegetative requirements, the PAN available during the first year as well as for future years should be estimated. It should be noted that while the PAN for the first year may be estimated using Eq. (7.2), the PAN for future years is simply equal to the quantity of organic nitrogen mineralized during the year under consideration (note that it is assumed that all the nitrate and/or ammonia applied in the first application is consumed during that year). The PAN available after the first year of biosolids application may be estimated using Eq. (7.16):

$$\mathrm{N_m} = SK_{\min}\mathrm{N_o} \tag{7.16}$$

where $\mathrm{N_m}$ = quantity of organic nitrogen mineralized in year under consideration (plant-available nitrogen)

S = biosolids application rate, metric tons per hectare

$K_{\min}$ = mineralization factor for year under consideration; see Table 7.11 or site-specific measurement

$\mathrm{N_o}$ = fraction of organic nitrogen originally in biosolids

To quantify the excess nitrogen $\mathrm{N_{excess}}$ being applied to the reclamation site, the nitrogen requirement of the vegetation is subtracted from the PAN. If it is found that excess nitrogen is being applied, it is possible to conservatively estimate the resulting groundwater nitrate concentration by assuming that all the excess nitrogen is converted to nitrate that percolates to the groundwater undiluted [100]. Using this assumption, the maximum groundwater nitrate concentration may be estimated using Eq. (7.17).

Nitrate concentration in groundwater (mg/liter)

$$= \frac{(\mathrm{N_{excess}})\,(10^6\ \mathrm{mg/kg})\,(10^3\ \mathrm{cm^3/liter})}{(10^8\ \mathrm{cm/ha})\,P_1\,(1 - ET)} \tag{7.17}$$

where $\mathrm{N_{excess}}$ = nitrogen applied in excess of agronomic rate (kg/ha)

P_1 = precipitation rate that infiltrates ground (centimeters)

ET = evapotranspiration rate (fraction of infiltration that is lost through evapotranspiration)

If the estimated groundwater nitrate concentration is unacceptable to the regulatory agency, the biosolids application rate must be lowered. Alternatively, if the permissible nitrate concentration is established to be the maximum contaminant level (MCL) of 10 mg/liter, Eq. (7.17) can be used in reverse to estimate the maximum permissible excess nitrate loading ($\mathrm{N_{excess}}$). Example 7.9 illustrates the approach to determining the maximum one-time biosolids application rate for land reclamation.

Example 7.9 The Ross County Water Reclamation Facility desires to land apply its anaerobically digested biosolids to reclaim 200 ha of deposited mine tailings. On average, the biosolids contain approximately 7 percent total nitrogen, of which 5 percent is ammonia and 0.1 percent is nitrate. The annual net infiltration P_1 has been estimated to be 64 cm, while the average fraction of evapotranspiration (ET) loss has been found to be approximately 40 percent. If it is desired to initially grow rye grass on the site (nitrogen uptake rate 290 kg/ha·yr) to minimize soil erosion and leaching of nitrogen, estimate the quantity of biosolids that can be applied to the mining land if a one-time application is desired. Assume that the biosolids are surface applied with no incorporation (i.e., $K_v = 0.5$) and that the metal content of the liquid biosolids is given by the data in the following table.

Metal	Concentration (dry basis)* (mg/kg)	CPLR† (kg/ha)
Arsenic	54	41
Cadmium	36	39
Copper	3950	1500
Lead	522	300
Mercury	34	17
Nickel	247	420
Selenium	26	100
Zinc	2456	2800

NOTE: lb/acre·yr·1.1209 = kg/ha·yr.

*Molybdenum is the ninth toxic metal to be regulated. At present, the USEPA is reevaluating the CPLR value for molybdenum.

†Cumulative pollutant loading rate, adapted from the 40 CFR Part 503 rule.

solution

Step 1. Estimate the maximum biosolids application rate (metric tons/ha) for each metal. The following example for the metal arsenic illustrates the approach.

Application rate (metric tons/ha)

$$= \frac{\text{cumulative pollutant loading rate (kg/ha)}}{\text{toxic metal concentration in biosolids (mg/kg)} \cdot 0.001}$$

$$= \frac{41\ \text{kg/ha}}{54\ \text{mg/kg} \cdot 0.001} = 759\ \text{metric tons/ha}$$

Step 2. Perform similar calculations for the other toxic metals, as illustrated in the following table.

Metal	Biosolids application rate $S_{\max}$ (metric tons/ha)
Arsenic	759
Cadmium	1083
Copper	380
Lead	575
Mercury	500
Nickel	1700
Selenium	3850
Zinc	1140

Step 3. From the preceding calculations, identify the limiting pollutant as the metal that gives the lowest biosolids application rate. From the analysis, copper will limit the biosolids application rate. The maximum lifetime application of these biosolids to the mining area will be 380 metric tons/ha.

Step 4. Estimate the maximum biosolids application rate for the 200-ha mine tailing area:

Total amount of biosolids to be applied (metric tons)

$$= \text{area (ha)} \cdot \text{application rate (metric tons/ha)}$$

$$= 200 \text{ ha} \cdot 380 \text{ metric tons/ha}$$

$$= 76{,}000 \text{ metric tons (83,600 U.S. tons)}$$

Step 5. Estimate the percentage of organic nitrogen mineralized during the first 3 years by finding the appropriate mineralization rates for anaerobically digested biosolids. From Table 7.11, $K_{\min}$ has the following values for the first 3 years of biosolids application:

Year	$K_{\min}$
0–1	30
1–2	15
2–3	8

Step 6. Estimate the fraction of organic nitrogen N_o in the biosolids:

$$\mathrm{N_o} = \text{total nitrogen} - (\text{nitrate} + \text{ammonia})$$

$$= 7 \text{ percent} - (0.1 \text{ percent} + 5 \text{ percent})$$

$$= 1.9 \text{ percent (or 0.019)}$$

Step 7. Estimate the PAN per metric ton of biosolids for the first year if a one-time biosolids application of 380 metric tons/ha is applied to the site using Table 7.11 and Eq. (7.2). Note that K_v equals 0.5 because the biosolids are not immediately incorporated into soil.

$$\text{PAN (kg/ha)} = S\,[\,(\mathrm{NO_3}) + K_v\,(\mathrm{NH_4}) + K_{\min}\,(\mathrm{N_o})\,]\,10$$

$$= 380 \text{ metric tons/ha } [0.1 + 0.5\,(5.0) + 0.3\,(1.9)\,]\,10$$

$$= 12{,}046 \text{ kg N/ha}$$

Step 8. Estimate the PAN during years 2 and 3 using Eq. (7.16):

Year 2: $$\mathrm{N_m} = SK_{\min}\mathrm{N_o}$$

$$= 380 \text{ metric tons/ha} \cdot 0.15 \cdot 1.9$$

$$= 108.3 \text{ kg/ha}$$

Year 3: $N_m = SK_{\min}N_o$

$= 380$ metric tons/ha $\cdot\ 0.08 \cdot 1.9$

$= 57.8$ kg/ha

Step 9. Estimate the excess nitrogen applied by subtracting the crop nitrogen requirement (i.e., 290 kg/ha) from the PAN over the first 3 years:

Year 1 $N_{\text{excess}} = 12{,}046 \text{ kg/ha} - 290 \text{ kg/ha} = 11{,}756 \text{ kg/ha}$

Year 2 $N_{\text{excess}} = 108.3 \text{ kg/ha} - 290 \text{ kg/ha} = 0 \text{ kg/ha}$

Year 3 $N_{\text{excess}} = 57.8 \text{ kg/ha} - 290 \text{ kg/ha} = 0 \text{ kg/ha}$

It should be noted that in year 1 the available nitrogen is approximately 40 times the crop uptake, and in this case, significant amounts of nitrate leaching should be expected. For years 2 and 3, the nitrogen uptake exceeds the PAN, and no nitrate leaching should be anticipated.

Step 10. Estimate the maximum nitrate concentration in groundwater during the first year as a result of the one-time biosolids application to the land reclamation site using Eq. (7.17):

Nitrate concentration in groundwater (mg/liter)

$$= \frac{(N_{\text{excess}})\,(10^6 \text{ mg/kg})\,(10^3 \text{ cm}^3\text{/liter})}{(10^8 \text{ cm/ha})\,(P_1)\,(1 - ET)}$$

$$= \frac{(11{,}756 \text{ kg N/ha})\,(10^6 \text{ mg/kg})\,(10^3 \text{ cm}^3\text{/liter})}{(10^8 \text{ cm/ha})\,(64 \text{ cm})\,(1 - 0.4)}$$

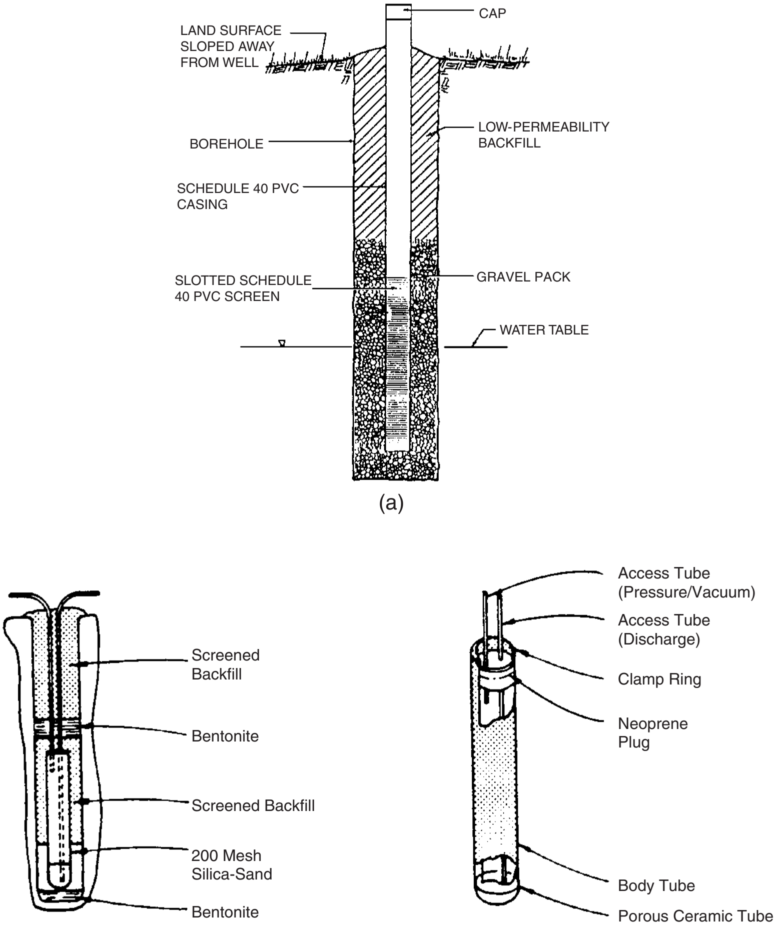
$$= 3061 \text{ mg } NO_3 - \text{N/liter}$$

Clearly, the resulting concentration of nitrate in groundwater due to the one-time biosolids application is unacceptable. The one-time biosolids application rate should be reduced significantly to a level that results in a more environmentally acceptable nitrate concentration.

7.6.9 Monitoring

To evaluate the environmental impact of applying biosolids to a reclamation site, a regulatory agency–approved monitoring system must be developed and implemented [70]. In most cases, the monitoring system design will consist of both up- and downgradient groundwater wells as well as onsite lysimeters (Fig. 7.13).

The groundwater wells are used to quantify the changes in groundwater quality as a result of biosolids application, whereas the



(a)

(b)

Figure 7.13 Schematic diagrams of (*a*) a monitoring well; (*b*) pressure/vacuum lysimeter.

lysimeters permit sampling of soil leachate. The locations of groundwater wells are selected after the groundwater flow pattern has been determined. Lysimeters are positioned in areas representative of general site conditions.

The complexity of the monitoring program can vary significantly depending on state and local regulations as well as site-specific conditions [70,100]. At a minimum, it is desirable to analyze the soil at the reclamation site 1 year after biosolids have been applied to document changes in soil pH and toxic metal concentrations. In addition to establishing a monitoring schedule to document system performance, state regulatory authorities generally will require an annual surface and groundwater analysis for nutrients and heavy metals. Adjustments to the biosolids land-application program (including corrective action) may be necessary if monitoring data suggest the undesirable migration of nutrients and/or heavy metals from the land reclamation site [48,100,108].

7.6.10 Surface runoff storage volume required

Land reclamation sites usually require that storage be provided for surface runoff resulting from precipitation and snowmelt. Various alternatives for disposing of surface runoff range from evaporation ponds to treatment and reapplication to the site. In all cases, an estimate of the runoff storage volume is required for sizing of containment structures. Approaches for designing surface water runoff collection facilities may be found in refs. [59,75,104].

7.6.11 Groundwater leachate collection and control

Subsurface drainage systems may be needed at land reclamation sites when natural drainage is restricted by relatively impermeable layers in the soil profile [98]. As a result of the restrictive layer, shallow groundwater tables can extend close to the soil surface. High groundwater tables are a serious concern because they may create problems such as ponding, anaerobic soil conditions, and muddy surfaces.

If a subsurface drainage collection system is installed beneath the reclamation site, the leachate collected within the drainage system must be treated, stored, and/or disposed. As in the case of surface runoff, an estimate of the leachate storage volume is required for the appropriate sizing of containment structures [98].

7.7 Land Application of Biosolids to Public-Access Sites

The 40 CFR Part 503 rule defines *public-access sites* as land with a high potential for public contact such as parks, ball fields, cemeteries, plant nurseries, turf farms, golf courses, etc. [89,100]. Biosolids normally are supplied to public access sites in bags or other containers that are sold or given away. Moreover, these biosolids typically are processed and marketed by municipalities or private firms as a brand-name fertilizer and/or soil-conditioning product (Fig. 7.14). The use of public-access sites for biosolids land application is particularly useful for municipalities with limited land available (e.g.,

Figure 7.14 Photograph of the biosolids fertilizer product sold by the City of Los Angeles, California.

highly populated areas with few agricultural, forest, or reclamation sites available for biosolids application).

Biosolids land application at public-access sites requires that the most stringent biosolids quality be met (e.g., Class A pathogen reduction, annual pollutant loading rates, etc.). These strict quality requirements are necessary because of the high potential for human contact and because it is infeasible to impose site restrictions for this method of biosolids land application (Chap. 1).

To be in compliance with the 40 CFR Part 503 rule, a label must be affixed to the biosolids container or an information sheet must be provided when biosolids meeting the annual pollutant limit rate (APLR) limits are sold or given away. In addition to ensuring that a label or information sheet is furnished, the label or information sheet must specify the biosolids application rate that can be used without causing the APLRs to be exceeded. This biosolids application rate is referred to as the *annual whole-sludge application rate* (AWSAR), which may be estimated using Eq. (7.18):

Annual whole-sludge application rate (dry metric tons/ha)

$$= \frac{\text{APLR (kg/ha} \cdot \text{yr)}}{\text{concentration of pollutant in biosolids (mg/kg)} \cdot 0.001} \qquad (7.18)$$

The AWSAR must be calculated for each of the nine regulated metals, from which the lowest AWSAR is established by the biosolid preparer as the allowable AWSAR. Example 7.10 illustrates the approach used for estimating the allowable AWSAR.

Example 7.10 The Langston County Water Reclamation Facility is considering applying dewatered biosolids as fertilizer for use in local city parks. If the metal content in the biosolids is described by the data in the following table, estimate the AWSAR in dry metric tons per hectare per year.

Metal*	Concentration in biosolids (mg/kg)
Arsenic	15
Cadmium	47
Copper	1600
Lead	250
Mercury	3
Nickel	220
Selenium	41
Zinc	1100

*The USEPA is currently reevaluating the limitations for molybdenum.

solution

Step 1. Calculate the AWSAR for each regulated pollutant using Eq. (7.18) and the APLRs from Table 1.4. For example, for arsenic, the annual pollutant-loading limit is 2.0 kg/ha·yr (Table 1.4). Given this APLR, the AWSAR can be estimated as follows:

Annual whole-sludge application rate (dry metric tons/ha · yr)

$$= \frac{\text{APLR (kg/ha} \cdot \text{yr)}}{\text{concentration of pollutant in biosolids (mg/kg)} \cdot 0.001}$$

$$= \frac{2.0\ \text{kg/ha} \cdot \text{yr}}{(15\ \text{mg/kg}) \cdot 0.001}$$

$$= 133.3\ \text{dry metric tons/ha} \cdot \text{yr}$$

Step 2. The AWSAR can be calculated for each heavy metal using the same procedure. The results are given in the following table:

Metal	Concentration in biosolids (mg/kg)	APLR (kg/ha·yr)	AWSAR (metric tons/ha·yr)
Arsenic	15	2.0	133.3
Cadmium	47	1.9	40.4
Copper	1600	75.0	46.9
Lead	250	15.0	60.0
Mercury	3	0.9	300.0
Nickel	220	21.0	95.5
Selenium	41	5.0	122.0
Zinc	1100	140.0	154.0

Step 3. The lowest AWSAR is 40.4 metric tons/ha·yr, which was estimated for cadmium. Therefore, the maximum annual biosolids application rate for these biosolids is 40.4 metric tons/ha·yr. This information must be included on the label or information sheet accompanying the APLR biosolids.

In addition to specifying the AWSAR that ensures that the APLR limits are not exceeded, additional information that must be included on the required label or information sheet accompanying the biosolids includes the following:

1. Name and address of the person who prepared the biosolids
2. A statement that prohibits biosolids application except in accordance with the instructions on the label or information sheet provided
3. Nitrogen content

POTWs that desire to minimize the regulatory burden of the 40 CFR Part 503 rule on their biosolids management program may choose to generate and market exceptional-quality (EQ) biosolids. To be classified as EQ biosolids, these biosolids must meet the following standards: (1) ceiling limits for metals, (2) pollutant concentration limits for metals, (3) Class A pathogen-reduction criteria, and (4) one of the first eight vector attraction reduction options (Table 1.13). In addition to the absence of a labeling requirement, biosolids that are classified as exceptional quality may be applied to land as freely as any commercial fertilizer (i.e., no legal requirement to estimate and report an AWSAR).

7.7.1 Marketing of biosolids

After processing biosolids to achieve the minimum quality requirements, biosolids to be applied to land at public-access sites, lawns, and home gardens are marketed to distributors or end users (e.g., landscapers, home gardeners) as bagged biosolids. Successful biosolids land-application programs for public-access sites have been characterized by proactive community marketing programs [100]. A successful marketing program should highlight the following biosolids program characteristics:

- High-quality product
- Ready availability of product
- Competitive pricing
- Maintaining good public relations
- Community acceptance of biosolids land-application operations

In addition to advertising specific biosolids quality, having a diversified range of products is a useful approach for marketing biosolids. For example, a Class A biosolids could be produced for use by landscapers, public works departments, the public, etc., and a Class B bulk biosolids could be generated for used by agricultural, forest, and reclamation projects. Another potential product is exceptional-quality biosolids. If the biosolids meet the EQ criteria (e.g., Class A pathogen reduction, pollution concentration metal limits, etc.), they can be applied as freely as any other fertilizer or soil amendment to any type of land. Exceptional-quality biosolids are particularly appealing to customers with a variety of fertilizer needs. To maintain an economically sustainable biosolids program, the costs of the various biosolids products should reflect the degree of processing required to achieve a given quality of material.

7.7.2 Marketing cost considerations

The costs of a biosolids marketing program may be high relative to the costs of actual land application. Major cost factors in marketing biosolids include the following:

- Biosolids processing
- Transportation
- Market development

Biosolids processing (e.g., enhanced stabilization, dewatering, etc.) can involve significant capital expenditures, and some generators/preparers of biosolids may choose to subcontract some of the processing work. The choice to subcontract biosolids processing will be an economic decision based on the biosolids selling price and the resulting payback period for capital equipment expenditures [77,100].

Transportation costs represent a significant economic expenditure in marketing biosolids. Transportation costs may include conveying biosolids from the wastewater treatment plant to the processing center, transport of bulking materials (e.g., composting), and distribution of the finished product to the user. Although extending the geographic biosolids marketing area would increase shipping costs, it may be cost-effective if customers in the extended area are willing to pay a higher price for the product. General price information for biosolids is summarized in Table 7.20.

In some cases, the municipality does not make a profit from selling biosolids, but the sales revenue can reduce the overall POTW operating costs. In other cases, the demand for biosolids has exceeded supply.

TABLE 7.20 General Price Information of Wastewater Treatment Plants Selling Biosolids*

Type of wastewater plant†	Percent that sell biosolids‡	Price per ton§	Price per cubic yard§
a	60.0	$63	$5
b	71.4	$34	$7
c	42.1	—	$4
d	37.5	—	$7

*Adapted from ref. [100].

†Flow-rate group: (a) influent wastewater flow rate greater than 100 MG/day, (b) influent wastewater flow rate between 10 and 100 MG/day, (c) influent wastewater flow rate between 1 and 10 MG/day, (d) influent wastewater flow rate less than 1 MG/day.

‡Percents based on a total of 46 wastewater treatment plants surveyed.

§Only 50 percent of the wastewater treatment plants that reported selling biosolids provided price data.

In all circumstances, prices for biosolids should be adjusted to reflect market demand.

7.7.3 Developing product demand

To create and maintain product demand, many municipalities or private firms use a trade name to enhance marketability of biosolids (e.g., Milorganite is produced by the city of Milwaukee, Wisconsin). Some municipalities also conduct market surveys to determine who would be interested in purchasing their product. The wastewater treatment plant or other preparer of biosolids may be able to increase marketability by offering the customer important “services” such as the following:

- Storing users’ purchased biosolids at the wastewater treatment plant
- Providing users with results of biosolids quality and site characteristic tests
- Offering transport of biosolids to the land-application site
- Assisting in obtaining regulatory permits

In some areas of the United States, biosolids land-application programs involving public-access sites have failed because of inconsistent product quality and/or unsatisfactory delivery schedules. These problems normally are the result of poor management, including the lack of strategic planning and quality control. It should be noted that it is difficult to reverse a negative public perception regarding a biosolids program once it has occurred. Therefore, before initiating a biosolids marketing program, the municipality should obtain a suitable level of funding to ensure professional management of its biosolids program.

7.8 Land Application of Domestic Septage

Domestic septage is defined in the 40 CFR Part 503 rule as the liquid or solid material removed from a septic tank, portable toilet, type III marine sanitation device, or a similar system that receives only domestic septage (i.e., household, noncommercial, nonindustrial sewage). Land application of domestic septage is an economical and environmentally sound practice used by many rural communities. The 40 CFR Part 503 rule specifies minimum requirements for the application of domestic septage to agricultural fields, forest land, and reclamation sites (e.g., non-public-contact sites). For land application of domestic septage to public-contact sites, the 40 CFR Part 503 rule specifies that domestic septage land application must meet the same quality standards, management practices, pathogen and vector attraction reduction criteria, etc. as the land application of bulk biosolids [87,93].

Domestic septage applied to non-public-contact sites is not required to meet the specific ceiling, cumulative, or pollution concentration limits that are applicable to land-applied biosolids. When these limits are not met, the maximum annual domestic septage application rate must be estimated using Eq. (7.19). The factor 0.0026 in Eq. (7.19) was obtained by assuming that (1) the nitrogen content in septage is completely mineralized over a 3-year period, (2) domestic septage is approximately 2.5 percent solids, and (3) the nitrogen content in septage is approximately 350 mg/kg (dry-weight basis) [95].

$$\text{Annual application rate (gal/acre}\cdot\text{yr)} = \frac{\text{lb nitrogen required by the crop (lb N/acre}\cdot\text{yr)}}{0.0026} \tag{7.19}$$

As illustrated by Eq. (7.19), the annual domestic septage application rate is a function of the nitrogen requirement of the crop being grown. Nitrogen requirements of a crop depend on expected yield, soil conditions, and other factors such as temperature, rainfall, and length of the growing season (Table 7.21). Example 7.11 illustrates use of Eq. (7.19) in estimating the annual application rate of domestic septage.

The primary reason for requiring the use of Eq. (7.19) is to minimize the application of nitrogen in excess of vegetative needs and to reduce the potential of groundwater contamination. It should be noted that although use of Eq. (7.19) facilitates the beneficial use of domestic septage by minimizing the technical and economic burden

TABLE 7.21 Typical Domestic Septage Application Rates*

Crop	Yield (bushels/acre·year)	Nitrogen requirement (lb N/acre·year)	Application rate† (gal/acre·year)
Corn	100	100	38,500
Oats	90	60	23,000
Barley	70	60	23,000
Grass/hay	4 tons/acre	200	77,000
Sorghum	60	60	23,000
Peanuts	40	30	11,500
Wheat	150	250	96,100
Wheat	70	105	40,400
Soybeans	40	30	11,500
Cotton	1.0 bale/acre	50	19,200
Cotton	1.5 bales/acre	90	35,000

*Adapted from ref. [95].
†Calculated using Eq. (7.19).

to the septage land appliers, in some cases use of Eq. (7.19) may result in the significant overapplication of nitrogen. For example, in cases where the domestic septage has been dewatered prior to land application (e.g., which typically occurs during storage of domestic septage), the solids content will be significantly greater than the 2.5 percent used in the development of Eq. (7.19). Under these circumstances, it is recommended that the actual nitrogen content of the septage be measured prior to land application and the septage land-application rate adjusted, if necessary. In other cases, the domestic septage may have a nitrogen content significantly greater than the 350 mg/kg assumed in Eq. (7.19). In these circumstances, good practice would dictate that the volume of domestic septage that is applied to land be reduced from the value calculated using Eq. (7.19).

Example 7.11 The Bethune County Sewer District has zoned 450 acres of agricultural land for the land application of domestic septage. If Kentucky bluegrass is being grown at the site, estimate the septage application rate if the crop nitrogen requirement is 225 lb/acre·yr.

solution Estimate the maximum annual septage application rate using Eq. (7.19):

Maximum annual application rate (gal/acre · yr)

$$= \frac{\text{lb nitrogen required by the crop (lb N/acre} \cdot \text{yr)}}{0.0026}$$

$$= \frac{(225 \text{ lb/ N/acre} \cdot \text{yr})}{0.0026}$$

$$= 86{,}538 \text{ gal/acre} \cdot \text{yr}$$

7.8.1 Domestic septage pathogen-reduction requirements

Domestic septage that is to be applied to land on a non-public-contact site must be managed so that pathogens are reduced. The 40 CFR Part 503 rule offers domestic septage land appliers two alternatives to meet this requirement. The first alternative uses crop, grazing, and site restrictions exclusively with no chemical treatment, whereas the second alternative requires raising the pH of the septage to 12 for 30 minutes through the addition of alkaline material [100].

For septage land appliers who choose the second alternative to meet pathogen-reduction criteria, the alkaline materials most commonly used to raise the pH of domestic septage are hydrated lime [$Ca(OH)_2$] and quicklime (CaO). Regardless of the chemical chosen, at a minimum, two separate representative samples of the chemically treated domestic septage must be taken at no less than a 30-minute interval to verify that the septage meets the pH requirement [95].

Although there is the added expense of chemical purchase and handling, neither grazing nor site restrictions are placed on the land application of alkali-stabilized septage (Alternative I—Pathogen Reduction Option). For septage land appliers who choose not to chemically treat septage, crop, grazing, and site restrictions are specified under the 40 CFR Part 503 rule (Alternative II—Pathogen Reduction Option). Table 7.22 provides a summary of the restrictions for land-applied domestic septage.

7.8.2 Domestic septage vector attraction reduction

For application of domestic septage to agricultural land, forest, or reclamation sites, the 40 CFR Part 503 rule requires that one of the following three options be implemented to reduce vector attraction: (1) subsurface injection, (2) incorporation (surface application followed by plowing within 6 hours), or (3) alkali stabilization. If the septage pathogen-reduction requirements are met using Alternative I, vector attraction reduction requirements can only be met by either subsurface injection of the septage or soil incorporation within 6 hours after land application. If the septage pathogen-reduction requirements are met using Alternative II (i.e., alkaline treatment of septage), vector attraction reduction requirements are assumed to be met, and no additional measures are necessary.

Subsurface injection of domestic septage places a barrier of earth between the septage and vectors such as flies or rodents that could transmit disease. In addition to limiting vector interaction, when septage is injected, the soil removes water from the septage, which reduces its mobility and odor. A vehicle typically used to inject septage is shown in Fig. 7.15.

TABLE 7.22 Summary of Restrictions for the Land Application of Domestic Septage*

Alternative I—Septage That Is Land Applied without Chemical Treatment

Crop restrictions

- Food crops with harvested parts that touch the septage-soil mixture and are totally above ground shall not be harvested for 14 months after application of domestic septage.
- Food crops with harvested parts below the soil surface of the land shall not be harvested for 38 months after application of domestic septage.
- Animal feed, fiber, and those food crops which do not touch the soil surface shall not be harvested for 30 days after application of the domestic septage.
- Turf grown on land where domestic septage is applied shall not be harvested for 1 year after application of the domestic septage when the harvested turf is placed on either a lawn or land with a high potential for public exposure, unless otherwise specified by the permitting authority.

Grazing restrictions

- Animals shall not be allowed to graze on land for 30 days after application of domestic septage.

Site restrictions

- Public access to land with a low potential for public exposure shall be restricted for 30 days after application of domestic septage. Examples of restricted access include remoteness of site, posting with no trespassing signs, and/or fencing.

Alternative II—Chemically Treated Septage That Is Land Applied†

Crop restrictions

- Food crops with harvested parts that touch the septage-soil mixture and are totally above ground shall not be harvested for 14 months after application of domestic septage.
- Food crops with harvested parts below the soil surface of the land shall not be harvested for 20 months after application of domestic septage when the domestic septage remains on the land surface for 4 months or longer prior to incorporation into the soil.
- Food crops with harvested parts below the surface of the land shall not be harvested for 38 months after application of domestic septage when the domestic septage remains on the land surface for less than 4 months prior to incorporation into soil.
- Animal feed, fiber, and those food crops which do not touch the soil surface shall not be harvested for 30 days after application of the domestic septage.
- Turf grown on land where domestic septage is applied shall not be harvested for 1 year after application of the domestic septage when the harvested turf is placed on either a lawn or land with a high potential for public exposure, unless otherwise specified by the permitting authority.

Grazing restrictions

- None

Site restrictions

- None

*Adapted from ref. [95].

†Domestic septage has had its pH raised to 12 or higher by addition of alkaline material, and without adding more alkaline material, the domestic septage remains at a pH of 12 or higher for at least 30 minutes prior to land application.

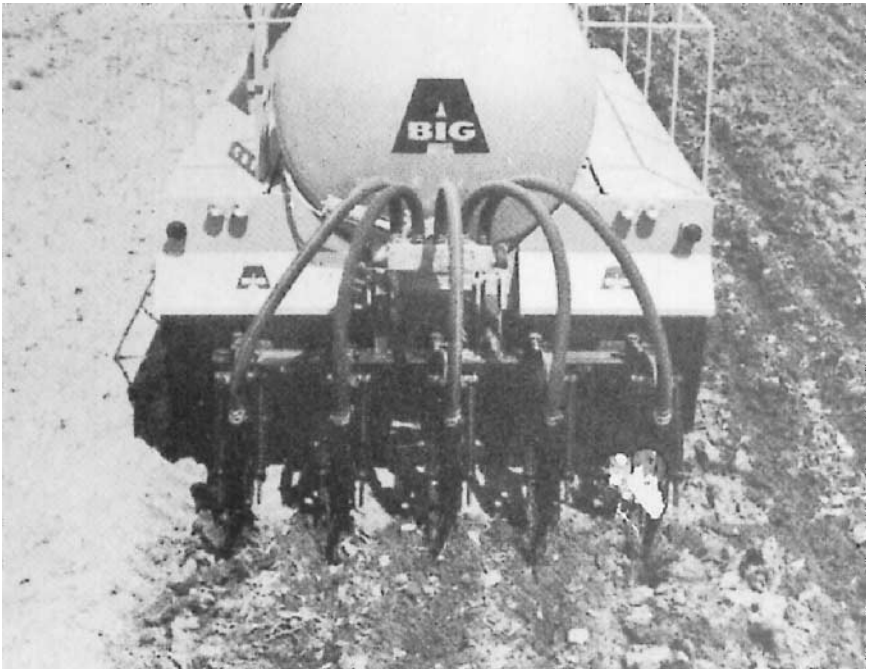


Figure 7.15 Pumper truck equipped for subsurface injection of domestic septage.

In addition to subsurface injection, incorporating surface-applied septage within 6 hours meets the vector attraction reduction requirements of the 40 CFR Part 503 rule. Standard agricultural disks or other tillage equipment pulled by a tractor or bulldozer can incorporate septage into soil (Fig. 7.16).

Alkali stabilization of domestic septage is achieved when its pH is raised to 12 through alkali addition and maintained at a pH of 12 or higher for 30 minutes without adding more alkali. When this option is used, every container of domestic septage must be monitored for pH to demonstrate that it meets the requirement. Once domestic septage is lime stabilized, it can be surface-applied, injected below the surface, or incorporated (Fig. 7.17). A summary of the vector attraction reduction alternatives for land application of domestic septage is provided in Table 7.23.

Finally, the 40 CFR Part 503 rule requires that the land applier of domestic septage must sign a certification form that states that the pathogen and vector attraction reduction requirements of the 40 CFR Part 503 rule have been met. The septage land applier must retain this certification for at least 5 years. A typical pathogen and vector attraction reduction certification form for land-applied domestic septage is provided in Fig. 7.18.

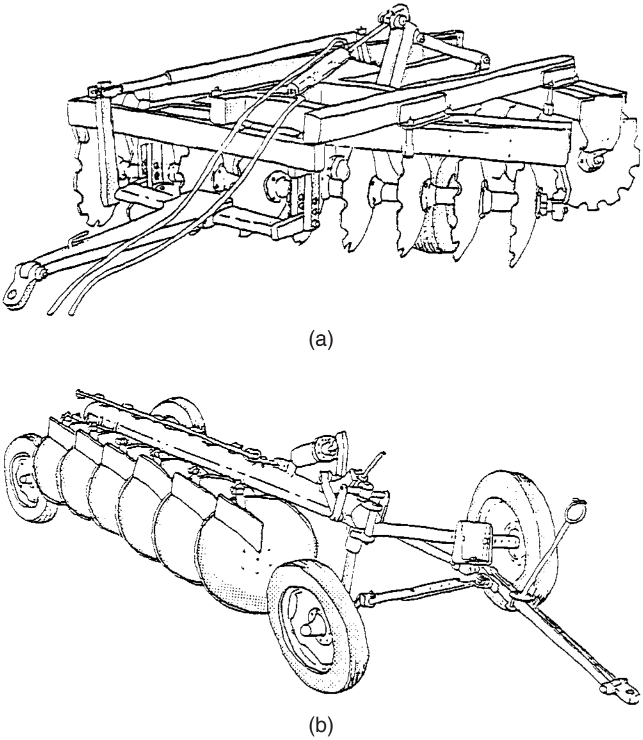


Figure 7.16 Schematic diagrams of (*a*) disk tiller; (*b*) a disk plow.

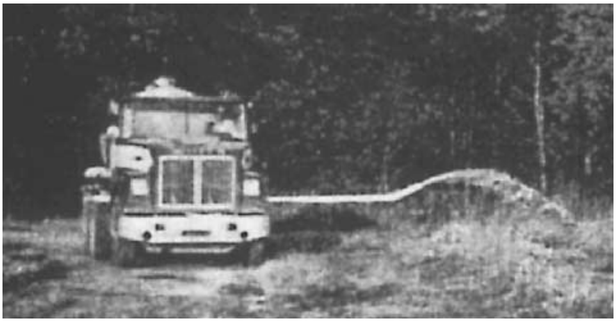


Figure 7.17 Surface application of domestic septage.

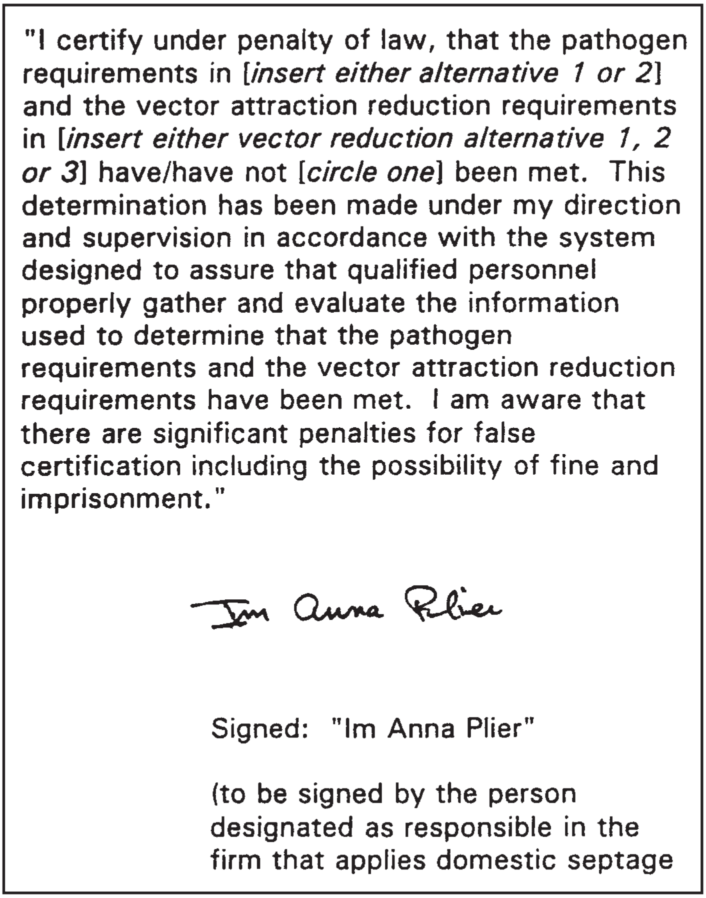
7.8.3 Domestic septage management practices

There are no management practice requirements for land appliers of domestic septage to non-public-contact sites specified in the 40 CFR Part 503 rule. However, many states have specific management practice requirements that must be followed. State or local requirements

TABLE 7.23 Domestic Septage Vector Attraction Reduction Alternatives*

Alternative I	Domestic septage shall be injected below the surface of the land, *and* no significant amount of the domestic septage shall be present on the land surface within 1 hour after the domestic septage has been injected.
Alternative II	Domestic septage applied to the land surface shall be incorporated into the soil surface plow layer within 6 hours after application.
Alternative III	The pH of domestic septage shall be raised to 12 or higher by addition of alkaline material and, without the addition of more alkaline material, shall remain at 12 or higher for 30 minutes.

*Adapted from ref. [95].



"I certify under penalty of law, that the pathogen requirements in [*insert either alternative 1 or 2*] and the vector attraction reduction requirements in [*insert either vector reduction alternative 1, 2 or 3*] have/have not [*circle one*] been met. This determination has been made under my direction and supervision in accordance with the system designed to assure that qualified personnel properly gather and evaluate the information used to determine that the pathogen requirements and the vector attraction reduction requirements have been met. I am aware that there are significant penalties for false certification including the possibility of fine and imprisonment."

Im Anna Plier

Signed: "Im Anna Plier"

(to be signed by the person designated as responsible in the firm that applies domestic septage

Figure 7.18 Typical certification for land appliers of domestic septage.

TABLE 7.24 Records That Must Be Kept by Septage Haulers/Land Appliers*

1. The location of the site on which domestic septage is applied.
2. The number of acres in the application site.
3. The date and time domestic septage is applied.
4. The nitrogen requirement of the crop or vegetation grown on the site in a 365-day period.
5. The rate in gallons per 365-day period at which domestic septage is applied.
6. Certification that pathogen and vector attraction reduction requirements have been met.
7. Descriptions of how pathogen and vector attraction reduction requirements have been met.

*Adapted from ref. [95].

may include the specification of minimum distances between domestic septage land-application sites and drinking-water wells and/or surface water. Good practice also would suggest a caution against applying domestic septage to flooded, frozen, or snow-covered land that could result in contaminated runoff.

7.8.4 Domestic septage recordkeeping

For land application of domestic septage to non-public-contact sites, the records that must be kept for at least 5 years are summarized in Table 7.24.

7.9 Supporting Facilities at Biosolids Land-Application Sites

The cost of supporting facilities, such as permanent all-weather access roads, fences, personnel buildings, etc., normally can be justified for high-rate biosolids application sites that will be used over a long project life. Each of the major supporting facilities typically found at biosolids land-application sites are described in the following sections.

7.9.1 Access roads

A permanent road should be constructed and maintained to allow biosolids transport and other vehicles to travel from the public road system to the land-application site. For large biosolids application sites, the roadway should be at least 20 to 24 ft wide for two-way traffic. However, for smaller sites, a 15-ft-wide road should suffice. To provide all-weather access, at a minimum, the roadway should be gravel surfaced. Road grades should not exceed equipment limitations, with maximum grades of 7 percent being typical.

7.9.2 Site fencing and security

Access to biosolids land-application sites should be limited to one or two entrances that have gates that can be locked when the site is unattended. Fencing requirements will vary and are generally influenced by the relative isolation of the site. Facilities that are located in isolated rural areas may require a less sophisticated type of fence or only fencing at the entrances to keep unauthorized vehicles out. Depending on the topography and vegetation on the site and adjoining areas, entrance gates may suffice to prevent unauthorized vehicular access, whereas at other sites it may be necessary to construct peripheral fences to restrict trespassers and animals.

7.9.3 Equipment and personnel buildings

At larger biosolids land-application sites or where climates are extreme, buildings are necessary for office space, equipment, and employee facilities. Restroom facilities should be provided for both site and transport personnel. At smaller facilities where buildings cannot be justified, trailers are normally employed.

7.9.4 Lighting and other utilities

If biosolids land-application operations occur at night, portable lighting should be provided at the operating area. Lights may be affixed to transport vehicles and onsite equipment. Lighting should be positioned to provide illumination to areas not covered by the regular headlights of the vehicle. If the facility has structures (employee facilities, office buildings, equipment repair or storage sheds), or if the access road is in continuous use, permanent security lighting may be needed. Larger biosolids land-application sites may need electrical, water, communications, and restroom services. Remote sites may have to extend existing services or use acceptable substitutes.

Water should be available for drinking, dust control, vehicle washing, and employee sanitary facilities. Telephone or radio communications systems are essential because accidents or spills can occur that require the on-site capability to call for assistance.

7.10 Problems

7.1 The Atieno County Sewer Improvement District is currently land applying 10 metric tons/ha (dry weight) of aerobically digested and thickened biosolids annually on pastureland. Chemical analysis of the biosolids reveals 2.6 percent nitrate, 0.1 percent ammonia, and 2.7 percent organic nitrogen (all on a dry-weight basis). Assuming that the liquid biosolids are surface applied, estimate the plant-available nitrogen (kg/ha) for the first 3 years from a one-time biosolids application.

7.2 The Odongo County Wastewater Treatment Plant is subsurface injecting 7 U.S. tons/acre (dry weight) of anaerobically digested and thickened biosolids annually to grow coastal Bermuda grass. If chemical analysis of the biosolids reveals that they contain 0.1 percent nitrate, 2.9 percent ammonia, and 2.5 percent organic nitrogen (all on a dry-weight basis), estimate the plant-available nitrogen (lb/acre) for the first 3 years from a one-time application.

7.3 The Akinyi City Wastewater Treatment Plant is considering land application of aerobically digested and thickened biosolids by subsurface injection into agricultural cropland. If the adjusted nitrogen fertilizer requirement for the crop (e.g., Kentucky bluegrass) is 224 kg/ha, estimate the biosolids agronomic rate S in (A) dry metric tons/ha, (B) wet metric tons/ha, and (C) m^3/ha if the biosolids chemical analyses generated the following qualitative data. Assume that the density of the liquid biosolids is 1040 kg/m^3.

NO_3-N: 2.6 percent (dry basis)
NH_4-N: 0.2 percent (dry basis)
Total nitrogen: 2.9 percent (dry basis)
Percent dry solids: 5.4 percent

7.4 The Benin County Water Reclamation Plant is surface applying its anaerobically digested and thickened biosolids onto nearby hayfields. If the adjusted nitrogen fertilizer requirement for the crop (e.g., alfalfa) is 282 kg/ha, estimate the biosolids agronomic rate S in (A) dry metric tons/ha, (B) wet metric tons/ha, and (C) m^3/ha if the chemical analyses of the biosolids generated the following qualitative data. Assume that the density of the liquid biosolids is 1020 kg/m^3.

NO_3-N: 0.05 percent (dry basis)
NH_4-N: 2.6 percent (dry basis)
Total nitrogen: 2.7 percent (dry basis)
Percent dry solids: 4.9 percent

7.5 The Bioko County Wastewater Treatment Plant is currently surface applying its aerobically digested and thickened biosolids to agricultural cropland. If the adjusted nitrogen fertilizer requirement for the crop (e.g., cotton) is estimated to be 80 lb nitrogen per acre, estimate the biosolids agronomic rate in (A) dry tons/acre, (B) wet tons/acre, and (C) gallons/acre. Chemical analyses of the liquid biosolids generated the following qualitative data. Assume that the density of the liquid biosolids is 8.50 lb/gal.

NO_3-N: 3.0 percent (dry basis)
NH_4-N: 0.05 percent (dry basis)
Total nitrogen: 3.2 percent (dry basis)
Total solids: 6.0 percent

7.6 The Libreville City Water Reclamation Plant is subsurface injecting its anaerobically digested and thickened biosolids to agricultural cropland. If the adjusted nitrogen fertilizer requirement for the crop (e.g., soybeans) is estimated to be 120 lb/acre, estimate the biosolids agronomic rate in (A) dry tons/acre, (B) wet tons/acre, and (C) gallons/acre. Chemical analyses of the liquid biosolids generated the following qualitative data. Assume that the density of the liquid biosolids is 8.50 lb/gal.

NO_3-N: 0.1 percent (dry basis)
NH_4-N: 2.8 percent (dry basis)
Total nitrogen: 3.1 percent (dry basis)
Total solids: 5.2 percent

7.7 The Dakoro County Sewer Improvement District is currently surface applying its aerobically digested biosolids on local farmland. If the biosolids contain 2.4 percent phosphorus, 2.6 percent nitrate, 0.05 percent ammonia, and 3.2 percent total nitrogen (all on a dry-weight basis), estimate the biosolids agronomic rate in tons (dry) per acre based on both phosphorus and nitrogen requirements if the crop to be grown is barley having a phosphorus and nitrogen demand of 16 lb phosphorus per acre and 60 lb nitrogen per acre, respectively. Assume that the phosphorus in biosolids is all inorganic phosphorus, of which 50 percent is plant available.

7.8 Navakchott City Water Reclamation Plant is currently subsurface injecting liquid biosolids into local pastureland. If the biosolids contain 2.1 percent phosphorus, 0.3 percent nitrate, 3.2 percent ammonia, and 4.4 percent total nitrogen (all on a dry-weight basis), estimate the agronomic rate in metric tons per hectare based on both phosphorus and nitrogen requirements if the crop to be grown is Reed Canary grass having a phosphorus and nitrogen demand of 45 kg phosphorus per hectare and 400 kg nitrogen per hectare, respectively. Assume that the phosphorus in biosolids is all inorganic phosphorus, of which 50 percent is plant available.

7.9 The Diourbel County Sewer Improvement District is generating biosolids having the following heavy metal concentrations. If the biosolids quality will remain constant for the active life of the land-application site, estimate the maximum biosolids application rate in U.S. tons per acre.

Metal	Concentration in biosolids (dry basis) (mg/kg)	CPLR (lb/acre)
Arsenic	55.0	37.0
Cadmium	12.0	35.0
Copper	2790.0	1339.0
Lead	540.0	268.0
Mercury	9.3	15.0
Nickel	280.0	375.0
Selenium	8.0	89.0
Zinc	3720.0	2500.0

7.10 The Calatrava City Wastewater Treatment Plant is generating biosolids having the following heavy metal concentrations. If the biosolids quality will remain constant for the active life of the land-application site, estimate the maximum biosolids application rate in metric tons per hectare.

Metal	Concentration in biosolids (dry basis) (mg/kg)	CPLR (kg/ha)
Arsenic	18.0	41.0
Cadmium	67.0	39.0
Copper	1475.0	1500.0
Lead	238.0	300.0
Mercury	3.0	17.0
Nickel	127.0	420.0
Selenium	34.0	100.0
Zinc	2710.0	2800.0

7.11 The Niamey County Wastewater Treatment Plant is surface applying anaerobically digested and thickened biosolids to forestlands. If no previous biosolids have been applied to the site and the forest contains primarily southern pine with understory (N_{req} = 260 kg/ha), estimate the biosolids application rate in metric tons per hectare for the first 3 years of application. Assume that the biosolids will be applied each year and that the denitrification rate (i.e., K_{den}) was determined through laboratory tests to be 0.20. The biosolids chemical analyses (dry-mass basis) are given by the following data:

NO_3-N: 0.1 percent

NH_4-N: 2.9 percent

Total nitrogen: 4.5 percent

Percent dry solids: 4.9 percent

7.12 The Bitam City Water Reclamation Plant is subsurface injecting aerobically digested and thickened biosolids to forestlands that contain mainly mixed hardwood trees (N_{req} = 250 lb/acre). If no previous biosolids have been applied to the site, estimate the biosolids application rate in U.S. tons per acre for the first 3 years of application. Assume that the biosolids will be applied each year and that the denitrification rate (i.e., K_{den}) was determined through laboratory tests to be 0.10. The biosolids chemical analyses (dry-mass basis) are given by the following data:

NO_3-N: 2.8 percent

NH_4-N: 0.1 percent

Total nitrogen: 3.0 percent

Percent dry solids: 4.3 percent

7.13 The Geidam County Sewer Improvement District desires to land apply its thickened biosolids to reclaim arid rangeland for agricultural production. If the thickened biosolids will be stored initially in a surface impoundment constructed near the land-application site, estimate the minimum storage volume (cubic meters) required for the surface impoundment if the following operational and safety specifications have been developed. Assume that the density of the liquid biosolids is 1020 kg/m^3.

Average biosolids generation rate: 2010 kg/day (dry-mass basis)
Liquid biosolids: 3.6 percent solids
Liquid storage capacity required by regulatory agency: 90 days
Safety factor: 1.5

7.14 The Cacine County Water Reclamation Plant desires to land apply its thickened biosolids to reclaim 80 ha of deposited mine tailings. The biosolids contain 5 percent total nitrogen, of which 3.5 percent is ammonia and 0.1 percent is nitrate. If rye grass will be grown on the site (nitrogen uptake rate = 290 kg/ha·year), estimate the quantity of biosolids that can be applied to the mining land if a one-time biosolids application is planned. Assume that the biosolids are subsurface injected and that the biosolids metal content is given by the data contained in the following table. The annual net infiltration P_1 and fraction of evapotranspiration loss (ET) have been estimated at 58 cm and 26 percent (0.26), respectively.

Metal	Concentration (dry basis)(mg/kg)	CPLR (kg/ha)
Arsenic	35	41
Cadmium	61	39
Copper	2900	1500
Lead	479	300
Mercury	19	17
Nickel	180	420
Selenium	26	100
Zinc	2650	2800

7.15 The Makurdi Sand and Gravel Works, Inc., desires to land apply locally generated biosolids to revegetate 260 acres of mining property. If tall fescue is to be grown on the site (nitrogen uptake rate = 185 lb N/acre·year), estimate the quantity of biosolids that should be applied to the mining land if a one-time biosolids application were planned. Assume that the biosolids contain 4.4 percent total nitrogen, of which 0.1 percent is ammonia and 3.1 percent is nitrate (dry-weight basis). The annual net infiltration P_1 and fraction of evapotranspiration loss (ET) have been estimated at 84 cm and 39 percent (0.39), respectively. The biosolids are surface applied and contain a metal content that is described by the data contained in the following table:

Metal	Concentration in biosolids (dry basis) (mg/kg)	CPLR (lb/acre)
Arsenic	45.0	37.0
Cadmium	32.0	35.0
Copper	2190.0	1339.0
Lead	510.0	268.0
Mercury	14.3	15.0
Nickel	240.0	375.0
Selenium	28.0	89.0
Zinc	2720.0	2500.0

7.16 The Bitam County Public Works Department is considering using thickened biosolids as fertilizer and soil amendment for maintaining several public golf courses. If the average metal content in biosolids is described by the data contained in the following table, estimate the annual whole-sludge application rate in dry metric tons per hectare per year.

Metal	Concentration in biosolids (mg/kg)
Arsenic	22
Cadmium	39
Copper	1350
Lead	212
Mercury	17
Nickel	87
Selenium	29
Zinc	1840

7.17 The Geidam City Public Works Department desires to use thickened biosolids as an inexpensive fertilizer and soil conditioner in public parks. If the metal content in biosolids is described by the data contained in the following table, estimate the annual whole-sludge application rate in dry U.S. tons per acre per year.

Metal	Concentration in biosolids (mg/kg)
Arsenic	15
Cadmium	12
Copper	980
Lead	115
Mercury	27
Nickel	210
Selenium	19
Zinc	1200

7.18 The Bameda County Sewer Improvement District has zoned 135 ha of pastureland for the land application of domestic septage. If Kentucky bluegrass is being grown at the site, estimate the septage application rate if the crop nitrogen requirement is 190 kg nitrogen/ha·year.

7.19 The Catete City Wastewater Treatment Plant has allocated 200 acres of agricultural land for the land application of domestic septage. If grain sorghum is to be grown at the site, estimate the septage application rate if the crop nitrogen requirement is 120 lb nitrogen/acre·year.

7.20 The Windhoek County Water Reclamation Plant has leased 120 acres of farmland for the land application of domestic septage. If wheat is to be grown at the site, estimate the septage application rate if the crop nitrogen requirement is 65 lb nitrogen/acre·year.

7.11 References

1. Aguilar, R., J. Gosz, and T. Ward (1992), "Sewage Sludge Application in Semiarid Grasslands: Effects on Vegetation and Water Quality. Annual Report of Attainment" Project No.1423645, New Mexico Water Resources Research Institute (NMSU), Las Cruces, N.Mex.
2. American Public Health Association (1985), *Standard Methods for the Examination of Water and Wastewater,* APHA, Washington.
3. American Society of Agronomy (1974), *Drainage for Agriculture,* ASA, Madison, Wis.
4. Bastian, R. K. (1997), "Biosolids Management in the United States," *Water Environment and Technology,* pp. 45–50.
5. Bastian, R. K. (1997), "The Biosolids (Sludge) Treatment, Beneficial Use and Disposal Situation in the USA," *European Water Pollution Control,* pp. 62–79.
6. Bastian, R. K., P. Sobel, L. Setlow, D. Sauders, and D. Condra (1998), "Radiation in Biosolids and Ash: Guidance for POTWs and Initial Survey Results," Water Environment Federation, CP3805.
7. Beauchamp, E. G., G. E. Kidd, and G. Thurtell (1978), "Ammonia Volatilization from Sewage Sludge Applied in the Field," *Journal of Environmental Quality* 7:141–146.
8. Beauchamp, E. G., G. E. Kidd, and G. Thurtell (1982), "Ammonia Volatilization from Sewage Sludge Applied in the Field," *Canadian Journal of Soil Science* 62:11–19.
9. Bohn, H. L., B. L. McNeal, and G. A. O'Connor (1979), *Soil Chemistry,* Wiley, New York.
10. Brady, N. C. (1984), *The Nature and Properties of Soil,* 9th ed., Macmillan, New York.
11. Brobst, R. B. (1999), "EPA's Biosolids Data Management System and Plans for Evaluating Biosolids Quality," WEF/ASSA Joint Residuals and Biosolids Management Conference, Charlotte, N.C.
12. Brunke, R., P. Aivo, P. Schuepp, and R. Gordon (1988), "Effect of Meteorological Parameters on Ammonia Loss from Manure in the Field," *Journal of Environmental Quality* 17:431–436.
13. Burkhardt, J., W. Miller, and M. Azad (1993), "Biosolids Application to Rangelands," *Water Environment and Technology* 5(5):68–71.
14. Carter, M., ed. (1993), *Soil Sampling and Methods of Analysis,* Lewis Publishers, Boca Raton, Fla.
15. Chae, Y. M., and M. A. Tabatabai (1986), "Mineralization of Nitrogen in Soils Amended with Organic Wastes," *Journal of Environmental Quality* 15:193–198.
16. Chaney, R. L. (1983), "Plant Uptake of Inorganic Waste Constituents," in J. F. Parr, P. B. Marsh, and J. M. Kla (eds.), *Land Treatment of Hazardous Wastes,* pp. 50–76, Noyes Data Corporation, Park Ridge, N.J.
17. Chaney, R. L. (1984), "Potential Effects of Sludge-born Heavy Metals and Toxic Organics on Soils, Plants, and Animals, and Related Regulatory Guidelines," in Workshop on the International Transportation, and Utilization or Disposal of Sewage Sludge, Including Recommendations (December 12–15), Final Report PNSP/85-01, Pan American Health Organization, Washington.
18. Code of Practice for Agricultural Use of Sewage Sludge (1992), Dd2946107, United Kingdom.
19. Coker, E. G. (1983), "The Use of Sewage Sludge in Agriculture," *Water Science Technology* 15:195–208.
20. Colorado State University (1995), "Effects of Land Application of Biosolid in Arid and Semi-Arid Environments—Proceedings," Metro Wastewater Reclamation District, Denver, Colo.
21. Corey, R. B. (1992), "Phosphorus Regulations: Impact of Sludge Regulations," *Crops Soils Newsletter* 20:5–10.
22. Crohn, D. M. (1996), "Planning Biosolids Land Application Rates for Agricultural Systems," *Journal of Environmental Engineering,* pp. 1058–1066.
23. Cuenca, R. H. (1989), *Irrigation System Design: An Engineering Approach,* Prentice-Hall, Englewood Cliffs, N.J.

24. Dowdy, R. H., and W. E. Larson (1975), "Metal Uptake by Barley Seedlings Grown on Soils Amended with Sewage Sludge," *Journal of Environmental Quality* 4:229–233.
25. Dowdy, R. H., and W. E. Larson (1975), "The Availability of Sludge-Borne Metals to Various Vegetable Crops," *Journal of Environmental Quality* 4:278–282.
26. Duncomb, D. R., W. E. Larson, C. E. Clapp, R. H. Dowdy, D. R. Linden, and W. K. Johnson (1982), "Effect of Liquid Wastewater Sludge Application on Crop Yield and Water Quality," *Journal of Water Pollution Control Federation* 54:1185–1193.
27. Falahi-Ardakani, A., J. C. Bouwkamp, F. R. Gouin, and R. L. Chaney (1988), "Growth Response and Mineral Uptake of Lettuce and Tomato Transplants Grown in Media Amended with Composted Sewage Sludge," *Journal of Environmental Horticulture* 6:130–132.
28. Fresquez, P., R. Francis, and G. Dennis (1990), "Soil and Vegetation Responses to Sewage Sludge on a Degraded Semiarid Broom Snakeweed/Blue Gamma Plant Community," *Journal of Range Management* 43(4):325–331.
29. Fresquez, P. R., and W. C. Lindemann (1982), "Soil and Rhizosphere Microorganisms in Amended Mine Spoils," *Soil Science Society Annual Journal* 46:751–755.
30. Fresquez, P. R., R. Aguilar, R. E. Francis, and E. F. Aldon (1991), "Heavy Metal Uptake by Blue Gamma Growing in a Degraded Semiarid Soil Amended with Sewage Sludge," *Journal of Water Air and Soil Pollution* 57–58:903–912.
31. Fresquez, P. R., R. E. Francis, and G. L. Dennis (1990), "Soil and Vegetation Responses to Sewage Sludge on a Degraded Semiarid Broom Snakeweed/Blue Gamma Plant Community," *Journal of Range Management* 43:325–331.
32. Fresquez, P. R., R. E. Francis, and G. L. Dennis (1990), "Effects of Sewage Sludge on Soil and Plant Quality in a Degraded Semiarid Grassland," *Journal of Environmental Quality* 19:324–329.
33. Garvey, D., C. Guarino, and R. Davis (1993), "Sludge Disposal Trends Around the Globe," *Water/Engineering & Management,* pp. 17–20.
34. Gilmor, J. T., F. Roman, and M. D. Clark (1996), "Decomposition of Biosolids in a Disposal Site Soil," *Journal of Environmental Quality* 25:1083–1086.
35. Gilmour, J., and M. Clark (1988), "Nitrogen Release from Wastewater Sludge: A Site-Specific Approach," *Journal of Water Pollution Control Federation* 60:494–341.
36. Gouin, F. R., and J. M. Walker (1977), "Deciduous Tree Seedling Response to Nursery Soil Amended with Composted Sewage Sludge," *Horticulture Science* 12:45–47.
37. Haug, R. T. (1986), "Composting Process Design Criteria," *Biocycle September* 1986:36–39.
38. Hegstrom, L. J., and S. D. West (1989), "Heavy Metal Accumulation in Small Mammals Following Sewage Sludge Application to Forests," *Journal of Environmental Quality* 18:345–349.
39. Henry, C. (1991), "Nitrogen Dynamics of Pulp and Paper Sludge to Forest Soils," *Water Science and Technology,* 24:417–425.
40. Henry, C., D. Cole, T. Hinckley, and R. Harrison (1993), "The Use of Municipal and Pulp and Paper Sludges to Increase Production in Forestry," *Journal of Sustainable Forestry* 1:41–55.
41. Huddleston, J. H., and M. P. Ronayne (1990), "Guide to Soil Suitability and Site Selection for Beneficial Use of Sewage Sludge," Oregon State University Extension Service.
42. Jewell, W. J. (1980), "Use and Treatment of Municipal Wastewater and Sludge in Land Reclamation and Biomass Production Projects: An Engineering Assessment," pp. 448–480 in *Utilization of Municipal Wastewater and Sludge: An Engineering Assessment for Land Reclamation and Biomass Production,* EPA 430/9-81-012, Washington.
43. Jewell, W. J. (1988), "Anaerobic Sewage Treatment," *Environmental Science and Technology* 22:14–21.
44. Keeney, D. (1982), "Nitrogen-Availability Indices," in Page, A. L. (ed.), *Methods of Soil Analysis,* Part 2, 2d ed., pp. 711–733, American Society of Agronomy, Madison, Wis.
45. Keeney, D., K. Lee, and L. Walsh (1975), "Guidelines for the Application of Wastewater Sludge to Agricultural Land in Wisconsin," Technical Bulletin 88, Wisconsin Department of Natural Resources, Madison, Wis.

46. Kelling, K. A., A. E. Peterson, L. M. Walsh, J. A. Ryan, and D. R. Keeney (1977), "A Field Study of Agricultural Use of Sewage Sludge: Effect on Crop Yield and Uptake of N and P," *Journal of Environmental Quality* 6:339–345.
47. Lance, J. C., and C. P. Gerba (1984), "Virus Movement in Soil During Saturated and Unsaturated Flow," *Applied and Environmental Microbiology* 47:335–337.
48. Loehr, R., W. Jewell, J. Novak, W. Clarkson, and G. Friedman (1979), *Land Application of Wastes,* Vol. 2, Van Nostrand Reinhold, New York.
49. Loehr, R. C. (1977), *Land as a Waste Management Alternative,* Ann Arbor Science, Ann Arbor, Mich.
50. Logan, T. J., and R. L. Chaney (1983), "Utilization of Municipal Wastewater and Sludge on Land: Metals," pp. 235–326 in A. L. Page et al. (eds.), *Utilization of Municipal Wastewater and Sludge on Land,* Univ. of California, Riverside, Calif.
51. Luthin, J. N. (1973), *Drainage Engineering,* Krieger Publishing Company.
52. Magdoff, F. R., D. Ross, and J. Amadon (1984), "A Soil Test for Nitrogen Availability to Corn," *Soil Science Society of America Journal* 48:1301–1304.
53. McNab, W. H., and C. R. Berry (1985), "Distribution of Aboveground Biomass in Three Pine Species Planted on a Devastated Site Amended with Sewage Sludge or Inorganic Fertilizer," *Forest Science* 31:373–382.
54. National Association of Conservation Districts (1982), "Sludge and the Land: The Role of Soil and Water Conservation Districts in Land Application of Sewage Sludge," U.S. Environmental Protection Agency, 430/9-82-007, Washington.
55. National Research Council (1996), "Use of Reclaimed Water and Sludge in Food Crop Production," National Academy Press, Washington.
56. Ottolenghi, A. C., and V. V. Hamparian (1987), "Multi-year Study of Sludge Application to Farmland: Prevalence of Bacterial Enteric Pathogens and Antibody Status of Farm Families," *Applied and Environmental Microbiology* 53:1118–1124.
57. Power, J. F., and J. Alessi (1971), "Nitrogen Fertilization of Semiarid Grasslands: Plant Growth and Soil Mineral Nitrogen Levels," *Journal of Agronomy* 63:277–280.
58. Rauzi, F., R. L. Lang, and L. I. Painter (1968), "Effects of Nitrogen Fertilization on Native Rangeland," *Journal of Range Management* 21:287–291.
59. Reed, S., R. Crites, and E. Middlebrooks (1994), *Natural Systems for Waste Management and Treatment,* McGraw-Hill, New York.
60. Roberts, J. A., W. L. Daniels, J. C. Bell, and D. C. Martens (1988), "Tall Fescue Production and Nutrient Status on Southwest Virginia Mine Soils," *Journal of Environmental Quality* 17:55–62.
61. Roberts, J. A., W. L. Daniels, J. C. Bell, and J. A. Burger (1988), "Early Stages of Mine Soil Genesis as Affected by Topsoiling and Organic Amendments," *Soil Science Society of America Journal* 52:730–738.
62. Roberts, J. A., W. L. Daniels, J. C. Bell, and J. A. Burger (1988), "Early Stages of Mine Soil Genesis in a Southwest Virginia Mine Spoil Lithosequence," *Soil Science Society of America Journal* 52:716–723.
63. Sabey, B. R., R. L. Pendleton, and B. L. Webb (1990), "Effect of Municipal Sewage Sludge Application on Growth of Two Reclamation Shrub Species in Copper Mine Spoils," *Journal of Environmental Quality* 19:580–586.
64. Seaker, E. M., and W. E. Sopper (1988), "Municipal Sludge for Mine Spoil Reclamation: Effects on Microbial Populations and Activity," *Journal of Environmental Quality* 17:591–597.
65. Sloan, J. J., and N. T. Basta (1995), "Remediation of Acid Soils by Using Alkaline Biosolids," *Journal of Environmental Quality* 24:1097–1103.
66. Sommers, L. E. (1977), "Chemical Composition of Sewage Sludge and Analysis of Their Potential Use as Fertilizers," *Journal of Environmental Quality* 6:225–232.
67. Soon, Y. K., T. E. Bates, and J. R. Moyer (1978), "Land Application of Chemically Treated Sewage Sludge: II. Effects on Plant and Soil Phosphorus, Potassium, Calcium, Magnesium and Soil pH," *Journal of Environmental Quality* 7:269–273.
68. Soon, Y. K., T. E. Bates, and J. R. Moyer (1980), "Land Application of Chemically Treated Sewage Sludge: In Effects on Soil and Plant Heavy Metal Content," *Journal of Environmental Quality* 9:497–504.

69. Soon, Y. K., T. E. Bates, E. G. Beauchamp, and J. R. Moyer (1978), "Land Application of Chemically Treated Sewage Sludge: In Effects on Crop Yield and Nitrogen Availability," *Journal of Environmental Quality* 7:264–269.
70. Sopper, W. (1993), *Municipal Sludge Use in Land Reclamation,* Lewis Publishers, Boca Raton, Fla.
71. Sopper, W. E. (1992), "Reclamation of Mined Land Using Municipal Sludge," *Advances in Soil Science* 17:351–431.
72. Sorber, C. A., B. E. Moore, D. E. Johnson, H. J. Hardy, and R. E. Thomas (1984), "Microbiological Aerosols from the Application of Liquid Sludge to Land," *Journal of Water Pollution Control Federation,* 56(6).
73. South Carolina Department of Health (1987), *Land Application of Sludge Guidance Manual.*
74. Stark, S. A., and C. E. Clapp (1980), "Residual Nitrogen Availability from Soils Treated with Sewage Sludge in a Field Experiment," *Journal of Environmental Quality* 9:505–512.
75. Tchobanoglous, G. (1991), *Wastewater Engineering: Treatment Disposal and Reuse,* 3rd ed., McGraw-Hill, New York.
76. U.S. Army Corps of Engineers (1987), "Wetlands Delineation Manual," Technical Report Y-87-1, Waterways Experiment Station, Vicksburg, Miss.
77. U.S. Department of Agriculture (1994), *Sewage Sludge: Land Utilization and the Environment,* American Society of Agronomy, Crop Science Society of America, Soil Science Society of America, Washington.
78. U.S. Environmental Protection Agency (1983), "Process Design Manual: Land Application of Municipal Sludge," EPA/625/1-83/016, Washington.
79. U.S. Environmental Protection Agency (1979), "NPDES Compliance Sampling Manual," PB81-153215, Washington.
80. U.S. Environmental Protection Agency (1979), "Process Design Manual for Sludge Treatment and Disposal," EPA/625/1-79/011, Washington.
81. U.S. Environmental Protection Agency (1984), "Handbook: Septage Treatment and Disposal," EPA/625/6-84/009, Washington.
82. U.S. Environmental Protection Agency (1985), "Handbook Estimating Sludge Management Costs," EPA-625/6-85010, Washington.
83. U.S. Environmental Protection Agency (1990), "Guidance for Writing Case-by-Case Permit Requirements for Municipal Sewage Sludge," EPA/505/8-90/001, Washington.
84. U.S. Environmental Protection Agency (1990), "Motor Freight Terminals/Railroad Transportation," EPA/530/SW-90/027K, Washington.
85. U.S. Environmental Protection Agency (1991), "Cooperative Testing of Municipal Sewage Sludges by the Toxicity Characteristic Leaching Procedure and Compositional Analysis," EPA 430/09-91-007, Washington.
86. U.S. Environmental Protection Agency (1993), "Domestic Septage Regulatory Guidance—A Guide to The EPA 503 Rule," EPA/832/B-92/005, Washington.
87. U.S. Environmental Protection Agency (1993), "Preparing Sewage Sludge for Land Application or Surface Disposal: A Guide for Preparers of Sewage Sludge on the Monitoring, Recordkeeping, and Reporting Requirements of the Federal Standards for the Use or Disposal of Sewage Sludge, 40 CFR Part 503," EPA/831/B-93/002a, Washington.
88. U.S. Environmental Protection Agency (1993), "Regulatory Impact Analysis of the Part 503 Sewage Sludge Regulation," EPA/821/R-93/006, Washington.
89. U.S. Environmental Protection Agency (1993), "Standards for Use or Disposal of Sewage Sludge. Final Rule, 40 CFR Part 503," *Federal Register* 58(32):9248–9415.
90. U.S. Environmental Protection Agency (1993), "Technical Support Document for Land Application of Sewage Sludge, Vol. I," PB93-110575; Vol. II, PB93110583, Washington.
91. U.S. Environmental Protection Agency (1993), "Standards for the Use or Disposal of Sewage Sludge," *Federal Register* 58(32):9248–9415.
92. U.S. Environmental Protection Agency (1994), "A Plain English Guide to the EPA Part 503 Biosolids Rule," EPA/832/R-93/003, Washington.

93. U.S. Environmental Protection Agency (1994), “Biosolids Recycling: Beneficial Technology for a Better Environment,” EPA/832/R-94/009, Washington.
94. U.S. Environmental Protection Agency (1994), “Guide to Septage Treatment and Disposal,” EPA/625/R-94/002, Washington.
95. U.S. Environmental Protection Agency (1994), “Land Application of Sewage Sludge: A Guide for Land Appliers on the Requirements of the Federal Standards for the Use or Disposal of Sewage Sludge, 40 CFR Part 503,” EPA/831/B-93/002b, Washington.
96. U.S. Environmental Protection Agency (1995), “A Guide to the Biosolids Risk Assessments for the EPA Part 503 Rule,” EPA/832/B-93/005, Washington.
97. U.S. Environmental Protection Agency (1995), “Ground-Water and Leachate Treatment Systems,” EPA/625/R-94/005, Washington.
98. U.S. Environmental Protection Agency (1995), “Part 503 Implementation Guidance,” EPA/833/R-95/001, Washington.
99. U.S. Environmental Protection Agency (1995), “Process Design Manual: Land Application of Sewage Sludge and Domestic Septage,” EPA/625/R-95/001, Washington.
100. U.S. Environmental Protection Agency (1995), “Process Design Manual: Surface Disposal of Sewage Sludge and Domestic Septage,” EPA/625/R-95/002, Washington.
101. U.S. Environmental Protection Agency (1999), “Environmental Regulations and Technology: Control of Pathogens and Vector Attraction in Sewage Sludge,” EPA/625/R-92/013, Washington.
102. Vesilind, P. A., G. C. Hattman, and E. T. Shene (1986), *Sludge Management and Disposal for the Practicing Engineer,* Lewis Publishers, Boca Raton, Fla.
103. Viessman, W., and M. J. Hammer (1985), *Water Supply and Pollution Control,* 4th ed., Harper Collins, New York.
104. Weber, W. (1972), *Physicochemical Processes for Water Quality Control,* Wiley, New York.
105. Wei, Q. F., B. Lowery, and A. E. Peterson (1985), “Effect of Sludge Application on Physical Properties of a Silty Clay Loam Soil,” *Journal of Environmental Quality* 14:178–180.
106. Whitford, W. G., E. A. Aldon, D. W. Freckman, Y. Steinberger, and L. W. Parker (1989), “Effects of Organic Amendments on Soil Biota on a Degraded Rangeland,” *Journal of Range Management* 42:56–60.
107. Yingming, L., and R. B. Corey (1993), “Redistribution of Sludge-Borne Cadmium, Copper, and Zinc in a Cultivated Plot,” *Journal of Environmental Quality* 2:1–8.

Chapter 8

Sampling and Quality Assurance

8.0 Introduction

Throughout the municipal wastewater industry, the terms *sludge* and *biosolids* are often used interchangeably. However, from a regulatory standpoint, only municipal wastewater treatment sludge (including domestic septage) that meets certain quality criteria specified in the 40 CFR Part 503 rule should be designated as biosolids [42,48]. Sludge generated from municipal wastewater treatment operations that does not meet these criteria and/or domestic septage that contains industrial and/or commercial wastes are not considered biosolids.

Sampling is the first step in developing a database to monitor performance or to control processing within a municipal wastewater treatment plant. Within publicly owned treatment works (POTW), both sludge and biosolids are sampled routinely by treatment plant personnel to verify compliance with regulatory limits as well as to evaluate the effectiveness of the various sludge/biosolids processing operations. This chapter provides descriptions of both the general sludge and biosolids sampling procedures typically employed at POTWs and those specific biosolids sampling requirements mandated by the 40 CFR Part 503 rule. Finally, for the management of biosolids beneficial-use operations, descriptions of the general sampling procedures used to monitor the effects of biosolids land-application operations on environmental quality (including soil and vegetation) are provided.

8.1 General Sampling of Sludge and Biosolids

Accurate characterization of sludge/biosolids is required to identify and isolate operational problems at the POTW as well as to signal potential limitations to the use or disposal of biosolids. To the extent practicable, the POTW should have a sludge/biosolids sampling program that adequately addresses the random and cyclic variation in influent wastewater quality and the potential for human exposure to biosolids once they are beneficially used or disposed of [28,29,37].

When developing a sludge/biosolids sampling program, the type of sample to be taken must be selected. In general, the selection of the appropriate sample type is governed by (1) the information sought, (2) the unit process being sampled, (3) the regulatory permit requirements, and (4) the variability of constituents over a period of time. The two types of samples available for sludge/biosolids sampling are the grab and composite samples.

A *grab sample* (sometimes referred to as an *individual discrete sample*) is defined as a single sample that is collected at a particular time and location. In the case of sludge/biosolids sampling, single grab samples will represent only the instantaneous composition of the sludge/biosolids being sampled. Although its composition is only reflective of the sludge/biosolids quality at a specific time and location, a grab sample is often required in certain types of sludge/biosolids tests (e.g., estimation of pathogen densities).

A *composite sample* refers to a mixture of grab samples collected at the same sampling point at different times. The composite sample represents the average conditions of the sludge/biosolids over a particular period of time (typically 24 hours). The analytical results from a composite sample provide a more accurate description of the time- and location-weighted average concentrations present in the sludge/biosolids stream. Examples of sampling situations where a composite sample should be obtained include the following: (1) the average sludge/biosolids conditions must be known for process control, (2) a composite sample is specified in the regulatory permit, or (3) the POTW desires to estimate the treatment plant's overall pollutant-removal efficiency. It should be noted that although a 24-hour composite sample is often used to establish sludge/biosolids quality, these data are only descriptive of the sludge/biosolids quality for that particular day. Historical data are necessary to accurately establish the long-term trend in sludge/biosolids quality [37].

8.1.1 Representative samples

Obtaining a sludge/biosolids sample that reflects the actual compositional characteristics of a particular stream is termed a *representative* *sample*. To effectively sample a sludge/biosolids stream, it is necessary for POTW management personnel to be aware of the physical characteristics of the sludge/biosolids that may affect the collection of a representative sample. Some of the physical characteristics of municipal wastewater sludge are described in Table 8.1.

Two physical characteristics of sludge/biosolids that have an impact on the representativeness of a given sample are its solids content and viscosity. The *solids content* is defined as the percent (by weight) of solid material contained in a given volume of sludge/biosolids [29,30,37]. The sludge/biosolids solids content and the settling behavior that characterizes the solids determine whether a given sample

TABLE 8.1 Physical Characteristics of Various Sludge Types Found at Municipal Wastewater Treatment Plants*

Sludge type	Description
Anaerobically digested sludge	Thick slurry of dark-colored particles and entrained gases. When well digested, anaerobically digested sludge dewaters easily and has a nonoffensive odor. Depending on the mode of operation, the percent solids of anaerobically digested sludges ranges from 4 to 8 percent.
Aerobically digested sludge	Aerobically digested sludge is a dark-brown, flocculent, relatively inert suspension produced by long-term aeration of sludge. Aerobically digested sludge is bulky and generally difficult to thicken. The effluent percent sludge solids from aerobic digestion is less than that of the influent sludge percent solids (if not decanted), since approximately 50 percent of the volatile solids are converted to gaseous end products.
Dewatered sludge	Dewatering converts liquid sludge from a fluid mixture to a cakelike substance. The specific physical characteristics of the dewatered sludge depend on the type of sludge, chemical conditioning, and treatment processes employed. The percent solids content of a dewatered cake ranges from approximately 15 to 50 percent.
Composted sludge	Composting is a process in which the organic material in dewatered sludge undergoes biologic degradation to a stable endproduct. Properly composted sludge is a humuslike material containing 75 to 80 percent solids.
Dried powdered sludge	Dried powdered sludge is the residue from heat drying processes. Sludge drying reduces the sludge's moisture content by vaporization. The moisture content of dried powdered sludge is less than 10 percent.

*Adapted from ref. [37].

will separate (i.e., stratify) into various fractions under normal sampling conditions.

In addition to affecting sludge/biosolids sampling, the sludge solids content also affects the accuracy and precision of some analytical measurements. For example, in measuring pathogen densities, a sludge/biosolids sample with a relatively high solids content normally requires dilution prior to analysis, a preparation step that can lead to an increase in experimental error. Similarly, when attempting to quantify the toxic organic compounds or heavy metal concentrations in sludge/biosolids, analytical precision and accuracy may be adversely affected when analyzing samples with a high solids content due to interfering compounds and/or matrix effects.

Viscosity is defined as the characteristic of a material that causes it to resist flow when subjected to an applied force (e.g., pumping action) [37]. The importance of viscosity in sludge/biosolids sampling stems from the use of certain types of automatic liquid sampling equipment. In general, automatic sampling devices that rely on a vacuum lift to withdraw a sample can be employed for continuous sampling if the sludge/biosolids has a solids content of less than 15 percent. Although these types of systems have been employed for sampling sludge/biosolids flows containing higher concentrations of solids, their application in these situations has proven to be unreliable [37]. For sampling sludges/biosolids with a solids content greater than 15 percent, manual sampling equipment is recommended (Fig. 8.1).

8.1.2 Composite sample development

To develop a composite sample whose compositional characteristics reflect the actual sludge/biosolids conditions, accurate volumetric flow (i.e., gallons per day) and/or solids flux (i.e., pounds per day) data must be obtained. Although volumetric flow data can be developed for a liquid sludge/biosolids flow by installing a reliable flowmeter, obtaining solids flux for liquid sludge/biosolids flow requires having information on both the volumetric flow and the average solids content. Given the availability of accurate flow and solids content data, Eq. (8.1) may be used to estimate an average solids flux for a liquid sludge/biosolids flow stream:

$$\begin{aligned}\text{Solids flux (lb solids/day)} &\\ = \text{average flow rate (MG/day)} &\cdot \text{solids concentration (mg/liter)}\\ &\cdot 8.34 \text{ lb/MG (mg/liter)} \qquad (8.1)\end{aligned}$$

where MG/day is million gallons per day.

For a solid or semisolid sludge/biosolids flow, the solids flux may be estimated using appropriately sized industrial equipment scales.

Figure 8.1 Photograph of a wastewater treatment operator taking a dewatered biosolids/sludge sample. (*Courtesy of the Water Environment Federation.*)

Solids flux data from industrial weighing scales normally are corrected to a dry-mass basis by accounting for the average moisture content of the sludge/biosolids flow. Table 8.2 summarizes the types of measurement equipment typically employed to estimate both the volumetric flow and solids flux rates of sludge/biosolids flows.

Having accurate volumetric flow or solids flux data is particularly important when developing a sampling plan for a sludge/biosolids treatment process in which several streams are either entering or leaving the unit operation. To determine the average condition of a set of multiple sludge/biosolids streams, a composite sample that reflects the compositional characteristics for the confluent sludge stream is developed by taking grab samples from each individual stream. Several alternative approaches exist to generate an accurate composite sample for multiple sludge streams (Table 8.3). Example 8.1 illustrates the application of volumetric flow and solids flux data in preparing a composite sludge/biosolids sample.

Example 8.1 The Rogers County Wastewater Treatment Plant has two anaerobic digesters operating in parallel that discharge their treated effluent to a common drying bed for dewatering. The sludge management personnel desire to develop a 1-gal composite sample that characterizes the average conditions of the combined effluent from both anaerobic digesters. If the first digester (digester A) has an average daily flow rate of 4620

TABLE 8.2 Flow-Measuring Devices

Sludge type	Measurement device: type of measurement
Stabilized liquid sludge	Venturi meter: volumetric flow rate Flow-tube meter: volumetric flow rate Magnetic meter: volumetric flow rate Positive-displacement pump: volumetric flow rate
Thickened sludge	Magnetic meter: volumetric flow rate Positive-displacement pump: volumetric flow rate
Dewatered sludge	Belt press scales: solids flux rate
Dried sludge Composted sludge Thermally reduced sludge	Bulk container or truck scales: solids flux rate

*Adapted from refs. [28,29,37].

TABLE 8.3 Alternatives for Sampling Multiple Sludge Streams*

Alternative 1	The simplest alternative for sampling multiple sludge streams is to withdraw equal volumes of samples from each of the multiple sludge streams to create a composite sample. This approach is justified in the case of multiple sludge/biosolids streams having identical volumetric flow rates and solids content. In the municipal wastewater industry, this type of composite sampling approach is often called a *fixed-volume composite sample.*
Alternative 2	A second alternative for sampling multiple sludge streams is to generate a composite sample by taking grab samples from each sludge flow. In this approach, the size of each grab sample is proportional to the volumetric flow rate of each sludge stream. In the municipal wastewater industry, this type of sample is called a *flow-proportioned composite sample.* This approach to composite sampling can only be used if accurate volumetric flow data exist.
Alternative 3	The final alternative for sampling multiple sludge streams is to generate a composite sample by taking grab samples from each sludge flow. The size of each grab sample should be proportional to the solids flux (lb solids/day) of each sludge stream. In the municipal wastewater industry, this type of sample is called a *solids-proportioned composite sample.* To employ this sampling approach, the average solids concentration and volumetric flow rate for each sludge stream must be estimated. These parameters can be used to estimate the average solids flux for each sludge/biosolids stream using Eq. (8.1).

*Adapted from ref. [37].

gal/day and the second digester (digester B) has an average daily flow rate of 5980 gal/day, estimate the volume of each individual digester sample used to make up the flow-proportioned composite sample. If the percent solids content of the flows from digester A and B are 2 and 1.4 percent, respectively, estimate the volume of each individual digester sample used to make up a solids-proportioned composite sample.

solution

Step 1. Develop a flow-proportioned composite sample based on the average daily digester flow rates:

Total digestive flow (gal/day)

$$\begin{aligned}&= \text{digester A flow (gal/day)} + \text{digester B flow (gal/day)}\\ &= 4620 \text{ gal/day} + 5980 \text{ gal/day}\\ &= 10{,}600 \text{ gal/day}\end{aligned}$$

$$\text{Digester A (fraction of total flow \%)} = \frac{4620 \text{ gal/day}}{10{,}600 \text{ gal/day}} \cdot 100$$

$$= 44 \text{ percent}$$

$$\text{Digester B (fraction of total flow \%)} = \frac{5980 \text{ gal/day}}{10{,}600 \text{ gal/day}} \cdot 100$$

$$= 56 \text{ percent}$$

Step 2. Estimate the volume of the contribution of each digester flow to the 1-gal composite sample:

$$\text{Digester A} = \frac{1 \text{ gal}}{\text{composite sample}} \cdot 0.44 = 0.44 \text{ gal (1.67 liters)}$$

$$\text{Digester B} = \frac{1 \text{ gal}}{\text{composite sample}} \cdot 0.56 = 0.56 \text{ gal (1.67 liters)}$$

Step 3. Develop a solids-proportioned composite sample based on the average daily solids flux rates:

Digester A solids flux (lb solids/day)

$$\begin{aligned}&= \text{average flow rate (MG/day)} \cdot \text{solids concentration (mg/liter)}\\ &\quad \cdot 8.34 \text{ lb/MG (mg/liter)}\\ &= \left(\frac{4620 \text{ gal/day}}{10^6 \text{ gal/MG}}\right) \cdot (20{,}000 \text{ mg/liter}) \cdot 8.34 \text{ lb/MG (mg/liter)}\\ &= 770.62 \text{ lb/day}\end{aligned}$$

Digester B solids flux (lb solids/day)

$$= \text{average flow rate (MG/day)} \cdot \text{solids concentration (mg/liter)}$$

$$\cdot\ 8.34\ \text{lb/MG (mg/liter)}$$

$$= \left(\frac{5980\ \text{gal/day}}{10^6\ \text{gal/MG}}\right) \cdot (14{,}000\ \text{mg/liter}) \cdot 8.34\ \text{lb/MG (mg/liter)}$$

$$= 698.22\ \text{lb/day}$$

$$\text{Total solids flux (lb/day)} = 770.62\ \text{lb/day} + 698.22\ \text{lb/day}$$

$$= 1468.84\ \text{lb/day}$$

Step 4. Estimate the volume of the contribution of each digester flow to the 1-gal composite sample:

$$\text{Digester A} = 1\ \text{gal/composite sample} \cdot \left(\frac{770.62\ \text{lb/day}}{1468.84\ \text{lb/day}}\right)$$

$$= 0.52\ \text{gal (1.97 liters)}$$

$$\text{Digester B} = 1\ \text{gal/composite sample} \cdot \left(\frac{698.22\ \text{lb/day}}{1468.84\ \text{lb/day}}\right)$$

$$= 0.48\ \text{gal (1.82 liters)}$$

NOTE: The solids-proportioned composite sample should be used if the parameter of interest is associated with the solids fraction of the sludge/biosolids flow (e.g., heavy metals).

8.1.3 Sampling location

While some sludge/biosolids pollutants are predominantly associated with the solids fraction (e.g., heavy metals), others are associated with the liquid fraction (e.g., nitrate). For this reason, sludge/biosolids samples should be obtained from locations within the POTW that are well mixed. Failure to acquire a sample with representative solid/liquid fractions can significantly affect the analytical results of a given sample. Since turbulence ensures a well-mixed sample, the following recommendations should be followed when sampling sludge/biosolids: (1) in sludge/biosolids processing trains, samples should be taken from taps on the discharge side of the pumps, and (2) if a sludge/biosolids sample is drawn from a tap on a pipe that is distant from the sludge pumps, the average slow velocity through the pipe should be at least 2 ft/s. Table 8.4 identifies locations within the POTW suitable for obtaining representative sludge/biosolids samples.

TABLE 8.4 Locations for Sampling Various Sludge Types*

Sludge type	Sampling location
Anaerobically digested sludge	Sample from taps on the discharge side of positive-displacement pumps.
Aerobically digested sludge	Sample from taps on discharge lines from pumps. If batch digestion is used, sample directly from the digester. Two considerations with regard to directly sampling the digester include the following: (1) if the digester is aerated during sampling, air entrains in the sample, which may cause the volatile organic compounds (VOCs) to escape when the sample container is opened, and (2) if aeration system is shut off during digester sampling, solids may separate causing stratification of the sludge.
Thickened sludge	Sample from taps on the discharge side of positive-displacement pumps.
Heat-treated sludge	Sample from taps on the discharge side of positive-displacement pumps after decanting.
Filtered (belt or vacuum) or centrifuged sludge	Sample sludge from discharge chute.
Sludge press (plate and frame)	Sample sludge from the storage bin. Select at least four points within the storage bin. Collect equal amounts of sample from each point and combine to generate a composite sample.
Drying beds	Divide the bed into quarters, take equal amounts of sample from the center of each quarter, and combine to form a composite sample for the total bed.
Composted sludge	Sample directly from the front-end loader as the sludge is being discharged into trucks to be hauled away.

*Adapted from refs. [37,47].

8.1.4 Sample size and sampling equipment

With regard to sample size, each analytical method will specify the minimum sample size required for accurate parameter estimation [1,37,46]. However, in the absence of specific sample size requirements, an appropriate sludge/biosolids sample size should be small enough to transfer conveniently and handle carefully in the laboratory but of sufficient size to be representative of the sludge/biosolids stream.

After determining the minimum sample size, the appropriate sampling equipment (manual or automatic) must be chosen. Equipment to sample sludge/biosolids varies with the type of facility and method of sludge/biosolids handling. In general, the following variables will influence the selection of sampling equipment:

1. Location to be sampled
2. Sampler installation (e.g., permanent or temporary)
3. Type of power available
4. Presence of adverse conditions (e.g., explosive or toxic gases)
5. Hydraulic conditions (pressurized flow, open channel, etc.)
6. General nature of sludge/biosolids (e.g., well mixed, stratified)
7. Type of flow proportioning (e.g., volumetric or solids-based)
8. Type of analysis (e.g., pathogens, metals, pH, nutrients, organics, etc.)
9. The period over which samples are to be collected (e.g., hourly, daily, weekly, etc.)
10. Type of sample preservation (e.g., refrigeration or chemical)

To ensure accurate laboratory analysis of sludge/biosolids samples, the sampling equipment must be made of materials that will not contaminate or react with the sample. Suitable materials for sludge/biosolids sampling equipment include (1) Teflon™, (2) glass, and (3) stainless steel. When the cost of Teflon™ and/or stainless steel equipment prohibits or restricts their use, plastic, steel, and/or aluminum may be substituted for most sludge/biosolids sampling activities. It should be noted that if a steel sampling device is used, the equipment should never be galvanized because galvanized materials will readily release zinc into the sample [37,46].

Finally, depending on the sludge/biosolids parameter of interest and the specific circumstances associated with the sampling activities, there is the potential for errors to be introduced during sampling that may affect analytical results. To minimize potential analytical errors and to maintain sample integrity, POTW operators should carefully consider the following aspects of sample management: (1) sample container material, (2) sample container preparation, (3) sample preservation, and (4) holding times prior to analysis. For method-specific details concerning all facets of sample preparation, preservation, holding times, etc., consult refs. [1,37,59].

8.1.4.1 Sampling of liquid sludge/biosolids streams. When sampling liquid sludge/biosolids from pipe taps or the discharge side of pumps (i.e., pressurized lines), 1-liter or 1-qt sampling containers usually suffice for obtaining a representative sample. In some cases (e.g., pathogen density determination), an automatic sampling device can be employed to minimize sample contamination during sampling of pressurized lines (Fig. 8.2).

Figure 8.2 Photograph of an automatic sampling device for use in pressurized sludge/biosolids pipelines. (*Courtesy of Bristol Equipment Company.*)

For sampling liquid sludge/biosolids under unpressurized conditions (e.g., aeration basin), a pond sampling device consisting of a beaker (or bottle) attached to a telescoping tube may be employed (Fig. 8.3).

8.1.4.2 Sampling of high solids sludge/biosolids streams. For sampling high-solids sludge/biosolids that has the consistency of a fluid (e.g., thickened sludge), a pond sampling device (or equivalent) may be employed for taking a representative sample. For sampling dewatered sludge in stockpiles, compost piles, lagoons, or surface impoundments, coring devices should be used. Figure 8.4 provides photographs of several coring devices specifically suited for sludge/biosolids sampling of piles, lagoons, and surface impoundments.

8.1.5 Health and safety considerations

Health and safety considerations are critical in sludge/biosolids sampling activities because of both the potential health-related effects of sludge/biosolids (e.g., presence of pathogens, heavy metals, toxic organics, etc.) and the hazards associated with treatment plant equipment. To minimize the potential effects of pathogen exposure, inoculation is recommended for all POTW personnel who may have direct or incidental contact with sludge/biosolids. At a minimum, inoculations should be given to protect POTW personnel from contracting waterborne diseases such as typhoid and tetanus [37]. The POTW health and safety personnel should emphasize avoidance of direct contact with sludge/biosolids as well as wastewater whenever possible. Direct contact of sludge/biosolids and wastewater can be minimized if certain precautions are taken by POTW personnel such as the wearing of rubber or latex gloves as well as the use waterproof garments when the risk of splashing exists [59].

With regard to minimizing physical safety hazards during sludge/biosolids sampling activities, there are several precautions that

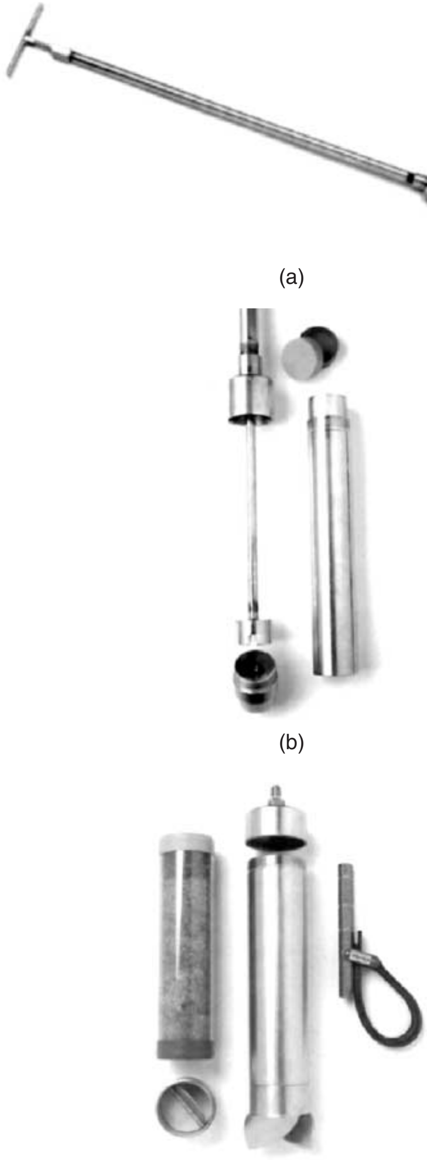
Figure 8.3 Photograph of a pond sampling device for unpressurized sludge/biosolids sampling. (*Courtesy of the Water Environment Federation.*)

should be followed. First, when sampling sludge/biosolids in confined spaces, particularly around anaerobic digesters, it must be remembered that the enclosed atmosphere may be dangerous. Atmospheres in the immediate vicinity of anaerobic digesters may be characterized by the presence of (1) explosive vapors (e.g., methane), (2) poisonous gases (e.g., H_2S, NH_3, etc.), and/or (3) a significant oxygen deficit. Therefore, to minimize the potential of an accident when sampling sludge/biosolids in confined spaces, the sampling area should be adequately ventilated, and the atmosphere should be checked frequently using an appropriate gas detection meter. Moreover, because of the potential for the air quality in confined spaces to become too dangerous to breathe, sampling personnel should be equipped with a supplemental breathing device.

To protect POTW personnel from the potential physical harm from moving equipment, particularly pumps, loose garments should never be worn during sampling. Moreover, to avoid physical injury by high pressure or temperature, sampling personnel must use caution when sampling high-pressure sludge/biosolids lines or lines containing high temperatures (e.g., wet-air oxidation systems).

8.1.6 Sample packaging and shipping

When sludge/biosolids analyses will be performed away from the POTW, samples must be packaged in order to protect them from physical dam-



(a)

(b)

(c)

Figure 8.4 Photographs of coring devices for sampling dewatered sludge/biosolids: (*a*) discrete sludge sampler to obtain sludge/biosolids at desired depth; (*b*) slotted piston for discrete sludge sampler; (*c*) core sludge sampler equipped with one-way valve. (*Courtesy of Art's Manufacturing & Supply, Inc.*)

age and to reduce the risk of leakage during transport. In all cases, sample containers should be positioned upright and cushioned from shock. In addition, sufficient insulation and/or refrigerant should be provided to maintain a sample temperature of 4°C (39°F) for the transport duration. If the samples are transported at temperatures above 4°C, gas production from biologically active samples may cause a significant pressure increase as well as a loss in the volatile solids content.

Under normal circumstances, the process of shipping municipal sludge/biosolids samples does not have to comply with the U.S. Department of Transportation (DOT) hazardous materials regulations. However, should the sample be deemed hazardous, DOT regulations concerning packaging, transportation, and labeling must be followed (49 CFR Parts 172, 173, and 178). Any sludge/biosolids sample may be considered hazardous by DOT if it fails one of the four hazardous characteristic tests, which include (1) corrosivity, (2) ignitability, (3) reactivity, or (4) toxicity [42].

8.1.7 Sample documentation

Adequate documentation of sludge/biosolids sampling activities is important to verify that the appropriate quality assurance/quality control (QA/QC) measures were applied in obtaining, handling, and processing of samples. Proper sampling documentation includes sample labeling, chain-of-custody procedures, and a logbook of sampling activities [37]. With regard to sample labeling, it is important that each sludge/biosolids sample label include the following information:

- Sampling organizational name
- Facility name
- Bottle number
- Sample number
- Type of sample (grab, composite)
- Data and time
- Sample location
- Preservatives
- Analytical parameters
- Collector's name
- Special conditions or remarks

To minimize the loss of any critical sample information, it is important that the ink used to fill out the labels be waterproof and that the labels be secured to sample containers with clear waterproof tape [37].

In addition to including a properly completed label, each sludge/biosolids sample that is transported requires a chain-of-custody document. The chain-of-custody document records each sample's collection and handling history from the time of collection until analysis, as well as the information listed on each sample container [37]. A chain-of-custody document protects the integrity of the sample by ensuring that only authorized persons have custody of the sample, which is necessary if the sample results are to be used in a judicial proceeding alleging a violation in regulatory standards. Finally, all sampling activities should be documented in a bound logbook. The logbook duplicates all information included in the chain-of-custody document and records all relevant observations regarding sampling conditions.

8.2 Quality Assurance and Sampling Frequency

Collection of sludge/biosolids data of known quality is critical both for the regulatory authority to verify facility compliance and for POTW management personnel to assess the effectiveness of various sludge/biosolids treatment operations. It should be noted that two terms that often generate considerable confusion when attempting to define data quality with respect to sludge/biosolids sampling activities are *quality assurance* and *quality control*. *Quality assurance* (QA) is defined by the U.S. Environmental Protection Agency (USEPA) as an integrated system of management activities including planning, implementation, assessment, reporting, and quality improvement that ensures that a process, item, or service is of the type and quality needed and expected by the client [57,58]. Conversely, *quality control* (QC) is defined by the USEPA as an overall system of technical activities that measures the attributes and performance of a process, item, or service against defined standards to verify that they meet the stated requirements established by the customer.

Although there is no regulatory prescribed approach for estimating the number of sludge/biosolids samples necessary to ensure a desired level of data quality, the USEPA requires that sampling activities performed by POTWs on behalf or funded by the USEPA (or its delegated authority) be described by a regulatory-approved quality assurance project plan (QAPP). The purpose of the QAPP is to provide a project-specific "blueprint" for obtaining the type and quality of environmental data needed for a specific decision or use. Therefore, if sludge/biosolids sampling is conducted as part of a regulatory compliance program, a regulatory-approved QAPP must be developed prior to the collection of any environmental data [56]. However, even if the sludge/biosolids sampling activities are being conducted by the POTW to generate data that will only be used internally, the preparation of a QAPP is recommended because it will document that data of the appropriate quality for decision making have been generated.

8.2.1 Quality assurance project plan (QAPP)

For proper development of the QAPP, the USEPA recommends that the QAPP be composed of standardized, recognizable elements covering the entire sampling activity, including planning, implementation, and data assessment (Table 8.5). At a minimum, the QAPP must provide sufficient detail to demonstrate the following:

1. That the project's technical and quality objectives are identified and found to be satisfactory by all relevant stakeholders
2. That the intended measurements or data-acquisition methods are appropriate for achieving sampling objectives
3. That the data-assessment procedures are sufficient for confirming that the data are of the type and quality needed and expected
4. That any limitations on the use of the data are identified and documented

TABLE 8.5 The Four Principal Groups of Elements That Comprise a Quality Assurance Project Plan (QAPP)*

Group	Description
A	*Project management.* This group of QAPP elements covers the basic area of project management, including the project history and objectives, roles and responsibilities of the participants, etc. These elements ensure that the project has a defined goal, that the participants understand the goal and the approach to be used, and that the planning outputs have been documented.
B	*Measurement/data acquisition.* This group of QAPP elements covers all aspects of measurement systems design and implementation, ensuring that the appropriate methods for sampling, analysis, data handling, and quality control are employed and are properly documented.
C	*Assessment/oversight.* This group of QAPP elements addresses the activities for assessing the effectiveness of the implementation of the project and associated QA/QC. The purpose of assessment is to ensure that the QAPP is implement as prescribed.
D	*Data validation and usability.* This group of QAPP elements covers the quality-assurance activities that occur after the data-collection phase of the project is completed. Implementation of these elements ensures that the data conform to the specified criteria, thus achieving the project objectives.

*Adapted from ref. [56].

Of critical importance in developing the QAPP is the regulatory requirement that the quality of environmental data be defined and documented. To define data quality, the USEPA recommends using a systematic approach such as the data-quality objectives (DQO) process or its equivalent [48]. The results of applying any systematic approach to defining data quality will determine the level of detail required in the QAPP. While use of the USEPA's DQO process may not be warranted in the design of all sampling activities, its application does promote careful consideration of the planning steps used to develop the sampling design. The following section provides a brief summary of the USEPA's data-quality objectives (DQO) process.

8.2.2 Data-quality objectives (DQO) process

The data-quality objectives (DQO) process is a systematic planning tool based on the scientific method for establishing criteria for data quality. The data-collection elements for which specific criteria are defined by the DQO process include the following: (1) when and where to collect samples, (2) the tolerable probability limits of decision errors, and (3) the minimum number of monitoring samples needed to satisfy the data-quality criteria. The DQO process consists of seven steps that are illustrated by Fig. 8.5.

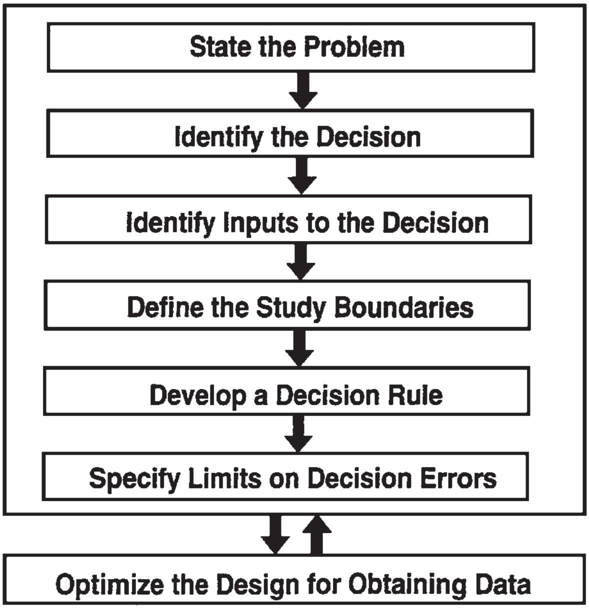


Figure 8.5 Conceptual framework of the data-quality objective process.

Application of the first five steps of the DQO process results in generating descriptive information that addresses the following issues: (1) why the sampling activity is required, (2) what type of medium is to be sampled, and (3) where and when sampling will take place. The information generated in these first five steps of the DQO process is collectively termed the *qualitative data-quality objectives.*

The sixth step of the DQO process, which is termed *specify limits on decision errors,* allows the decision maker to define the probability limits (i.e., quantitative data-quality objectives) of making a decision error based on both the inherent variability in the parameter of concern and the decision maker's level of discomfort in making an incorrect decision. Once the probability limits on decision errors have been defined, Step 7 of the DQO process is employed to estimate the minimum number of samples that must be taken to satisfy the quantitative data-quality objectives. Due to their importance in defining data quality, the following section provides a more detailed description of decision errors and how they may be controlled in the sludge/biosolids sampling activity.

8.2.2.1 Control of decision errors in sampling. Due to both measurement error and/or natural parameter variability, the results of sampling data may lead the decision maker to make an incorrect choice (i.e., decision error) regarding the true state of environmental conditions. While the possibility of making a decision error can never be totally eliminated, it can be controlled. Several approaches are available to reduce the probability of making decision errors, including (1) collecting a large number of samples (which reduces sampling design error), (2) analyzing individual samples several times, and (3) using more precise laboratory methods (which reduces measurement error). It should be noted, however, that implementing approaches to reduce the probability of making a decision error generally increases sampling and analysis costs.

Given the inherent variability in the sampling activity, the probability of making a decision error can be managed by employing a process called *significance testing.* In significance testing, the strength of the sampling data is used to choose between two alternative conditions. The first condition is called the *null hypothesis* H_0 and is defined as the baseline condition that is presumed to be true in the absence of strong evidence to the contrary. The *alternative hypothesis* H_a is chosen if the sampling data were sufficiently convincing to allow the decision maker to reject the null hypothesis.

If the sludge/biosolids sampling activity is associated with regulatory compliance decision making, the baseline or null hypothesis is normally assumed to be the noncompliant condition. Using this approach, the burden of proof is placed on the sampling data to demonstrate that the null hypothesis can be rejected and that a compliant condition actually exists. More important, if the sampling data are inconclusive, the regulatory consequences to the facility will have been minimized because the decision maker initially had presumed that a noncompliant condition had existed and would have taken all the necessary precautions to protect the facility against regulatory action.

A decision error is said to occur when the decision maker, based on sampling data results, rejects the null hypothesis when it is actually true or fails to reject the null hypothesis when it is actually false. More specifically, when the null hypothesis is rejected when it is actually true, the decision maker has committed a false-positive decision error. An example of a false-positive decision error is illustrated by a decision maker who concludes from the sampling data that the mean nickel concentration of land-applied sludge is compliant with the ceiling limits specified in the 40 CFR Part 503 rule when it actually is not. This type of decision error has significant consequences to the decision maker, since committing a false-positive decision error will result in a regulatory violation, which also may include a substantial fine, negative publicity, and/or legal action against the facility. The magnitude of the false-positive decision error, which is also called a *type I error* in statistical texts, is normally referred to as alpha (α).

A false-negative decision error occurs when the null hypothesis is not rejected when it is false. An example of a false-negative decision error would be if the decision maker concludes from the sampling data that the mean nickel concentration of treated municipal sludge is above the ceiling limits specified in the 40 CFR Part 503 rule when it actually is less than the limit. Under these circumstances, sludge that meets the regulatory requirements for biosolids beneficial use would be directed to a disposal option (e.g., landfilling or incineration) at a considerable cost to the POTW. It must be noted that although a false-negative decision error may result in the unnecessary expenditure of resources, it does not normally result in a regulatory violation. The magnitude of the false-negative decision error, which is also called a *type II error* in statistical texts, is normally referred to as beta (ß).

To minimize unnecessary costs and effort in controlling decision errors, the decision maker specifies the tolerable probability limits for making a decision error (e.g., limiting the probability of making a false-positive decision error to 1 percent would be tantamount to assigning α a value of 0.01). Once these tolerable probability limits for making a decision error are defined by the decision maker (i.e., quantitative DQOs), then the number of monitoring samples necessary to satisfy these probability limits in the compliance sampling design can be determined in Step 7 of the DQO process. For example, to estimate the minimum sample size necessary to satisfy the quantitative data-quality objectives when comparing a random sample mean against a regulatory threshold, Eq. (8.2) may be used [48]. Example 8.2 illustrates the use of the DQO process in designing a sludge/biosolids sampling activity.

$$n = \frac{\sigma^2 (Z_{1-\beta} + Z_{1-\alpha})^2}{\Delta^2} + 0.5Z^2_{1-\alpha} \tag{8.2}$$

where n = minimum number of samples required to meet probability limits for decision errors

σ = estimated standard deviation (from pilot study)

σ^2 = estimated variance (from pilot study)

$Z_{1-\beta}$ = the $(1 - \beta)$ percentile of the standard normal distribution (from standard statistical tables)

$Z_{1-\alpha}$ = the $(1 - \alpha)$ percentile of the standard normal distribution (from standard statistical tables)

Δ = minimum detectable difference, width of gray region

Example 8.2 The Carver County Water Reclamation Facility is currently dewatering its sludge using a belt filter press. The sludge management personnel are concerned that the recent increase in the volumetric flow rate from a metal plating facility may cause the facility's dewatered sludge to exceed the 40 CFR Part 503 ceiling concentration limits for nickel. Assuming that over the past 6 months the mean concentration of nickel in the dewatered sludge (dry basis) has been found to be 345 ± 14 mg/kg (standard deviation), use the DQO process to determine the number and frequency of sludge samples that must be taken in the annual monitoring program to ensure that the ceiling concentration limits are not being exceeded at a 99 percent confidence level.

solution

Step 1. State the problem.

- The first step in any decision-making process is to define the problem that has initiated the sampling effort. The four objectives of this first step of the DQO process are described as follows:
  1. Identification of the members of the planning team (e.g., POTW director, POTW sludge/biosolids management supervisor, POTW operators, state regulatory agency, contract laboratory personnel, etc.).
  2. Identification of each member's role during the DQO process and the primary decision maker (e.g., POTW director will serve as the primary decision maker).
  3. Develop a concise description of the problem (e.g., the POTW desires to know with 99 percent confidence whether the dewatered sludge from the belt filter press has a mean nickel concentration that is equal to or below the ceiling limits specified in the 40 CFR Part 503 rule).

4. Specify the available resources and relevant deadlines (e.g., sampling data must be collected and interpreted for inclusion in the annual sludge report to the regulatory authority).

Step 2. Identify the decision.

- The goal of the second step of the DQO process is to define the question that the sampling effort will attempt to resolve and to identify the alternative actions that may be taken based on the outcome of the study. The two specific objectives of this step of the DQO process are described as follows:
  1. Identification of the principal study question (e.g., Can a resource-effective sampling plan be implemented that provides the POTW with a 99 percent confidence level that the mean nickel concentration in the dewatered sludge is below the prescribed ceiling limit specified in the 40 CFR Part 503 rule?).
  2. Define the alternative actions that could result from resolution of the principal study question and combine the principal study question and the alternative actions into a decision statement (e.g., If a resource effective sampling plan can be developed to verify POTW compliance with the ceiling concentration limit for nickel in the dewatered sludge, the plan will be implemented and the results will be included in the annual sludge report, which is submitted to the regulatory agency. If a resource-effective sampling plan cannot be developed to verify POTW compliance with the ceiling concentration limit for nickel in the dewatered sludge, the facility will impose more stringent controls on the local industrial discharge limits for nickel, and in the short term, the resulting sludge will be landfilled rather than used in any beneficial-use programs).

Step 3. Identify the inputs to the decision.

- In Step 3 of the DQO process, the planning team identifies the different types of information that will be needed to resolve the decision statement. The four specific objectives that characterize this step are described as follows:
  1. Identify the information that will be required to resolve the decision statement (e.g., the mean nickel concentration of the dewatered sludge from the belt filter press).
  2. Determine the sources for each item (e.g., random samples of dewatered sludge taken from the belt filter press).
  3. Identify the information that is needed to establish the action level (e.g., the regulatory nickel ceiling limit as specified in the 40 CFR Part 503 rule).
  4. Confirm that the appropriate measurement methods exist to provide the necessary data (nickel concentrations will be determined using analytical methods specified in the 40 CFR Part 503 rule, i.e., EPA methods 3050/3051 for preservation of samples and EPA methods 6010/7520 for analysis of samples).

Step 4. Define the boundaries of the study.

- The purpose of Step 4 of the DQO process is to define the spatial and temporal components of the population that will be covered by the decision statement. The two objectives of this fourth step of the DQO process are described as follows:
    1. Identify the spatial boundaries that define the physical area to be studied and from where the samples should be taken (e.g., The study area will consist of dewatered sludge discharged from the belt filter press. Samples will be taken directly from the sludge discharge bin).
    2. Identify the temporal boundaries that describe the timeframe that the study data will represent and when the samples should be taken (e.g., Samples will be taken quarterly to account for seasonal variations in influent wastewater quality. Since the regulatory agency requires an annual sludge report, results from the sampling activities will only be valid for the 1-year period.)

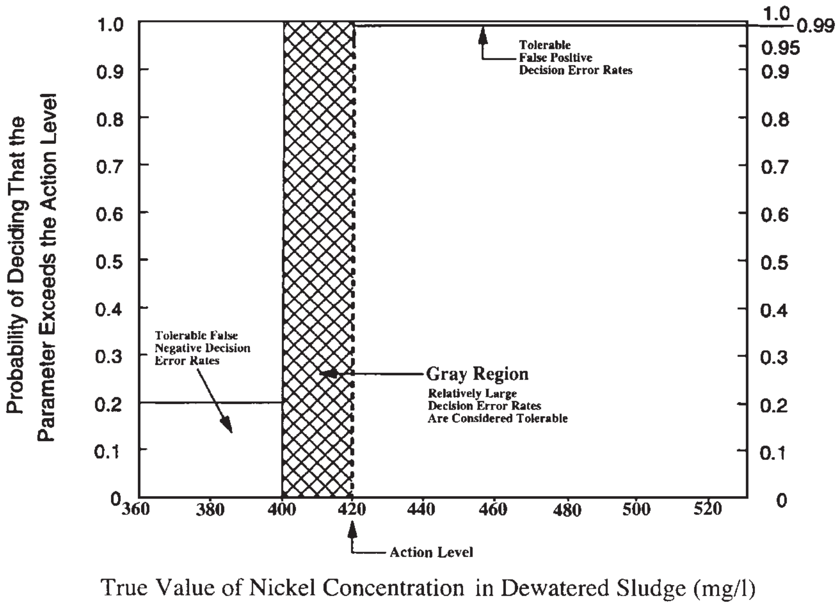
Step 5. Develop a decision rule.

- The purpose of Step 5 of the DQO process is to define the parameter of interest, specify the action level, and integrate previous DQO outputs into a single statement that describes a logical basis for choosing between alternative actions. The three objectives of this step of the DQO process are described as follows.
    1. Specify the statistical parameter that characterizes the population (e.g., mean nickel concentration of the dewatered sludge).
    2. Specify the action level for the study (e.g., the 40 CFR Part 503 rule specifies a ceiling concentration limit for nickel in land-applied sludge of 420 mg/kg).
    3. Develop a decision rule (i.e., an if, …, then statement) that incorporates the parameter of interest, the scale of decision making, the action level, and the actions that would result from resolution of the decision (e.g., If the mean nickel concentration of the dewatered sludge is found through quarterly sampling to be below the ceiling limit of 420 mg/kg, the sludge will be transported to the compost area to be treated for pathogen reduction followed by processing for local sale as an organic fertilizer. If the mean nickel concentration of the dewatered sludge is found through quarterly sampling to be above the ceiling limit of 420 mg/kg, the sludge will be stockpiled, after which it will be transported to a municipal landfill for disposal).

Step 6. Specify the tolerable limits on decision errors.

- The goal of Step 6 of the DQO process is to develop a data-collection design that reduces the probability of making a decision error to a tolerable level. The four objectives of this step of the DQO process are described as follows:
    1. Determine the possible range for the parameter of interest (e.g., from historical records, the nickel concentration of the dewatered sludge has been found to vary from 85 to 395 mg/kg).

2. Identify the null (H_0) and alternative (H_a) hypotheses together with the false-positive and false-negative decision errors.
   - *Null hypothesis* (H_0). The mean nickel concentration of the dewatered sludge from quarterly sampling is above the regulatory ceiling limit of 420 mg/kg.
   - *Alternative hypotheses* (H_a). The mean nickel concentration of the dewatered sludge from quarterly sampling is below the regulatory ceiling limit of 420 mg/kg.
   - *False positive decision error* (α). As a result of the quarterly sampling data, the POTW director concludes that the mean nickel concentration of the dewatered sludge is below the regulatory limit of 420 mg/kg when it is actually above the limit.
   - *False negative decision error* (ß). As a result of the quarterly sampling data, the POTW director concludes that the mean nickel concentration of the dewatered sludge is above the regulatory limit of 420 mg/kg when it is actually below the limit.
3. Specify a range of possible parameter values near the action level where the consequences of false-negative decision errors are relatively minor (gray region). The gray region (or area of uncertainty) establishes the minimum distance from the action level where the decision maker would like to begin to control false-negative decision errors. In statistics, the width of this interval is called the minimum detectable difference and is expressed as the Greek letter delta (Δ).
   - For the sampling of the dewatered sludge, a Δ of 20 mg/kg is chosen by the POTW director. The choice of such a large Δ indicates that the POTW director is not concerned about distinguishing between mean nickel concentrations that are close to the regulatory limit.
4. Assign probability limits to points above and below the gray region that reflect the tolerable probability for the occurrence of decision errors. Use a performance goal diagram to illustrate the quantitative data-quality objectives.
   - Because of the potential of regulatory fines and legal action, the POTW director is concerned about any exceedance of the regulatory limit and therefore has established a false-positive decision error rate (α) of 0.01 (i.e., 99 percent confidence level). Conversely, the POTW director does not want to invest significant resources in controlling the false-negative decision error rate to the same level because this type of decision error will not result in a punitive regulatory action. However, to limit the unnecessary disposal of regulatory-compliant sludge, the POTW director has established a false-negative decision error rate (ß) of 0.20 (i.e., 80 percent confidence level). The results of the selected probability limits on decision errors may be illustrated in the performance goal diagram on the next page.



Step 7. Optimize the design for obtaining compliance data.

- The purpose of Step 7 of the DQO process is to identify the most resource-effective sampling plan that will generate data that satisfy the DQOs specified in the preceding six steps. The two specific objectives of this step of the DQO process are described as follows:
  1. Develop a general data-collection design alternative (e.g., For sampling the dewatered sludge, a simple random sample design is to be employed).
  2. Select the optimal sample size that satisfies the probability limits for decision errors (i.e., quantitative DQOs). Since a random sample design has been chosen and the results will allow comparison of a mean concentration to a regulatory threshold, Eq. (8.2) can be used to estimate the minimum number of samples necessary to satisfy the probability limits on decision errors.

The values for the specific quantitative data quality objectives to be substituted into Eq. (8.2) are summarized as follows:

$Z_{1-\alpha}$ ($Z_{1-0.01}$ or $Z_{0.99}$) = 2.3 (standard statistical tables —99 percent confidence [18])

$Z_{1-\beta}$ ($Z_{1-0.20}$ or $Z_{0.80}$) = 0.85 (standard statistical tables —80 percent confidence [18])

σ = 14 mg/kg (from historical data)

Δ = 20 mg/kg (assigned by POTW director)

The minimum number of samples necessary to satisfy the quantitative data quality objectives is estimated as follows:

$$n = \frac{\sigma^2 (Z_{1-\beta} + Z_{1-\alpha})^2}{\Delta^2} + 0.5Z^2_{1-\alpha}$$

$$= \frac{(14\ \text{mg/kg})^2 (0.85 + 2.3)^2}{(20\ \text{mg/kg})^2} + (0.5)(2.3)^2$$

$$= 7.5\ \text{(choose 8 samples)}$$

Therefore, for the POTW to be 99 percent confident that the nickel concentration in the dewatered sludge from the belt press is below the regulatory ceiling limit, the mean nickel concentration of at least eight samples taken each quarter must be below the regulatory limit of 420 mg/kg. Therefore, a minimum of 32 samples will be taken for the year. It should be noted that even if one or more of the monitoring samples has a nickel concentration that is above the regulatory limit, the POTW may still state in its annual sludge report that it is 99 percent confident that the nickel concentration is compliant with the regulatory ceiling limit if the *mean* of the quarterly samples is below the limit.

8.3 Sampling of Biosolids (40 CFR Part 503 Rule)

The 40 CFR Part 503 rule requires sampling and analysis of biosolids for certain pollutants (metals), pathogens, and vector attraction reduction if the biosolids are applied to land, placed on a surface-disposal site, or incinerated [46]. In most cases, the preparer of biosolids (usually the POTW) will be responsible for biosolids sampling. The 40 CFR Part 503 rule specifies the frequency for biosolids monitoring and lists analytical methods that must be used to analyze the various types of samples. Moreover, the 40 CFR Part 503 rule requires that biosolids meet the specific quality limits at the time of their actual use or disposal or at the time that they are prepared (if distributed in bags or other containers) [46]. It should be noted that although metals determinations can be conducted a considerable time prior to the scheduled biosolids disposal or use activity (e.g., days or weeks), because of the potential of microbial regrowth, pathogen concentrations must be made at the actual time of biosolids disposal or use.

Although the minimum frequency of biosolids monitoring is specified, the 40 CFR Part 503 rule does not, in general, provide instructions as to how biosolids sampling should be conducted. The only exception to this generalization occurs in the case where a POTW desires to demonstrate that its biosolids meet Class B quality through the use of fecal coliform measurements as indicators of biosolids pathogen levels, i.e., Alternative I [53]. For this specific case, the 40 CFR Part 503 rule specifies that a minimum of seven random samples must be taken during each sampling event to demonstrate regulatory compliance [42,46,53].

To develop an effective biosolids sampling and analysis program, a number of factors must be taken into account, including (1) the size of biosolids sample, (2) the accuracy of the analytical techniques, (3) the variability in parameter values, (4) the presence of interfering materials, (5) the stability of the analytes being determined, and (5) the timeframe for parameter determination (e.g., instantaneous, monthly averages, etc.; Table 8.6). While for some of these sampling factors the 40 CFR Part 503 rule provides specific requirements (e.g., minimum size of sample), for most aspects of the biosolids sampling and analysis program, the final decision regarding specific sampling details remains at the discretion of POTW personnel and/or the regulatory authority.

TABLE 8.6 Guidance for Sampling Biosolids*

Use or disposal practice	Parameter	Time frame of determination
Land application	Pollutant limit	
	Ceiling limit	Instantaneous; may not be exceeded
	Pollutant concentration (pollutant limit or EQ)	Monthly averages
	Cumulative pollutant loading rate	May not be exceeded at any site
	Annual pollutant loading rate	May not be exceeded during a 365-day period
Surface disposal	Methane gas	Continuously monitored in air: instantaneous; may not be exceeded
	Metals	Instantaneous; may not be exceeded
Incineration	Metals (except beryllium and mercury)	Daily concentration; if required to report once per month, average of each day operated during the month
	Total hydrocarbons or CO	Continuously monitored; monthly average is reported
	Oxygen	Continuously monitored
	Temperature	Continuously monitored
	Moisture	Continuously monitored

*Adapted from refs. [46,53].

8.3.1 Monitoring frequency

The 40 CFR Part 503 rule specifies the minimum monitoring frequencies for biosolids that will be applied to land, placed on an active surface-disposal site, or incinerated. The frequency of monitoring ranges from once a year for facilities using or disposing relatively small amounts of biosolid (i.e., less than 320 U.S. tons per year) to once a month for facilities using or disposing of larger amounts of biosolids (i.e., more than 16,500 U.S. tons per year). Table 8.7 lists the minimum monitoring frequency for biosolids that are applied to land.

Although Table 8.7 provides the minimum monitoring frequency, the actual monitoring frequency as well as the minimum number of biosolids samples taken during each sampling event should account for the inherent variability in metals concentration, pathogen density, and vector attractiveness. In other words, a sampling plan should be developed and implemented that results in generating monitoring data of a quality appropriate for decision making. An effective method for developing a sampling plan that addresses data quality is the USEPA's data-quality objectives (DQO) process or any other equivalent systematic process for documenting data quality (Fig. 8.5). The benefit of employing a systematic approach for developing a monitoring sampling plan is that the biosolids data quality is established by the decision maker prior to the land application of any biosolids. Moreover, if the sampling plan adopts a statistical approach, the number of samples taken at each sampling event would be a function of the inherent variability of the parameter of interest and the decision maker's desired level of data quality. This type of approach to developing a biosolids sampling plan is recommended because it provides all the relevant stakeholders (e.g., POTW personnel, regulatory authority, public, etc.) with a clear understanding of the level of data quality used in decision making. Example 8.3 illustrates the use of the DQO process for the design of a sampling plan for monitoring the quality of land-applied biosolids.

TABLE 8.7 Frequency of Monitoring Pollutants, Pathogen Densities, and Vector Attraction Reduction for Land-Applied Biosolids*

Annual land-application rate†		
U.S. tons (2000 lb)	Metric tons (1000 kg)	Minimum monitoring frequency
0–320	0–290	Once per year
320–1650	290–1500	Once per quarter (4 times per year)
1650–16,500	1500–15,000	Once per 60 days (6 times per year)
>16,500	>15,000	Once per month (12 times per year)

*Adapted from ref. [46].

†Either the amount of bulk biosolids applied to the land or the amount of biosolids received by a person who prepares the biosolids for sale or give-away in a bag or other container for application to land (dry-weight basis).

Example 8.3 The Odongo County Water Reclamation Facility currently generates 5600 tons (U.S.) of composted biosolids (dry-weight basis) per year, which it desires to sell to local residents as an organic fertilizer. From a recent analysis of the biosolids metal content, nickel was found to limit the annual whole-sludge application rate (AWSAR) to 55 metric tons per hectare. Assuming that over the past 6 months the mean concentration of zinc in the biosolids (dry basis) has been found to be 1120 ± 16 mg/kg (standard deviation), use the DQO process to determine the number of composted biosolids samples that must be taken by POTW personnel at each sampling event if they want to ensure that the recommended AWSAR will not result in an exceedance of the annual pollutant loading rate (APLR) limit at a 95 percent confidence level.

solution

Step 1. State the problem.

1. Members of the planning team—POTW director, POTW sludge/biosolids management supervisor, POTW operators, state regulatory agency, contract laboratory personnel.
2. POTW director will serve as the primary decision maker.
3. *Problem description:* The POTW desires to know with 95 percent confidence whether the AWSAR of 55 metric tons/ha will result in an annual pollutant loading rate (APLR) that is below the regulatory limit for zinc.
4. *Relevant deadlines:* Sampling data must be collected and interpreted for inclusion in the annual biosolids report to the regulatory authority.

Step 2. Identify the decision.

1. *Principal study question:* What number of samples must be taken at each sampling event to provide the POTW with a 95 percent confidence that the AWSAR that it is recommending will not result in an exceedance in the regulatory APLR limits for zinc?
2. *Alternative actions:* If the results of the sampling effort indicate that the AWSAR recommended by the POTW will not result in an exceedance of the APLR with 95 percent confidence, the biosolids will continue to be sold to the general public. If the results of the sampling effort indicate that the AWSAR will result in an exceedance of the APLR for zinc, then the composted biosolids will be landfilled rather than used in any beneficial-use programs.

Step 3. Identify the inputs to the decision.

1. *Information required to resolve the decision statement:* The mean zinc concentration of the composted biosolids.
2. *Sources of information:* Random samples of composted biosolids from the curing pile.
3. *Information needed to establish the action level:* The mean zinc concentration of the composted biosolids and the annual pollutant loading rate (APLR) for zinc.
4. *Methods necessary to provide the required data:* Zinc concentrations will be determined using analytical methods specified in the 40 CFR Part 503 rule, i.e., USEPA methods 3050/3051 for

preservation of samples and USEPA methods 6010/7520 for analysis of samples.

Step 4. Define the boundaries of the study.

1. *Spatial boundary:* The study area will consist of composted biosolids taken directly from the curing pile.
2. *Temporal boundary:* Since the facility generates approximately 5600 tons (U.S.) of composted biosolids per year, the biosolids must be sampled six times per year (once every 60 days). The results of each sampling event will therefore represent a time frame of 2 months.

Step 5. Develop a decision rule.

1. *Statistical parameter that characterizes the population:* The mean zinc concentration of the composted biosolids.
2. *Action level:* Given a regulatory annual pollutant loading rate (APLR) for zinc of 140 metric tons per hectare and an estimated annual whole-sludge application rate of 55 metric tons per hectare, a maximum zinc concentration permitted in the composted biosolids is estimated to be 2545 mg/kg (dry weight). The maximum concentration permitted in the biosolids may be estimated using the following equation:

$$\text{Maximum pollutant concentration in biosolids (mg/kg)} = \frac{\text{annual pollutant loading rate (kg/ha}\cdot\text{yr)}}{\text{annual whole-sludge application rate (mt/ha}\cdot\text{yr)}\cdot 0.001}$$

3. *Decision rule:* If the *mean* zinc concentration of composted biosolids is found during the sampling event to be below 2545 mg/kg, the biosolids will be sold locally as organic fertilizer. If the *mean* zinc concentration of the composted biosolids is found during the sampling event to be above 2545 mg/kg, the biosolids will be stockpiled (stored), after which they will be transported to a municipal landfill for disposal.

Step 6. Specify the tolerable limits on decision errors.

1. *Range for the parameter of interest:* From historical records, the zinc concentration of the composted biosolids has been found to vary from 650 to 1800 mg/kg.
2. Identify the null (H_0) and alternative (H_a) hypotheses together with the false-positive and false-negative decision errors.
   - *Null hypothesis* (H_0). The mean zinc concentration of the composted biosolids from a sampling event is *above* the action level of 2545 mg/kg.
   - *Alternative hypotheses* (H_a). The mean zinc concentration of the composted biosolids from a sampling event is *below* the action level of 2545 mg/kg.
   - *False-positive decision error* (α). As a result of sampling data, the POTW director concludes that the mean zinc concentration of the composted biosolids is *below* the action level of 2545 mg/kg when it is actually *above* the limit.

- *False-negative decision error* (ß). As a result of sampling data, the POTW director concludes that the mean zinc concentration of the composted biosolids is *above* the action level of 2545 mg/kg when it is actually *below* the limit.

3. Specify a range of possible parameter values near the action level where the consequences of false-negative decision errors are relatively minor (gray region). For the sampling of the composted biosolids, a Δ of 15 mg/kg is chosen by the POTW director.
4. *Probability limits:* Because of the potential of regulatory fines and legal action, the POTW director is concerned about exceeding the regulatory limit and therefore has established a false-positive decision error rate (α) of 0.05 (i.e., 95 percent confidence level). Conversely, the POTW director does not want to invest significant resources in controlling the false-negative decision error rate to the same level because this type of decision error will not result in a punitive regulatory action. However, to limit the unnecessary disposal of regulatory compliant biosolids, the POTW director has established a false-negative decision error rate (ß) of 0.20 (i.e., 80 percent confidence level).

Step 7. Optimize the design for obtaining compliance data.

1. *General data-collection design:* For sampling the composted biosolids, a simple *random* sample design is to be employed.
2. *Number of samples per sampling event:* Since a random sample design has been chosen and the results will allow comparison of a mean concentration to a regulatory threshold, Eq. (8.2) can be used to estimate the minimum number of samples necessary to satisfy the probability limits on decision errors.

Quantitative data-quality objectives:

$Z_{1-\alpha}$ ($Z_{1-0.05}$ or $Z_{0.95}$) = 1.65 (standard statistical tables —95 percent confidence [18])

$Z_{1-\beta}$ ($Z_{1-0.20}$ or $Z_{0.80}$) = 0.85 (standard statistical tables —80 percent confidence [18])

σ = 16 mg/kg (from historical composted biosolids data)

Δ = 15 mg/kg (assigned by POTW director)

$$n = \frac{\sigma^2 (Z_{1-\beta} + Z_{1-\alpha})^2}{\Delta^2} + 0.5Z^2_{1-\alpha}$$

$$= \frac{(16\text{ mg/kg})^2 (0.85 + 1.65)^2}{(15\text{ mg/kg})^2} + 0.5\,(1.65)^2$$

$$= 8.47 \text{ (choose 9 samples)}$$

> Therefore, for the POTW to be 95 percent confident that the composted biosolids from the curing pile will not lead to an exceedance in the APLR when the AWSAR is 55 tons per hectare, the *mean* zinc concentration from at least nine random samples taken at each sampling event (i.e., every 2 months) must be below 2545 mg/kg. It should be noted that even if one or more of the samples during each sampling event has a zinc concentration that is above the action level, the POTW may still state in its annual biosolids report that it is 95 percent confident that the AWSAR will not result in a zinc concentration that exceeds the APLR if the *mean* zinc concentration in the land applied biosolids is below 2545 mg/kg.

For surface-disposal sites, the arsenic, chromium, and nickel concentrations must be monitored in biosolids. In addition, depending on the specific surface-disposal option chosen, pathogens, vector attraction reduction, and methane content in air may have to be monitored [55]. The minimum frequency for monitoring biosolids quality depends on the annual amount of biosolids placed in an active biosolids disposal site (Table 8.8).

Although the 40 CFR Part 503 rule specifies the minimum frequency for monitoring the quality of biosolids placed in surface-disposal sites, it does not specify the number of samples to be taken during each sampling event. The actual number of samples taken during each sampling event should be a function of the inherent variability in the parameter of interest and the decision maker's desired level of sampling data quality. To take these factors into account, a sampling plan developed using a systematic approach (e.g., DQO process) could be employed for monitoring the quality of biosolids to be surface disposed.

If biosolids are to be disposed through use of a biosolids incinerator, biosolids must be monitored for arsenic, beryllium, cadmium, chromium, lead, mercury, and nickel. Although it is the prerogative of the regulatory authority to determine how frequently the facility must monitor biosolids and/or the stack gases for beryllium and mercury,

TABLE 8.8 Frequency of Monitoring for Surface Disposal of Biosolids*

Biosolids disposal rate†		
U.S. tons (2000 lb)	Metric tons (1000 kg)	Minimum monitoring frequency
0–320	0–290	Once per year
320–1650	290–1500	Once per quarter (4 times per year)
1650–16,500	1500–15,000	Once per 60 days (6 times per year)
>16,500	>15,000	Once per month (12 times per year)

*Adapted from refs. [46,55].

†Amount of biosolids (other than domestic septage) placed in an active biosolids unit (dry-weight basis).

TABLE 8.9 Frequency of Monitoring for Biosolids Incinerators*

Pollutant parameter	Annual amount of biosolids fired to incinerator: U.S. tons (2000 lb)	Annual amount of biosolids fired to incinerator: Metric tons (1000 kg)	Monitoring frequency
Arsenic, cadmium, chromium, lead, nickel	0–320	0–290	Once per year
	320–1650	290–1500	Once per quarter
	1650–16,500	1500–15,000	Once every 60 days
	>16,500	>15,000	Once per month
Beryllium and mercury in biosolids in stack exit gas	N/A	N/A	Established by permitting authority
Total hydrocarbons or carbon monoxide in stack exit gas	N/A	N/A	Continuously with monthly averages reported
Oxygen concentration in stack exit gas	N/A	N/A	Continuously
Information to determine moisture content in stack exit gas	N/A	N/A	Continuously
Combustion temperature in furnace	N/A	N/A	Continuously
Air pollution control device conditions	N/A	N/A	Established by permitting authority

*Adapted from refs. [46,53].

the frequency for monitoring the concentration of the other metals in biosolids (i.e., arsenic, cadmium, chromium, lead, and nickel) is based on the annual amount of biosolids fired to the incinerator (Table 8.9).

Although the 40 CFR Part 503 rule specifies the minimum frequency for monitoring the quality of biosolids that are incinerated, it does not specify the number of samples to be taken during each sampling event. The actual number of samples taken during each sampling event should be a function of the inherent variability in the parameter of interest and the decision maker's desired level of sampling data quality. It is recommended that the DQO process or any other systematic process for documenting the required data quality be employed for developing a resource-effective sampling plan for monitoring the quality of incinerated biosolids.

8.3.2 Sampling location

While some biosolids pollutants are predominantly associated with the solids fraction (e.g., heavy metals), others are more associated with the liquid fraction (e.g., nitrate). For this reason, biosolids samples should be obtained from locations within the POTW that are well mixed. Depending on the type of biosolids material (liquid, dewatered, or dried) and the treatment process, certain sampling points will provide better samples. Table 8.4 identified possible locations within the POTW suitable for obtaining representative biosolids samples. Finally, it should be noted that if the sampling activity is being conducted to verify regulatory compliance, the sampling location must be documented in a quality-assurance project plan (QAPP). The QAPP should provide specific details regarding how the sampling location(s) was determined.

8.3.3 Sample size and sampling equipment

For the biosolids sampling requirements specified in the 40 CFR Part 503 rule, each analytical method will specify the minimum sample size required for accurate parameter estimation [1,46]. After determining the minimum sample size, the appropriate sampling equipment (manual or automatic) must be chosen based on the factors described previously (Sec. 8.1.4). To ensure accurate laboratory analysis of biosolids samples, the sampling equipment must be made of materials that will not contaminate or react with the sample. Suitable materials for biosolids sampling equipment will be specified in each analytical method [46,53]. Table 8.10 lists the appropriate containers, preservation methods, storage times, and minimum sample volumes for sampling biosolids for metals and pathogens.

8.3.4 Data quality

If sampling results indicated that regulatory quality limits were exceeded after biosolids were applied to land, disposed in a surface-disposal site, or incinerated, the biosolids would have been out of compliance, and the responsible person would be subject to enforcement actions. To minimize the potential of using or disposing biosolids that exceed the regulatory quality limits, use of a quality-assurance project plan (QAPP) is strongly recommended for documenting the sampling monitoring plan (Sec. 8.2.1). For compliance monitoring, it is imperative that a regulatory-approved QAPP be implemented to document that the sampling activity will generate monitoring data of a quality sufficient for decision making [56–58].

8.3.4.1 Analytical methods for biosolids. The 40 CFR Part 503 rule requires that specific methods be employed for analyzing biosolids samples for metals, pathogens, and vector attraction reduction (Table 8.11). In addition to the federal sampling and analysis requirements specified in the 40 CFR Part 503 rule, state regulatory programs may have specific requirements for monitoring biosolids. Prior to the use or

TABLE 8.10 Conditions for Biosolids Sampling*

Parameter	Wide-mouthed container	Preservative	Maximum storage time	Minimum volume
Metals				
Solid samples	Plastic, glass	Cool to 4°C	6 months	300 ml
Liquid samples	Plastic, glass	HNO_3 to pH <2	6 months	1000 ml
Liquid (Hg only)	Plastic, glass	HNO_3 to pH <2	28 days	500 ml
Pathogens	Plastic, glass, presterilized bags, stainless steel	Cool in ice and water to below 10°C if analysis is delayed more than one hour.	6 hours	1–4 liters
		Cool to less than 4°C	24 hours for bacteria and viruses; once month for helminth ova	1–4 liters
		Freeze and store viruses to 0°C	2 weeks	1–4 liters
Vector attraction reduction		Varies with analytical method	Varies with analytical method	Varies with analytical method

*Adapted from ref. [46].

TABLE 8.11 Analytical Methods for Biosolids Sampling*

Sample type	Analytical method
Enteric viruses	ASTM designation: D4994-89, *Standard Practice for Recovery of Viruses from Wastewater Sludges*
Fecal coliform	Part 9221E or Part 922D, *Standard Methods for the Examination of Water and Wastewater,* 18th ed., American Public Health Association
Helminth ova	“Occurrence of Pathogens in Distribution and Marketing Municipal Sludges,” EPA/600/1-87/014
Inorganic pollutants	“Test Methods for Evaluating Solid Wastes, Physical/Chemical Methods,” USEPA Publications SW-846, 3d ed. (1986)
Salmonella sp. bacteria	Part 9260D, *Standard Methods for the Examination of Water and Wastewater,* 18th ed., American Public Health Association
Specific oxygen uptake rate (SOUR)	Part 2710B, *Standard Methods for the Examination of Water and Wastewater,* 18th ed., American Public Health Association
Total, fixed, and volatile solids	Part 2540G, *Standard Methods for the Examination of Water and Wastewater,* 18th ed., American Public Health Association
Percent volatile solids reduction	“Environmental Regulations and Technology: Control of Pathogens and Vectors in Sewage Sludge,” EPA/625/R-92/013

*Adapted from refs. [46,53].

disposal of biosolids, the appropriate regulatory agencies should be contacted by the POTW sludge/biosolids management personnel to identify any applicable monitoring requirements.

8.4 Environmental Sampling at Beneficial-Use Sites

In addition to the monitoring of biosolids quality, regularly scheduled soil sampling and analysis are critical steps necessary for the proper management of biosolids beneficial-use sites. Although soil monitoring is not a specific requirement of the 40 CFR Part 503 rule, soil sampling at biosolids beneficial-use sites is required for determining the soil nutrient levels (e.g., nitrogen, phosphorus, and potassium concentrations) and for estimation of biosolids application rates [54]. In addition to soil nutrient analysis, soils may need to be monitored for soluble salts and/or boron in semiarid regions where irrigation is planned [5].

TABLE 8.12 Soil Parameters Monitored at Beneficial-Use Sites*

Monitoring prior to biosolids land application	Monitoring after biosolids land application
Particle size distribution	pH
pH	Electrical conductivity
Electrical conductivity	Lime requirement (acid soils)
Cation exchange capacity	Plant-available phosphorus (P) and potassium (K)
Lime requirement (acid soils)	Organic nitrogen
Plant-available phosphorus (P) and potassium (K)	Organic matter
Soil nitrogen	
NO_3-N	
NH_4-N	
Organic nitrogen	
Organic matter	
Carbon-nitrogen ratio	

*Adapted from ref. [54].

Table 8.12 summarizes the surface and subsurface soil parameters typically monitored before and after biosolids are applied to land.

After they are collected, soil samples should be air-dried, ground, and passed through a 2-mm sieve [4,5]. Chemical analyses generally are performed on air-dried samples, which do not require special preservation for most parameters. However, soil samples collected for nitrate, ammonia, and pathogen analyses must be refrigerated and analyzed as soon as possible. Standard methods for physical and chemical analyses of soil may be found in the following references [1,5,21].

8.4.1 Soil sampling location

To establish background soil conditions, soil samples should be collected from each field where biosolids are to be applied to land. The number and location of samples necessary to adequately characterize soils prior to biosolids land application are primarily a function of the spatial variability of the soils at the site. General approaches for establishing the average soil conditions (e.g., nutrient levels, pH, cation exchange capacity, etc.) are described in refs. [4,5,11,21]. In some situations, the state regulatory agency stipulates the minimum number of soil borings that must be analyzed to characterize background soil conditions. For example, in New Jersey, a minimum of 3 borings is specified for characterizing background conditions on small sites (i.e., those of 4 ha or less in area), whereas at least 24 borings are required to characterize background soil conditions on large sites (i.e., those having an area of over 80 ha) [5,22,37].

Once initial soil sampling and analysis are completed, the frequency of subsequent sampling will depend on land use and any state regulatory soil-monitoring requirements. For example, on agricultural biosolids beneficial-use sites, nitrogen tests are performed annually, whereas soil pH, phosphorus, and potassium evaluations typically are done every 2 years [22,37,54].

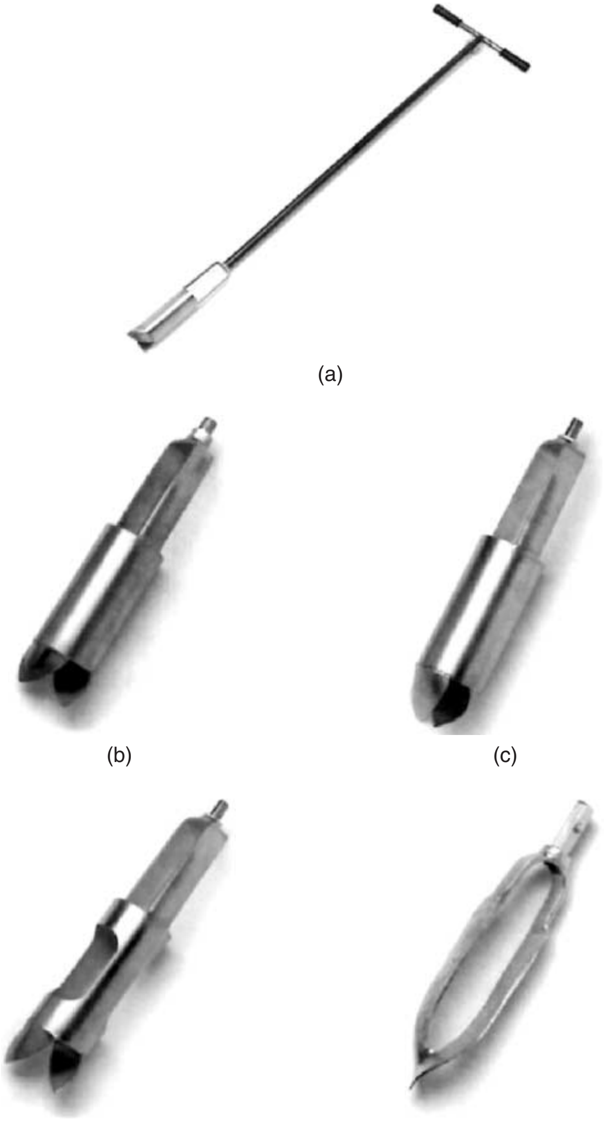
8.4.2 Sampling equipment

The proper selection of soil sampling equipment depends on the following soil characteristics: (1) soil texture, (2) presence of rock fragments, (3) soil depth, and (4) degree of allowable soil surface disturbance. Where field plots are to be sampled periodically, preferable sampling tools are those which disturb the plot the least. Cutaway soil sampling tubes, closed-cylinder augers, and tilling spades may be used depending on the size of the plot and allowable disturbance [5]. The cutaway soil sampling tube creates the least disturbance and works well in the plow layer and the upper subsoil of moist, stone-free, friable soils [4,5]. Figure 8.6 provides photographs of typical soil sampling equipment.

The depth to which the soil profile is sampled and the extent to which each horizon is vertically subdivided depend largely on the parameters to be analyzed, the vertical variations in soil character, and the objectives of the soil sampling program [4,5]. For initial characterization, samples typically are taken from each distinct soil horizon down to a depth of 120 to 150 cm (4–5 ft) [4,5,54]. Figure 8.7 depicts the use of an auger to obtain a soil sample from an agricultural field.

8.4.3 Surface and groundwater monitoring

The risk-based pollutant limits and the management practices for biosolids land application specified in the 40 CFR Part 503 rule were designed to sufficiently protect surface and groundwater so that onsite water quality monitoring is unnecessary. Moreover, by land applying biosolids at agronomic rates, a biosolids agricultural beneficial-use site poses no greater threat of nitrate contamination to surface or groundwater than does the use of conventional nitrogen fertilizers on farmland. However, in some cases, a state agency may require monitoring of surface or groundwater for additional protection of sensitive habitats [44,54]. In these cases, the state usually will specify monitoring locations and procedures. Figure 8.8 provides examples of typical equipment used for sampling surface and groundwater.



(a)

(b) (c)

(d) (e)

Figure 8.6 Photographs of (*a*) telescoping auger; (*b*) regular auger; (*c*) sand auger; (*d*) mud auger; (*e*) Dutch auger. (*Courtesy of Art's Manufacturing and Supply, Inc.*)

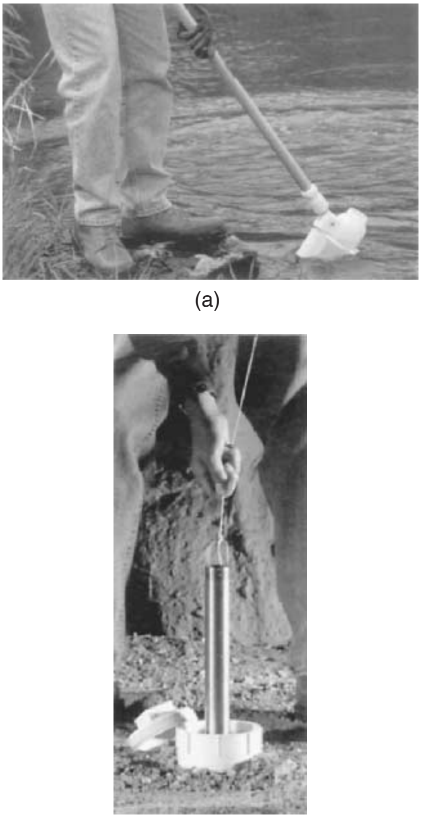
Figure 8.7 Photograph of a soil sample taken from an agricultural field using an auger. (*Courtesy of Art's Manufacturing and Supply, Inc.*)

8.4.4 Vegetation monitoring

The 40 CFR Part 503 rule includes pollutant limits and management practices for land application of biosolids that are designed to be sufficiently protective of vegetative quality to render the monitoring of vegetation unnecessary [54]. However, vegetation monitoring may be conducted for public relations and/or marketing purposes, e.g., when it is desirable to ensure private crop or tree farm owners that their crops will not be adversely affected by the use of biosolids as fertilizers and/or soil conditioners [53,54]. In limited situations, the state regulatory agency may specify vegetation monitoring requirements if there are concerns regarding the uptake of biosolids pollutants by vegetation and its potential impact on critical habitats.

8.4.5 Monitoring and sampling at land reclamation sites

If a biosolids land-application program at a reclamation site complies with applicable requirements, the biosolids will pose little potential for adverse environmental impact, and, therefore, no monitoring beyond that which is specified in the 40 CFR Part 503 rule is necessary.


(a)

(b)

Figure 8.8 Photographs depicting (*a*) pond sampler for surface water sampling; (*b*) stainless steel bailer for groundwater sampling. (*Courtesy of Laboratory Safety Supply, Inc.*)

However, because of the potential of complex site geochemistry, some state regulatory agencies require monitoring at a reclamation site after the biosolids have been applied.

Although standard soil sampling procedures employed on agricultural fields often can be used at reclamation sites, in heterogeneous materials (e.g., mine spoils), determination of soil conditions may require other, more complex sampling approaches [22,54]. In such cases, a site monitoring plan should be designed that takes into account both the variability of biosolids quality and the site heterogeneity. To develop a sampling plan that will generate monitoring data with sufficient quality for decision making, use of a systematic process for designing a data-collection activity (e.g., data-quality objectives process) is recommended.

8.4.5.1 Background sampling. When applying biosolids at rates that exceed the agronomic rate, soil samples should be collected from the site to determine pH, liming requirements, cation exchange capacity, available nutrients, and trace metals prior to biosolids land application [22]. Although monitoring of the biosolids land-application site after biosolids have been applied can vary from no monitoring to extensive sampling, it is generally desirable to analyze the soil after 1 year to document any changes in soil pH. Additional monitoring requirements (including surface and groundwater analyses) will depend on local regulations and site-specific conditions. Some states have specific requirements for monitoring at land reclamation sites, and the biosolids land applier should consult the appropriate regulatory agency prior to developing a biosolids land-application program [54].

8.5 Problems

8.1 The Mandera County Sewer Improvement District is currently pumping thickened sludge from two dissolved-air flotation (DAF) units operating in parallel to a single pasteurization unit for stabilization. If the first DAF unit (unit A) is generating sludge at an average daily flow rate of 1800 gal/day and the second unit (unit B) is generating sludge at an average daily flow rate of 2630 gal/day, estimate the volume of each individual DAF sludge sample that must be taken to develop a 1-gal flow-proportioned composite sample of both DAF units. If the solids content of the sludge generated in the first and second DAF units is 4.8 and 5.4 percent, respectively, estimate the volume of each individual DAF sample needed to make up a 1-gal solids-proportioned composite sample.

8.2 The Eldoret City Wastewater Treatment Plant has four gravity sludge thickeners operating in parallel that discharge into a single anaerobic digester. If the four thickener units (units A, B, C, and D) are generating sludge with an average daily flow rate of 3200, 1950, 2340, and 2500 gal/day, respectively, estimate the volume of each individual digester sample needed to develop a 1-gal flow-proportioned composite sample from all four thickeners. If the solids content of the sludge flows from thickeners A, B, C, and D are 3.8, 4.4, 3.5, and 4.2 percent, respectively, estimate the volume of each individual digester sample used to make up a solids-proportioned composite sample.

8.3 The Atieno County Water Reclamation Plant has received two tanker trucks of domestic septage for discharge into the plant headworks. If the first tanker truck (truck A) has a working volume of 6700 gal and the second tanker truck (truck B) has a working volume of 5500 gal, estimate the volume of each individual septage sample required to develop a 1-gal flow-proportioned composite sample of both trucks. If the solids content of the septage tanker trucks A and B are 7.1 and 6.0 percent, respectively, estimate the volume of the sample from each truck needed to develop a 1-gal solids-proportioned composite sample.

8.4 The Omondi City Wastewater Treatment Plant is concerned that the recent increase in discharge volume from a local metal finishing industry into the municipal sewer may cause the dewatered biosolids to exceed the 40 CFR Part 503 ceiling concentration limits for cadmium. If, over the past year, the mean concentration of cadmium in the dewatered biosolids (dry basis) has been found to be 56.7 ± 2.1 mg/kg (standard deviation), determine the number and frequency of biosolids samples that must be taken during each sampling event to demonstrate to the regulators that the ceiling concentration limits for cadmium are not being exceeded at the 99 percent confidence level. Assume that the facility has established a minimum detectable difference for cadmium of 3.0 mg/kg and a tolerable negative decision error rate of 30 percent.

8.5 The Akinyi City Wastewater Water Reclamation Plant currently generates composted biosolids that it desires to sell to local residents as an organic fertilizer. From a recent analysis of the biosolids metal content, selenium was found to limit the annual whole-sludge application rate (AWSAR) to 12 metric tons/ha. If, over the year, the mean concentration of selenium in the biosolids (dry basis) has been found to be 42.6 ± 1.2 mg/kg (standard deviation), determine the number of composted biosolids samples that must be taken by facility personnel at each sampling event to demonstrate to regulators that the recommended AWSAR does not result in an exceedance of the annual pollutant loading rate (APLR) limit for selenium at the 95 and 99 percent confidence level. Assume that the facility has established a minimum detectable difference for selenium of 1.0 mg/kg and a tolerable negative decision error rate of 20 percent.

8.6 References

1. American Public Health Association (1985), *Standard Methods for the Examination of Water and Wastewater,* APHA, Washington.
2. Bastian, R. K. (1997), "Biosolids Management in the United States," *Water Environment and Technology,* pp. 45–50.
3. Bohn, H. L., B. L. McNeal, and G. A. O'Connor (1979), *Soil Chemistry,* Wiley, New York.
4. Brady, N. C. (1984), *The Nature and Properties of Soil,* 9th ed., Macmillan, New York.
5. Carter, M., ed. (1993), *Soil Sampling and Methods of Analysis,* Lewis Publishers, Boca Raton, Fla.
6. Coker, E. G. (1983), "The Use of Sewage Sludge in Agriculture," *Water Science Technology* 15:195–208.
7. Crohn, D. M. (1996), "Planning Biosolids Land Application Rates for Agricultural Systems," *Journal of Environmental Engineering,* pp. 1058–1066.
8. Cuenca, R. H. (1989), *Irrigation System Design: An Engineering Approach,* Prentice-Hall, Englewood Cliffs, N.J.
9. Garvey, D., C. Guarino, and R. Davis (1993), "Sludge Disposal Trends Around the Globe," *Water/Engineering & Management,* pp. 17–20.
10. Huddleston, J. H., and M. P. Ronayne (1990), "Guide to Soil Suitability and Site Selection for Beneficial Use of Sewage Sludge," Oregon State University Extension Service.
11. Keeney, D. (1982), "Nitrogen-Availability Indices," in Page, A. L., ed., *Methods of Soil Analysis,* part 2, 2d ed., pp. 711–733, American Society of Agronomy, Madison, Wis.

12. Keeney, D., K. Lee, and L. Walsh (1975), "Guidelines for the Application of Wastewater Sludge to Agricultural Land in Wisconsin," Technical Bulletin 88, Wisconsin Department of Natural Resources, Madison, Wis.
13. Kelling, K. A., A. E. Peterson, L. M. Walsh, J. A. Ryan, and D. R. Keeney (1977), "A Field Study of Agricultural Use of Sewage Sludge: Effect on Crop Yield and Uptake of N and P," *Journal of Environmental Quality* 6:339–345.
14. Loehr, R., W. Jewell, J. Novak, W. Clarkson, and G. Friedman (1979), *Land Application of Wastes,* vol. 2, Van Nostrand Reinhold, New York.
15. Loehr, R. C. (1977), *Land as a Waste Management Alternative,* Ann Arbor Science, Ann Arbor, Mich.
16. Luthin, J. N. (1973), *Drainage Engineering,* Krieger Publishing Company.
17. Magdoff, F. R., D. Ross, and J. Amadon (1984), "A Soil Test for Nitrogen Availability to Corn," *Soil Science Society of America Journal* 48:1301–1304.
18. Moore, D. S., and G. P. McCabe (1993), *Introduction to the Practice of Statistics,* WH Freeman and Company, San Francisco.
19. National Research Council (1996), "Use of Reclaimed Water and Sludge in Food Crop Production," National Academy Press, Washington.
20. Ryan, J., and R. Chaney (1993), "Regulation of Municipal Sewage Sludge Under the Clean Water Act Section 503: A Model for Exposure and Risk Assessment for MSW-Compost," in *Science and Engineering of Composting,* Renaissance Publications, Worthington, Ohio.
21. Sander, D. H., D. T. Walthers, and K. D. Frank (1994), "Nitrogen Testing for Optimum Management," *Journal of Soil and Water Conservation* 49(2):46–52.
22. Sopper, W. (1993), *Municipal Sludge Use in Land Reclamation,* Lewis Publishers, Boca Raton, Fla.
23. South Carolina Department of Health (1987), *Land Application of Sludge Guidance Manual,* SCDH.
24. Sposito, G. (1989), *The Chemistry of Soils,* Oxford University Press, New York.
25. Tchobanoglous, G. (1991), *Wastewater Engineering: Treatment Disposal and Reuse,* 3d ed., McGraw-Hill, New York.
26. U.S. Army Corps of Engineers (1987), "Wetlands Delineation Manual," Technical Report Y-87-1, Waterways Experiment Station, Vicksburg, Miss.
27. U.S. Department of Agriculture (1994), *Sewage Sludge: Land Utilization and the Environment,* American Society of Agronomy, Crop Science Society of America, Soil Science Society of America, Washington.
28. U.S. Environmental Protection Agency (1979), "Inspectors Guide for Evaluation of Municipal Wastewater Treatment Plants," EPA/430/9-79/010, Washington.
29. U.S. Environmental Protection Agency (1979), "NPDES Compliance Sampling Manual," PB81-153215, Washington.
30. U.S. Environmental Protection Agency (1979), "Process Design Manual for Sludge Treatment and Disposal," EPA/625/1-79/011, Washington.
31. U.S. Environmental Protection Agency (1983), "Process Design Manual for Land Application of Municipal Sludge," EPA 625/1-83-016, Washington.
32. U.S. Environmental Protection Agency (1984), "Environmental Regulations and Technology: Use and Disposal of Municipal Wastewater Sludge," EPA/625/10-841/003, Washington.
33. U.S. Environmental Protection Agency (1984), "Handbook: Septage Treatment and Disposal," EPA/625/6-84/009, Washington.
34. U.S. Environmental Protection Agency (1985), "Environmental Regulations and Technology: The Electroplating Industry," EPA/625/10-85/001, Washington.
35. U.S. Environmental Protection Agency (1986), "Regrowth of *Salmonellae* in Composted Sewage Sludge," USEPA 600/2-86/106 (NTISPB 87-129532/AS), Springfield, Va.
36. U.S. Environmental Protection Agency (1987), "Guidance Manual on the Development and Implementation of Local Discharge Limitations under the Pretreatment Program," PB92-129188, Washington.
37. U.S. Environmental Protection Agency (1989), "POTW Sludge Sampling and Analysis Guidance Document," NTIS PB93227957, Washington.

38. U.S. Environmental Protection Agency (1990), "40 CFR Part 503, National Sewage Sludge Survey: Availability of Information and Data and Anticipated Impacts on Proposed Regulations," *Federal Register* 55(218):47210–47283.
39. U.S. Environmental Protection Agency (1991), "Residential and Commercial Toxic Pollutant Loadings and POTW Removal Efficiency," PB93-209872, Washington.
40. U.S. Environmental Protection Agency (1993), "Domestic Septage Regulatory Guidance: A Guide to The EPA 503 Rule," EPA/832/B-92/005, Washington.
41. U.S. Environmental Protection Agency (1993), "Preparing Sewage Sludge for Land Application or Surface Disposal: A Guide for Preparers of Sewage Sludge on the Monitoring, Recordkeeping, and Reporting Requirements of the Federal Standards for the Use or Disposal of Sewage Sludge, 40 CFR Part 503," EPA/831/B-93/002a, Washington.
42. U.S. Environmental Protection Agency (1993), "Regulatory Impact Analysis of the Part 503 Sewage Sludge Regulation," EPA/821/R-93/006, Washington.
43. U.S. Environmental Protection Agency (1993), "Standards for Use or Disposal of Sewage Sludge. Final Rule, 40 CFR Part 503," *Federal Register* 58(32):9248–9415.
44. U.S. Environmental Protection Agency (1993), "Technical Support Document for Land Application of Sewage Sludge," vol. I, PB93-110575; vol. II, PB93110583, Washington.
45. U.S. Environmental Protection Agency (1993), "Standards for the Use or Disposal of Sewage Sludge," *Federal Register* 58(32):9248–9415.
46. U.S. Environmental Protection Agency (1994), "A Plain English Guide to the EPA Part 503 Biosolids Rule," EPA/832/R-93/003, Washington.
47. U.S. Environmental Protection Agency (1994), "Biosolids Recycling: Beneficial Technology for a Better Environment," EPA/832/R-94/009, Washington.
48. U.S. Environmental Protection Agency (1994), "Guidance for the Data Quality Objectives Process: EPA QA/G-4," EPA/600/R-96/055, Washington.
49. U.S. Environmental Protection Agency (1994), "Guide to Septage Treatment and Disposal," EPA/625/R-94/002, Washington.
50. U.S. Environmental Protection Agency (1994), "Land Application of Sewage Sludge: A Guide for Land Appliers on the Requirements of the Federal Standards for the Use or Disposal of Sewage Sludge, 40 CFR Part 503," EPA/831/B-93/002b, Washington.
51. U.S. Environmental Protection Agency (1995), "A Guide to the Biosolids Risk Assessments for the EPA Part 503 Rule," EPA/832/B-93/005, Washington.
52. U.S. Environmental Protection Agency (1995), "Ground-Water and Leachate Treatment Systems," EPA/625/R-94/005, Washington.
53. U.S. Environmental Protection Agency (1995), "Part 503 Implementation Guidance," EPA/833/R-95/001, Washington.
54. U.S. Environmental Protection Agency (1995), "Process Design Manual: Land Application of Sewage Sludge and Domestic Septage," EPA/625/R-95/001, Washington.
55. U.S. Environmental Protection Agency (1995), "Process Design Manual: Surface Disposal of Sewage Sludge and Domestic Septage," EPA/625/R-95/002, Washington.
56. U.S. Environmental Protection Agency (1998), "Guidance for Quality Assurance Project Plans: EPA QA/G-5," EPA/600/R-98/018, Washington.
57. U.S. Environmental Protection Agency (1998), "EPA Quality Systems Requirements for Environmental Programs," EPA QA/R-1, Washington.
58. U.S. Environmental Protection Agency (1998), "EPA Requirements for Quality Management Plans," EPA QA/R-2, Washington.
59. U.S. Environmental Protection Agency (1999), "Environmental Regulations and Technology: Control of Pathogens and Vector Attraction in Sewage Sludge," EPA/625/R-92/013, Washington.