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Wastewater Treatment Processes and Systems

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This course was adapted from the USEPA Onsite Wastewater Treatment Systems Manual, Publication No. EPA/625/R-00/008, "Treatment Processes and Systems", Chapter 4, which is in the public domain.

Chapter 4

Treatment processes and systems

- 4.1 Introduction
- 4.2 Conventional systems and treatment options
- 4.3 Subsurface wastewater infiltration
- 4.4 Design considerations
- 4.5 Construction management and contingency options
- 4.6 Septic tanks
- 4.7 Sand/media filters
- 4.8 Aerobic Treatment Units

4.1 Introduction

This chapter contains information on individual onsite/decentralized treatment technologies or unit processes. Information on typical application, design, construction, operation, maintenance, cost, and pollutant removal effectiveness is provided for most classes of treatment units and their related processes. This information is intended to be used in the preliminary selection of a system of treatment unit processes that can be assembled to achieve predetermined pollutant discharge concentrations or other specific performance requirements. Complete design specifications for unit processes and complete systems are not included in the manual because of the number of processes and process combinations and the wide variability in their application and operation under various site conditions. Designers and others who require more detailed technical information are referred to such sources.

Chapter 4 is presented in two main sections. The first section contains information about *conventional* (soil-based or subsurface wastewater infiltration) systems, referred to as SWISs in this document. Both gravity-driven and mechanized SWISs are covered in this section of chapter 4. The second section contains a general introduction to sand filters (including other media), and a series of fact sheets on treatment technologies, *alternative* systems (e.g., fixed-film and suspended growth systems, evapotranspiration systems, and other applications), and special issues pertaining to the design, operation, and maintenance of onsite wastewater treatment systems (OWTSSs). This

approach was used because the conventional system is the most economical and practical system type that can meet performance requirements in many applications.

The first section is further organized to provide information about the major components of a conventional system. Given the emphasis in this manual on the design boundary (performance-based) approach to system design, this section was structured to lead the reader through a discussion of system components by working backwards from the point of discharge to the receiving environment to the point of discharge from the home or other facility served by the onsite system. Under this approach, soil infiltration issues are discussed first, the distribution piping to the infiltration system including graveless systems is addressed next, and matters related to the most common preliminary treatment device, the septic tank, are covered last.

The fact sheets in the second section of this chapter describe treatment technologies and discuss special issues that might affect system design, performance, operation, and maintenance. These treatment technologies are often preceded by a septic tank and can include a subsurface wastewater infiltration system. Some treatment technologies may be substituted for part or all of the conventional system, though nearly all alternative approaches include a septic tank for each facility being served. Fact sheets are provided for the more widely used and successful treatment technologies, such as sand filters and aerobic treatment units.

The component descriptions provided in this chapter are intended to assist the reader in screening components and technologies for specific applications. Chapter 5 presents a strategy and procedures that can be used to screen and select appropriate treatment trains and their components for specific receiver sites. The reader should review chapter 5 before selecting system components.

4.2 Conventional systems and treatment options

The three primary components of a conventional system (figure 4-1) are the soil, the subsurface wastewater infiltration system (SWIS; also called a leach field or infiltration trench), and the septic tank. The SWIS is the interface between the engineered system components and the receiving ground water environment. It is important to note that the performance of conventional systems relies primarily on treatment of the wastewater effluent in the soil horizon(s) below the dispersal and infiltration components of the SWIS. Information on SWIS siting, hydraulic and mass loadings, design and geometry, distribution methods, and construction considerations is included in this chapter. The other major component of a conventional system, the septic tank, is characterized by describing its many functions in an OWTS.

Treatment options include physical, chemical, and biological processes. Use of these options is determined by site-specific needs. Table 4-1 lists

common onsite treatment processes and methods that may be used alone or in combination to assemble a treatment train capable of meeting established performance requirements. Special issues that might need to be addressed in OWTS design include treatment of high-strength wastes (e.g., biochemical oxygen demand and grease from schools and restaurants), mitigation of impacts from home water softeners and garbage disposals, management of holding tanks, and additives (see related fact sheets).

4.3 Subsurface wastewater infiltration

Subsurface wastewater infiltration systems (SWISs) are the most commonly used systems for the treatment and dispersal of onsite wastewater. Infiltrative surfaces are located in permeable, unsaturated natural soil or imported fill material so wastewater can infiltrate and percolate through the underlying soil to the ground water. As the wastewater infiltrates and percolates through the soil, it is treated through a variety of physical, chemical, and biochemical processes and reactions.

Many different designs and configurations are used, but all incorporate soil infiltrative surfaces that are located in buried excavations (figure 4-1). The primary infiltrative surface is the bottom of the excavation, but the sidewalls also may be used for infiltration. Perforated pipe is installed to distribute the wastewater over the infiltration surface. A porous

Figure 4-1. Conventional subsurface wastewater infiltration system

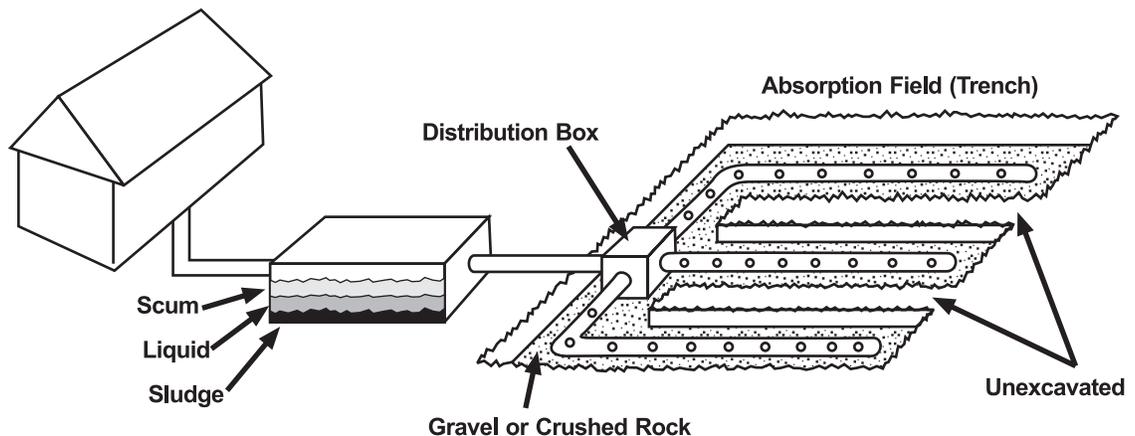


Table 4-1. Commonly used treatment processes and optional treatment methods

Treatment objective	Treatment process	Treatment methods
Suspended solids removal	Sedimentation	Septic tank Free water surface constructed wetland Vegetated submerged bed
	Filtration	Septic tank effluent screens Packed-bed media filters (incl. dosed systems) Granular (sand, gravel, glass, bottom ash) Peat, textile Mechanical disk filters Soil infiltration
Soluble carbonaceous BOD and ammonium removal	Aerobic, suspended-growth reactors	Extended aeration Fixed-film activated sludge Sequencing batch reactors (SBRs)
	Fixed-film aerobic bioreactor	Soil infiltration Packed-bed media filters (incl. dosed systems) Granular (sand, gravel, glass) Peat, textile, foam Trickling filter Fixed-film activated sludge Rotating biological contactors
	Lagoons	Facultative and aerobic lagoons Free water surface constructed wetlands
Nitrogen transformation	Biological Nitrification (N) Denitrification (D)	Activated sludge (N) Sequencing batch reactors (N) Fixed film bio-reactor (N) Recirculating media filter (N, D) Fixed-film activated sludge (N) Anaerobic upflow filter (N) Anaerobic submerged media reactor (D) Submerged vegetated bed (D) Free-water surface constructed wetland (N, D)
	Ion exchange	Cation exchange (ammonium removal) Anion exchange (nitrate removal)
Phosphorus removal	Physical/Chemical	Infiltration by soil and other media Chemical flocculation and settling Iron-rich packed-bed media filter
	Biological	Sequencing batch reactors
Pathogen removal (bacteria, viruses, parasites)	Filtration/Predation/Inactivation	Soil infiltration Packed-bed media filters Granular (sand, gravel, glass bottom ash) Peat, textile
	Disinfection	Hypochlorite feed Ultraviolet light
Grease removal	Flotation	Grease trap Septic tank
	Adsorption	Mechanical skimmer
	Aerobic biological treatment (incidental removal will occur; overloading is possible)	Aerobic biological systems

medium, typically gravel or crushed rock, is placed in the excavation below and around the distribution piping to support the pipe and spread the localized flow from the distribution pipes across the excavation cavity. Other gravelless or “aggregate-free” system components may be substituted. The porous medium maintains the structure of the excavation, exposes the applied wastewater to more infiltrative surface, and provides storage space for the wastewater within its void fractions (interstitial spaces, typically 30 to 40 percent of the volume) during peak flows with gravity systems. A permeable geotextile fabric or other suitable material is laid over the porous medium before the excavation is backfilled to prevent the introduction of backfill material into the porous medium. Natural soil is typically used for backfilling, and the surface of the backfill is usually slightly mounded and seeded with grass.

Subsurface wastewater infiltration systems provide both dispersal and treatment of the applied wastewater. Wastewater is transported from the infiltration system through three zones (see chapter 3). Two of these zones, the infiltration zone and vadose zone, act as fixed-film bioreactors. The infiltration zone, which is only a few centimeters thick, is the most biologically active zone and is often referred to as the “biomat.” Carbonaceous material in the wastewater is quickly degraded in this zone, and nitrification occurs immediately below this zone if sufficient oxygen is present. Free or combined forms of oxygen in the soil must satisfy the oxygen demand generated by the microorganisms degrading the materials. If sufficient oxygen is not present, the metabolic processes of the microorganisms can be reduced or halted and both treatment and infiltration of the wastewater will be adversely affected (Otis, 1985). The vadose (unsaturated) zone provides a significant pathway for oxygen diffusion to re-aerate the infiltration zone (Otis, 1997, Siegrist et al., 1986). Also, it is the zone where most sorption reactions occur because the negative moisture potential in the unsaturated zone causes percolating water to flow into the finer pores of the soil, resulting in greater contact with the soil surfaces. Finally, much of the phosphorus and pathogen removal occurs in this zone (Robertson and Harman, 1999; Robertson et al., 1998; Rose et al., 1999; Yates and Yates, 1988).

4.3.1 SWIS designs

There are several different designs for SWISs. They include trenches, beds, seepage pits, at-grade

systems, and mounds. SWIS applications differ in their geometry and location in the soil profile. Trenches have a large length-to-width ratio, while beds have a wide, rectangular or square geometry. Seepage pits are deep, circular excavations that rely almost completely on sidewall infiltration. Seepage pits are no longer permitted in many jurisdictions because their depth and relatively small horizontal profile create a greater point-source pollutant loading potential to ground water than other geometries. Because of these shortcomings, seepage pits are not recommended in this manual.

Infiltration surfaces may be created in natural soil or imported fill material. Most traditional systems are constructed below ground surface in natural soil. In some instances, a restrictive horizon above a more permeable horizon may be removed and the excavation filled with suitable porous material in which to construct the infiltration surface (Hinson et al., 1994). Infiltration surfaces may be constructed at the ground surface (“at-grades”) or elevated in imported fill material above the natural soil surface (“mounds”). An important difference between infiltration surfaces constructed in natural soil and those constructed in fill material is that a secondary infiltrative surface (which must be considered in design) is created at the fill/natural soil interface. Despite the differences between the types of SWISs, the mechanisms of treatment and dispersal are similar.

4.3.2 Typical applications

Subsurface wastewater infiltration systems are passive, effective, and inexpensive treatment systems because the assimilative capacity of many soils can transform and recycle most pollutants found in domestic and commercial wastewaters. SWISs are the treatment method of choice in rural, unsewered areas. Where point discharges to surface waters are not permitted, SWISs offer an alternative if ground water is not closely interconnected with surface water. Soil characteristics, lot size, and the proximity of sensitive water resources affect the use of SWISs. Table 4-2 presents characteristics for typical SWIS applications and suggests applications to avoid. Local codes should be consulted for special requirements, restrictions, and other relevant information.

Table 4-2. Characteristics of typical SWIS applications

Characteristic	Typical application	Applications to avoid ^a
Type of wastewater	Domestic and commercial (residential, mobile home parks, campgrounds, schools, restaurants, etc.)	Facilities with non-sanitary and/or industrial wastewaters. Check local codes for other possible restrictions
Daily flow	< 20 population equivalents unless a management entity exists	> 20 population equivalents without a management program. Check local codes for specific or special conditions (e.g., USEPA or state Underground Injection Control Program Class V rule)
Minimum pretreatment	Septic tank, Imhoff tank	Discharge of raw wastewater to SWIS
Lot orientation	Loading along contour(s) must not exceed the allowable contour loading rate	Any site where hydraulic loads from the system will exceed allowable contour loading rates
Landscape position	Ridge lines, hilltops, shoulder/side slopes	Depressions, foot slopes, concave slopes, floodplains
Topography	Planar, mildly undulating slopes of $\leq 20\%$ grade	Complex slopes of $> 30\%$
Soil texture	Sands to clay loams	Very fine sands, heavy clays, expandable clays
Soil structure	Granular, blocky	Platy, prismatic, or massive soils
Drainage	Moderately drained or well drained sites	Extremely well, somewhat poor, or very poorly drained sites
Depth to ground water or bedrock	> 5 feet	< 2 feet. Check local codes for specific requirements.

^aAvoid when possible.

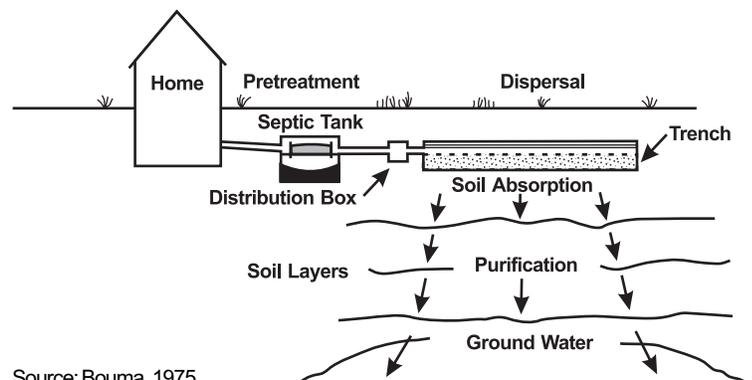
Source: Adapted from WEF, 1990.

4.3.3 Typical performance

Results from numerous studies have shown that SWISs achieve high removal rates for most wastewater pollutants of concern (see chapter 3) with the notable exception of nitrogen. Biochemical oxygen demand, suspended solids, fecal indicators, and surfactants are effectively removed within 2 to 5 feet of unsaturated, aerobic soil (figure 4-2). Phosphorus and metals are removed through adsorption, ion exchange, and precipitation reactions. However, the retention capacity of the soil is finite and varies with soil mineralogy, organic content, pH, redox potential, and cation exchange capacity. The fate of viruses and toxic organic compounds has not been well documented (Tomson et al., 1984). Field and laboratory studies suggest that the soil is quite effective in removing viruses, but some types of viruses apparently are able to leach from SWISs to the ground water. Fine-textured soils, low hydraulic loadings, aerobic subsoils, and high temperatures favor destruction of viruses and toxic organics. The most significant documented threats to ground water quality from

SWISs are nitrates. Wastewater nitrogen is nearly completely nitrified below properly operating SWISs. Because nitrate is highly soluble and environments favoring denitrification in subsoil are limited, little removal occurs (see chapter 3). Chlorides also leach readily to ground water because they, too, are highly soluble and are nonreactive in soil.

Figure 4-2. Lateral view of conventional SWIS-based system



Dispersion of SWIS percolate in the ground water is often minimal because most ground water flow is laminar. The percolate can remain for several hundred feet as a distinct plume in which the solute concentrations remain above ambient ground water concentrations (Robertson et al., 1989, Shaw and Turyk, 1994). The plume descends in the ground water as the ground water is recharged from the surface, but the amount of dispersion of the plume can be variable. Thus, drinking water wells some distance from a SWIS can be threatened if they are directly in the path of a percolate plume.

4.4 Design considerations

Onsite wastewater treatment system designs vary according to the site and wastewater characteristics encountered. However, all designs should strive to incorporate the following features to achieve satisfactory long-term performance:

- Shallow placement of the infiltration surface (≤ 2 feet below final grade)
- Organic loading comparable to that of septic tank effluent at its recommended hydraulic loading rate
- Trench orientation parallel to surface contours
- Narrow trenches (≤ 3 feet wide)
- Timed dosing with peak flow storage
- Uniform application of wastewater over the infiltration surface
- Multiple cells to provide periodic resting, standby capacity, and space for future repairs or replacement

Based on the site characteristics, compromises to ideal system designs are necessary. However, the designer should attempt to include as many of the above features as possible to ensure optimal long-term performance and minimal impact on public health and environmental quality.

4.4.1 Placement of the infiltration surface

Placement of a SWIS infiltration surface may be below, at, or above the existing ground surface (in an in-ground trench, at grade, or elevated in a

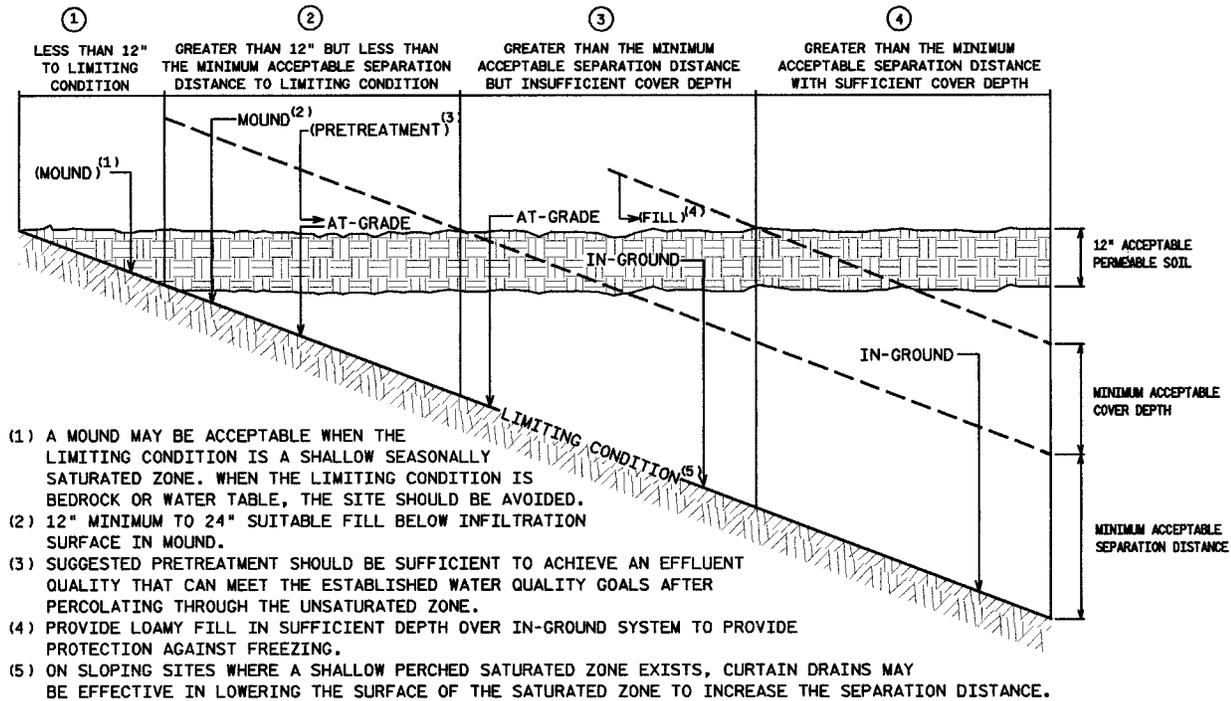
mound system). Actual placement relative to the original soil profile at the site is determined by desired separation from a limiting condition (figure 4-3). Treatment by removal of additional pollutants during movement through soils and the potential for excessive ground water mounding will control the minimum separation distance from a limiting condition. The depth below final grade is affected by subsoil reaeration potential. Maximum delivery of oxygen to the infiltration zone is most likely when soil components are shallow and narrow and have separated infiltration areas. (Erickson and Tyler, 2001).

4.4.2 Separation distance from a limiting condition

Placement of the infiltration surface in the soil profile is determined by both treatment and hydraulic performance requirements. Adequate separation between the infiltration surface and any saturated zone or hydraulically restrictive horizon within the soil profile (secondary design boundary as defined in section 5.3.1) must be maintained to achieve acceptable pollutant removals, sustain aerobic conditions in the subsoil, and provide an adequate hydraulic gradient across the infiltration zone. Treatment needs (performance requirements) establish the minimum separation distance, but the potential for ground water mounding or the availability of more permeable soil may make it advantageous to increase the separation distance by raising the infiltration surface in the soil profile.

Most current onsite wastewater system codes require minimum separation distances of at least 18 inches from the seasonally high water table or saturated zone irrespective of soil characteristics. Generally, 2- to 4-foot separation distances have proven to be adequate in removing most fecal coliforms in septic tank effluent (Ayles Associates, 1993). However, studies have shown that the applied effluent quality, hydraulic loading rates, and wastewater distribution methods can affect the unsaturated soil depth necessary to achieve acceptable wastewater pollutant removals. A few studies have shown that separation distances of 12 to 18 inches are sufficient to achieve good fecal coliform removal if the wastewater receives additional pretreatment prior to soil application (Converse and Tyler, 1998a, 1998b; Duncan et al., 1994). However, when effluents with lower organic and

Figure 4-3. Suggested subsurface infiltration system design versus depth (below the original ground surface) to a limiting condition



Source: Otis, 2001.

oxygen-demanding content are applied to the infiltration surface at greater hydraulic loading rates than those typically used for septic tank effluents (during extended periods of peak flow), treatment efficiency can be lost (Converse and Tyler, 1998b, Siegrist et al., 2000).

Reducing the hydraulic loading rate or providing uniform distribution of the septic tank effluent has been shown to reduce the needed separation distance (Bomblat et al., 1994; Converse and Tyler, 1998a; Otis, 1985; Siegrist et al., 2000; Simon and Reneau, 1987). Reducing both the daily and instantaneous hydraulic loading rates and providing uniform distribution over the infiltration surface can help maintain lower soil moisture levels. Lower soil moisture results in longer wastewater retention times in the soil and causes the wastewater to flow through the smaller soil pores in the unsaturated zone, both of which enhance treatment and can reduce the necessary separation distance.

Based only on hydraulics, certain soils require different vertical separation distances from ground

water to avoid hydrologic interference with the infiltration rate. From a treatment standpoint, required separation distances are affected by dosing pattern, loading rate, temperature, and soil characteristics. Uniform, frequent dosing (more than 12 times/day) in coarser soils maximizes the effectiveness of biological, chemical, and physical treatment mechanisms. To offset inadequate vertical separation, a system designer can raise the infiltration surface in an at-grade system or incorporate a mound in the design. If the restrictive horizon is a high water table and the soil is porous, the water table can be lowered through the use of drainage tile or a curtain drain if the site has sufficient relief to promote surface discharge from the tile piping. For flat terrain with porous soils, a commercial system has been developed and is being field tested. It lowers the water table with air pressure, thereby avoiding any aesthetic concerns associated with a raised mound on the site. Another option used where the terrain is flat and wet is pumped drainage surrounding the OWTS (or throughout the subdivision) to lower the seasonal high water table and enhance aerobic conditions beneath the

drainfield. These systems must be properly operated by certified operators and managed by a public management entity since maintenance of off-lot portions of the drainage network will influence performance of the SWIS.

The hydraulic capacity of the site or the hydraulic conductivity of the soil may increase the minimum acceptable separation distance determined by treatment needs. The soil below the infiltration surface must be capable of accepting and transmitting the wastewater to maintain the desired unsaturated separation distance at the design hydraulic loading rate to the SWIS. The separation distance necessary for satisfactory hydraulic performance is a function of the permeability of the underlying soil, the depth to the limiting condition, the thickness of the saturated zone, the percentage of rocks in the soil, and the hydraulic gradient. Ground water mounding analyses may be necessary to assess the potential for the saturated zone to rise and encroach upon the minimum acceptable separation distance (see section 5.4). Raising the infiltration surface can increase the hydraulic capacity of the site by accommodating more mounding. If the underlying soil is more slowly permeable than soil horizons higher in the profile, it might be advantageous to raise the infiltration surface into the more permeable horizon where higher hydraulic loading rates are possible (Hoover et al., 1991; Weymann et al., 1998). A shallow infiltration system covered with fill or an at-grade system can be used if the natural soil has a shallow permeable soil horizon (Converse et al., 1990; Penninger, and Hoover, 1998). If more permeable horizons do not exist, a mound system constructed of suitable sand fill (figure 4-4) can provide more permeable material in which to place the infiltration surface.

4.4.3 Depth of the infiltration surface

The depth of the infiltration surface is an important consideration in maintaining adequate subsoil aeration and frost protection in cold climates. The maximum depth should be limited to no more than 3 to 4 feet below final grade to adequately re-aerate the soil and satisfy the daily oxygen demand of the applied wastewater. The infiltrative surface depth should be less in slowly permeable soils or soils with higher ambient moisture. Placement below this depth to take advantage of more permeable

soils should be resisted because re-aeration of the soil below the infiltration surface will be limited. In cold climates, a minimum depth of 1 to 2 feet may be necessary to protect against freezing. Porous fill material can be used to provide the necessary cover even with an elevated (at-grade or mound) system if it is necessary to place the infiltration surface higher.

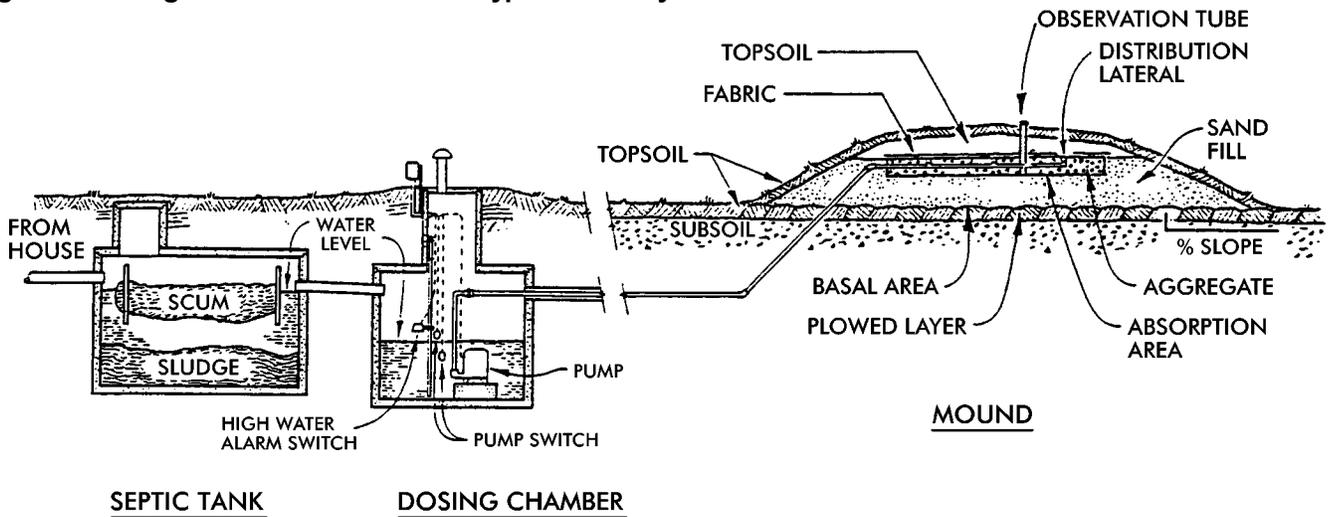
4.4.4 Subsurface drainage

Soils with shallow saturated zones sometimes can be drained to allow the infiltration surface to be placed in the natural soil. Curtain drains, vertical drains, underdrains, and mechanically assisted commercial systems can be used to drain shallow water tables or perched saturated zones. Of the three, curtain drains are most often used in onsite wastewater systems to any great extent. They can be used effectively to remove water that is perched over a slowly permeable horizon on a sloping site. However, poorly drained soils often indicate other soil and site limitations that improved drainage alone will not overcome, so the use of drainage enhancements must be carefully considered. Any sloping site that is subject to frequent inundation during prolonged rainfall should be considered a candidate for upslope curtain drains to maintain unsaturated conditions in the vadose zone.

Curtain drains are installed upslope of the SWIS to intercept the permanent and perched ground water flowing through the site over a restrictive horizon. Perforated pipe is laid in the bottom of upslope trenches excavated into the restrictive horizon. A durable, porous medium is placed around the piping and up to a level above the estimated seasonally high saturated zone. The porous medium intercepts the ground water and conveys it to the drainage pipe (figure 4-5). To provide an outfall for the drain, one or both ends of the pipe are extended downslope to a point where it intercepts the ground surface. When drainage enhancements are used, the outlet and boundary conditions must be carefully evaluated to protect local water quality.

The drain should avoid capture of the SWIS percolate plume and ground water infiltrating from below the SWIS or near the end of the drain. A separation distance between the SWIS and the drain that is sufficient to prevent percolate from the

Figure 4-4. Raising the infiltration surface with a typical mound system.



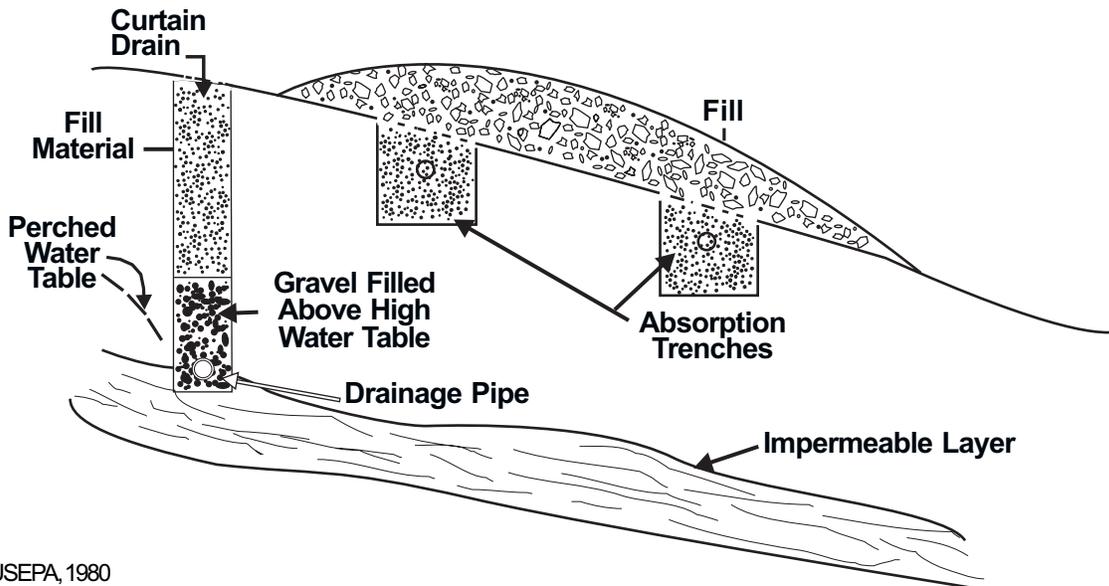
Source: ASAE, Converse and Tyler, 1998b.

SWIS from entering the drain should be maintained. The vertical distance between the bottom of the SWIS and the drain and soil permeability characteristics should determine this distance. As the vertical distance increases and the permeability decreases, the necessary separation distance increases. A 10-foot separation is used for most applications. Also, if both ends of the drain cannot be extended to the ground surface, the upslope end should be extended some distance along the surface contour beyond the end of the SWIS. If not done,

ground water that seeps around the end of the drain can render the drain ineffective. Similar cautions should be observed when designing and locating outlet locations for commercial systems on flat sites.

The design of a curtain drain is based on the permeability of the soil in the saturated zone, the size of the area upslope of the SWIS that contributes water to the saturated zone, the gradient of the drainage pipe, and a suitable outlet configuration.

Figure 4-5. Schematic of curtain drain construction



Source: USEPA, 1980

If the saturated hydraulic conductivity is low and the drainable porosity (the percentage of pore space drained when the soil is at field capacity) is small, even effectively designed curtain drains might have limited effect on soil wetness conditions. Penninger et al. (1998) illustrated this at a site with a silty clay loam soil at field capacity that became completely re-saturated with as little as 1-inch of precipitation. Figure 4-6 provides a useful design chart that considers most of these parameters. For further design guidance, refer to the U.S. Department of Agriculture's *Drainage of Agricultural Land* (USDA, 1973).

4.4.5 Sizing of the infiltration surface

The minimum acceptable infiltration surface area is a function of the maximum anticipated daily wastewater volume to be applied and the maximum instantaneous and daily mass loading limitations of the infiltration surface (see chapter 5). Both the bottom and sidewall area of the SWIS excavation can be infiltration surfaces; however, if the sidewall is to be an active infiltration surface, the bottom surface must pond. If continuous ponding of the infiltration surface persists, the infiltration zone will become anaerobic, resulting in loss of hydraulic capacity. Loss of the bottom surface for infiltration will cause the ponding depth to increase over time as the sidewall also clogs (Bouma, 1975; Keys et al., 1998; Otis, 1977). If allowed to continue,

hydraulic failure of the system is probable. Therefore, including sidewall area as an active infiltration surface in design should be avoided. If sidewall areas are included, provisions should be made in the design to enable removal of the ponded system from service periodically to allow the system to drain and the biomat to oxidize naturally.

Design flow

An accurate estimation of the design flow is critical to infiltration surface sizing. For existing buildings where significant changes in use are not expected, water service metering will provide good estimates for design. It is best to obtain several weeks of metered daily flows to estimate daily average and peak flows. For new construction, water use metering is not possible and thus waste flow projections must be made based on similar establishments. Tables of "typical" water use or wastewater flows for different water use fixtures, usage patterns, and building uses are available (see section 3.3.1). Incorporated into these guidelines are varying factors of safety. As a result, the use of these guides typically provides conservatively high estimates of maximum peak flows that may occur only occasionally. It is critical that the designer recognizes the conservativeness of these guides and how they can be appropriately adjusted because of their impacts on the design and, ultimately, performance of the system.

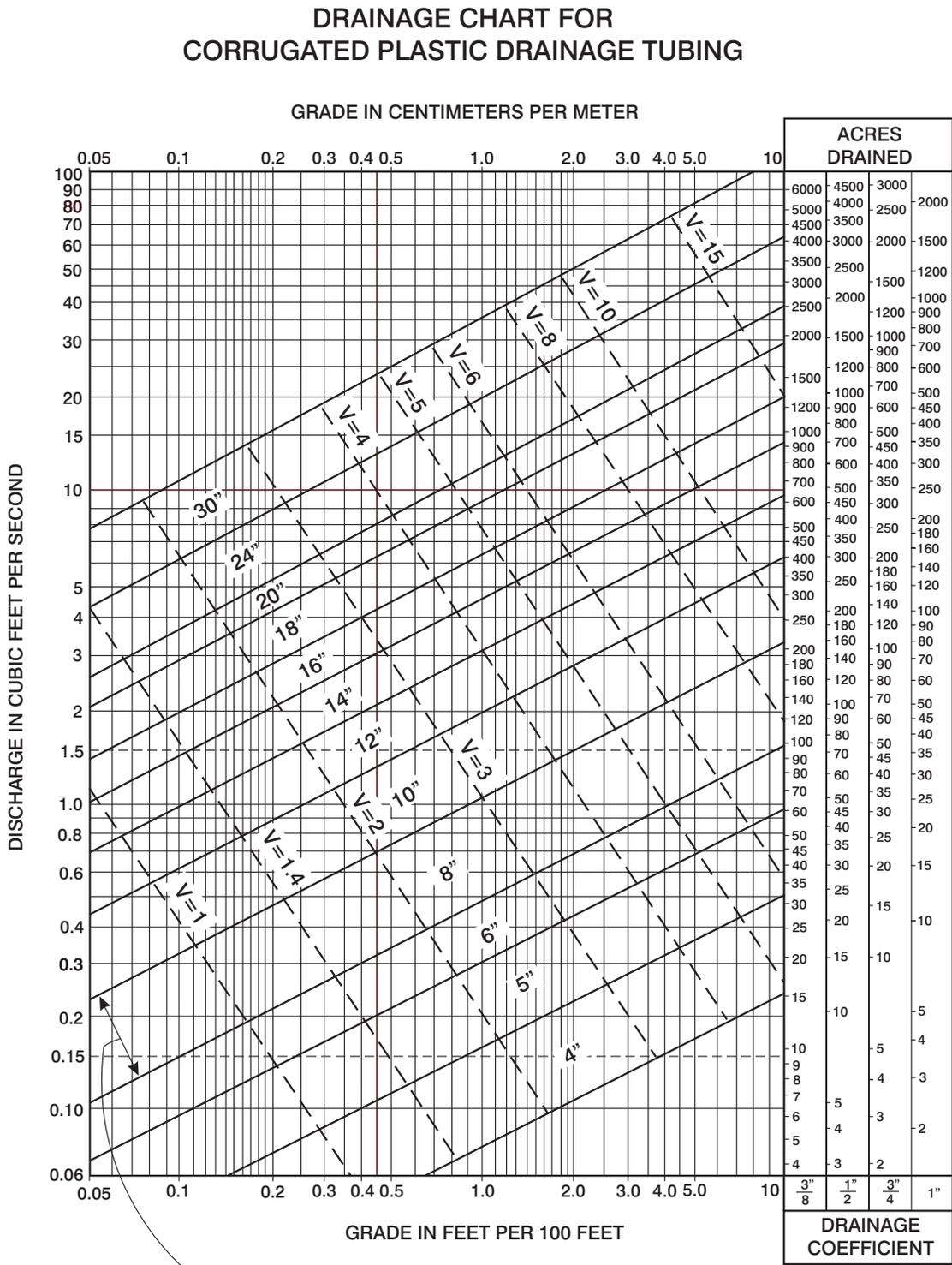
Curtain drain design

Curtain drain design (see preceding figures) is dependent on the size of the contributing drainage area, the amount of water that must be removed, the soil's hydraulic properties, and the available slope of the site.

The contributing drainage area is estimated by outlining the capture zone on a topographic map of the site. Drainage boundaries are determined by extending flow lines perpendicular to the topographic contours upslope from the drain to natural divides (e.g., ridge tops) or natural or man-made "no-flow" boundaries (e.g., rock outcrops, major roads). The amount of water that must be removed is an estimate of the volume of precipitation that would be absorbed by the soil after a rainfall event. This is called the *drainage coefficient*, which is expressed as the depth of water to be removed over a specified period of time, typically 24 hours. Soil structure, texture, bulk density, slope, and vegetated cover all affect the volume of water to be drained.

The slope of the drain can be determined after the upslope depth of the drain invert and the outfall invert are established. These can be estimated from the topographic map of the site. The contributing drainage area, water volume to be removed, and slope of the drain are estimated. Figure 4-6 can be used to determine the drain diameter. For example, the diameter of a curtain drain that will drain an area upslope of 50 acres with a drainage coefficient of $\frac{3}{4}$ inch on a slope of 5 percent would be 8 inches (see figure). At 0.5 percent, the necessary drain diameter would be 12 inches.

Figure 4-6. Capacity chart for subsurface drains



Space between lines is the range of drain capacity for the size shown between lines

V = velocity in feet per second
n = 0.015

Source: USDA, 1973.

Infiltration surface loading limitations

Infiltration surface hydraulic loading design rates are a function of soil morphology, wastewater strength, and SWIS design configuration. Hydraulic loadings are traditionally used to size infiltration surfaces for domestic septic tank effluent. In the past, soil percolation tests determined acceptable hydraulic loading rates. Codes provided tables that correlated percolation test results to the necessary infiltration surface areas for different classes of soils. Most states have supplemented this approach with soil morphologic descriptions. Morphologic features of the soil, particularly structure, texture, and consistence, are better predictors of the soil’s hydraulic capacity than percolation tests (Brown et al., 1994; Gross et al., 1998; Kleiss and Hoover,

1986; Simon and Reneau, 1987; Tyler et al., 1991; Tyler and Converse, 1994). Although soil texture analysis supplemented the percolation test in most states by the mid-1990s, soil structure has only recently been included in infiltrative surface sizing tables (table 4-3). Consistence, a measure of how well soils form shapes and stick to other objects, is an important consideration for many slowly permeable soil horizons. Expansive clay soils that become extremely firm when moist and very sticky or plastic when wet (exhibiting firm or extremely firm consistence) are not well suited for SWISs.

Not all soil conditions are represented in table 4-3, which is a generic guide to the effects of soil properties on the performance of SWISs. Also

Table 4-3. Suggested hydraulic and organic loading rates for sizing infiltration surfaces

Texture	Structure		Hydraulic loading (gal/ft ² -day)		Organic loading (lb BOD/1000ft ² -day)	
	Shape	Grade	BOD=150	BOD=30	BOD=150	BOD=30
Coarse sand, sand, loamy coarse sand, loamy sand	Single grain	Structureless	0.8	1.6	1.00	0.40
Fine sand, very fine sand, loamy fine sand, loamy very fine sand	Single grain	Structureless	0.4	1.0	0.50	0.25
Coarse sandy loam, sandy loam	Massive	Structureless	0.2	0.6	0.25	0.15
	Platy	Weak	0.2	0.5	0.25	0.13
		Moderate, strong				
	Prismatic, blocky, granular	Weak	0.4	0.7	0.50	0.18
Moderate, strong		0.6	1.0	0.75	0.25	
Fine sandy loam, very fine sandy loam	Massive	Structureless	0.2	0.5	0.25	0.13
	Platy	Weak, mod., strong				
		Weak	0.2	0.6	0.25	0.15
Loam	Prismatic, blocky, granular	Moderate, strong	0.4	0.8	0.50	0.20
		Weak	0.4	0.6	0.50	0.15
	Massive	Structureless	0.2	0.5	0.25	0.13
Silt loam	Platy	Weak, mod., strong				
		Weak	0.4	0.6	0.50	0.15
	Moderate, strong	0.6	0.8	0.75	0.20	
Sandy clay loam, clay loam, silty clay loam	Prismatic, blocky, granular	Structureless				
		Weak, mod., strong				
	Weak	0.2	0.3	0.25	0.08	
Sandy clay, clay, silty clay	Platy	Moderate, strong	0.4	0.6	0.50	0.15
		Weak, mod., strong				
	Prismatic, blocky, granular	Weak				
		Moderate, strong	0.2	0.3	0.25	0.08

Source: Adapted from Tyler, 2000.

available are many other state and local guides that include loadings for soils specific to local geomorphology. North Carolina, for example, uses the *long-term acceptance rate* (LTAR) for soil loadings, which is the volume of wastewater that can be applied to a square foot of soil each day over an indefinite period of time such that the effluent from the onsite system is absorbed and properly treated (North Carolina DEHNR, 1996). In the North Carolina rules, LTAR and loading rate values are the same.

Increasingly, organic loading is being used to size infiltration surfaces. Based on current understanding of the mechanisms of SWIS operation, organic loadings and the reaeration potential of the subsoil to meet the applied oxygen demand are critical considerations in successful SWIS design. Anaerobic conditions are created when the applied oxygen demand exceeds what the soil is able to supply by diffusion through the vadose zone (Otis, 1985, 1997; Siegrist et al., 1986). The facultative and anaerobic microorganisms that are able to thrive in this environment are less efficient in degrading the waste materials. The accumulating waste materials and the metabolic by-products cause soil clogging and loss of infiltrative capacity.

Further, higher forms of soil fauna that would help break up the biomat (e.g., worms, insects, non-wetland plants) and would be attracted to the carbon and nutrient-rich infiltration zone are repelled by the anoxic or anaerobic environment. If wastewater application continues without ample time to satisfy the oxygen demand, hydraulic failure due to soil clogging occurs. Numerous studies have shown that wastewaters with low BOD concentrations (e.g., < 50 mg/L) can be applied to soils at rates 2 to 16 times the typical hydraulic loading rate for domestic septic tank effluent (Jones and Taylor, 1965; Laak, 1970, 1986; Loudon et al., 1998; Otis, 1985; Siegrist and Boyle, 1987; Tyler and Converse, 1994).

The comparatively higher hydraulic loadings that highly treated wastewater (highly treated in terms of TSS, ammonium-nitrogen, and BOD) may permit should be considered carefully because the resulting rapid flow through the soil may allow deep penetration of pathogens (Converse and Tyler, 1998a, 1998b; Siegrist et al., 2000; Siegrist and Van Cuyk, 2001b; Tyler and Converse, 1994). The trench length perpendicular to ground water

movement (footprint) should remain the same to minimize system impacts on the aquifer.

Unfortunately, well-tested organic loading rates for various classes of soils and SWIS design configurations have not been developed. Most organic loading rates have been derived directly from the hydraulic loadings typically used in SWIS design by assuming a BOD₅ concentration (see box and table 4-3). The derived organic loading rates also incorporate the implicit factor of safety found in the hydraulic loading rates. Organic loadings do appear to have less impact on slowly permeable soils because the resistance of the biomat that forms at the infiltrative surface presents less resistance to infiltration of the wastewater than the soil itself (Bouma, 1975). For a further discussion of SWIS performance under various environmental conditions, see Siegrist and Van Cuyk, 2001b.

Constituent mass loadings

Constituent mass loadings may be a concern with respect to water quality. For example, to use the soil's capacity to adsorb and retain phosphorus when systems are located near sensitive surface waters, a phosphorus loading rate based on the soil adsorption capacity might be selected as the controlling rate of wastewater application to the infiltration surface to maximize phosphorus removal. Placement of the effluent distribution piping high in the soil profile can promote greater phosphorus removal because the permeability of medium- and fine-textured soils tends to decrease with depth and because the translocation of aluminum and iron—which react with phosphorus to form insoluble compounds retained in the soil matrix—occurs in some sandy soils, with the maximum accumulation usually above 45 cm (Mokma et al., 2001). Many lakes are surrounded by sandy soils with a low phosphorus adsorption capacity. If effluent distribution systems are installed below 45 cm in these sandy soils, less phosphorus will be removed from the percolating effluent. In the case of a soluble constituent of concern such as nitrate-nitrogen, a designer might decide to reduce the mass of nitrate per unit of application area. This would have the effect of increasing the size of the SWIS footprint, thereby reducing the potential concentration of nitrate in the ground water immediately surrounding the SWIS (Otis, 2001).

Factors of safety in infiltration surface sizing

Sizing of onsite wastewater systems for single-family homes is typically based on the estimated peak daily flow and the “long term acceptance rate” of the soil for septic tank effluent. In most states, the design flow is based on the number of bedrooms in the house. A daily flow of 150 gallons is commonly assumed for each bedroom. This daily flow per bedroom assumes two people per bedroom that generate 75 gpd each. Bedrooms, rather than current occupancy, are used for the basis of SWIS design because the number of occupants in the house can change.

Using this typical estimating procedure, a three-bedroom home would have a design flow of 150 gpd/bedroom x 3 bedrooms or 450 gpd. However, the actual daily average flow could be much less. Based on the 1990 census, the average home is occupied by 2.8 persons. Each person in the United States generates 45 to 70 gpd of domestic wastewater. Assuming these averages, the average daily flow would be 125 to 195 gpd or 28 to 44 percent of the design flow, respectively. Therefore, the design flow includes an implicit factor of safety of 2.3 to 3.6. Of course, this factor of safety varies inversely with the home occupancy and water use.

Unfortunately, the factors of safety implicitly built into the flow estimates are seldom recognized. This is particularly true in the case of the design hydraulic loading rates, which were derived from existing SWISs. In most codes, the hydraulic loading rates for sand are about 1.0 to 1.25 gpd/ft². Because these hydraulic loading rates assume daily flows of 150 gpd per bedroom, they are overestimated by a factor of 2.3 to 3.6. Fortunately, these two assumptions largely cancel each other out in residential applications, but the suggested hydraulic loading rates often are used to size commercial systems and systems for schools and similar facilities, where the ratios between design flows and actual daily flows are closer to 1.0. This situation, combined with a lack of useful information on allowable organic loading rates, has resulted in failures, particularly for larger systems where actual flow approximates design.

4.4.6 Geometry, orientation, and configuration of the infiltration surface

The geometry, orientation, and configuration of the infiltration surface are critical design factors that affect the performance of SWISs. They are important for promoting subsoil aeration, maintaining an acceptable separation distance from a saturated zone or restrictive horizon, and facilitating construction. Table 4-4 lists the design considerations discussed in this section.

Geometry

The width and length of the infiltration surface are important design considerations to improve performance and limit impacts on the receiving environment. Trenches, beds, and seepage pits (or dry wells) are traditionally used geometries. Seepage pits can be effective for wastewater dispersal, but they provide little treatment because they extend deep into the soil profile, where oxygen transfer and treatment are limited and the separation distance to ground water is reduced. They are not recommended for onsite wastewater treatment and are not included as an option in this manual.

Width

Infiltration surface clogging and the resulting loss of infiltrative capacity are less where the infiltration surface is narrow. This appears to occur because reaeration of the soil below a narrow infiltration surface is more rapid. The dominant pathway for oxygen transport to the subsoil appears to be diffusion through the soil surrounding the infiltration surface (figure 4-7). The unsaturated zone below a wide surface quickly becomes anaerobic because the rates of oxygen diffusion are too low to meet the oxygen demands of biota and organics on the infiltration surface. (Otis, 1985; Siegrist et al., 1986). Therefore, trenches perform better than beds. Typical trench widths range from 1 to 4 feet. Narrower trenches are preferred, but soil conditions and construction techniques might limit how narrow a trench can be constructed. On sloping sites, narrow trenches are a necessity because in keeping the infiltration surface level, the uphill side of the trench bottom might be excavated into a less suitable soil horizon. Wider trench infiltration surfaces have been successful in at-grade systems and mounds probably because the engineered fill material and elevation above the natural grade promote better reaeration of the fill.

Comparing hydraulic and organic mass loadings for a restaurant wastewater

Infiltration surface sizing traditionally has been based on the daily hydraulic load determined through experience to be acceptable for the soil characteristics. This approach to sizing fails to account for changes in applied wastewater strength. Since soil clogging has been shown to be dependent on applied wastewater strength, it might be more appropriate to size infiltration surfaces based on organic mass loadings.

To illustrate the impact of the different sizing methods, sizing computations for a restaurant are compared. A septic tank is used for pretreatment prior to application to the SWIS. The SWIS is to be constructed in a sandy loam with a moderate, subangular blocky structure. The suggested hydraulic loading rate for domestic septic tank effluent on this soil is 0.6 gpd/ft² (table 4-3). The restaurant septic tank effluent has the following characteristics:

BOD ₅	800 mg/L
TSS	200 mg/L
Average daily flow	600 gpd

Infiltration area based on hydraulic loading:

$$\text{Area} = 600 \text{ gpd} / 0.6 \text{ gpd/ft}^2 = 1,000 \text{ ft}^2$$

Infiltration area based on organic loading:

At the design infiltration rate of 0.6 gpd/ft² recommended for domestic septic tank effluent, the equivalent organic loading is (assuming a septic tank BOD₅ effluent concentration of 150 mg/L)

$$\begin{aligned} \text{Organic Loading} &= 150 \text{ mg/L} \times 0.6 \text{ gpd/ft}^2 \times (8.34 \text{ lb/mg/L} \times 10^{-6} \text{ gal}) \\ &= 7.5 \times 10^{-4} \text{ lb BOD}_5/\text{ft}^2\text{-d} \end{aligned}$$

Assuming 7.5 x 10⁻⁴ lb BOD₅/ft²-d as the design organic loading rate,

$$\begin{aligned} \text{Area} &= \frac{(800 \text{ mg-BOD}_5/\text{L} \times 600 \text{ gpd} \times 8.34 \text{ lbs/mg/L} \times 10^{-6} \text{ gal})}{(7.5 \times 10^{-4} \text{ lb BOD}_5/\text{ft}^2\text{-d})} \\ &= \frac{4.0 \text{ lb BOD}_5/\text{d}}{(7.5 \times 10^{-4} \text{ lb BOD}_5/\text{ft}^2\text{-d})} = 5337 \text{ ft}^2 \text{ (a 540\% increase)} \end{aligned}$$

Impact of a 40% water use reduction on infiltration area sizing

Based on hydraulic loading,

$$\text{Area} = \frac{(1 - 0.4) \times 600 \text{ gpd}}{0.6 \text{ gpd/ft}^2} = 600 \text{ ft}^2$$

Based on organic loading (note the concentration of BOD₅ increases with water conservation but the mass of BOD₅ discharged does not change),

$$\begin{aligned} \text{Area} &= \frac{(800 \text{ mg-BOD}_5/\text{L} \times 600 \text{ gpd}) \times (8.34 \text{ lb/mg/L} \times 10^{-6} \text{ gal})}{[(1 - 0.4) \times 600 \text{ gpd}] \times (7.5 \times 10^{-4} \text{ lb BOD}_5/\text{ft}^2\text{-d})} \\ &= \frac{4.0 \text{ lb BOD}_5/\text{d}}{(7.5 \times 10^{-4} \text{ lb BOD}_5/\text{ft}^2\text{-d})} = 5337 \text{ ft}^2 \text{ (an 890\% increase)} \end{aligned}$$

However, infiltration bed surface widths of greater than 10 feet are not recommended because oxygen transfer and clogging problems can occur (Converse and Tyler, 2000; Converse et al., 1990).

Length

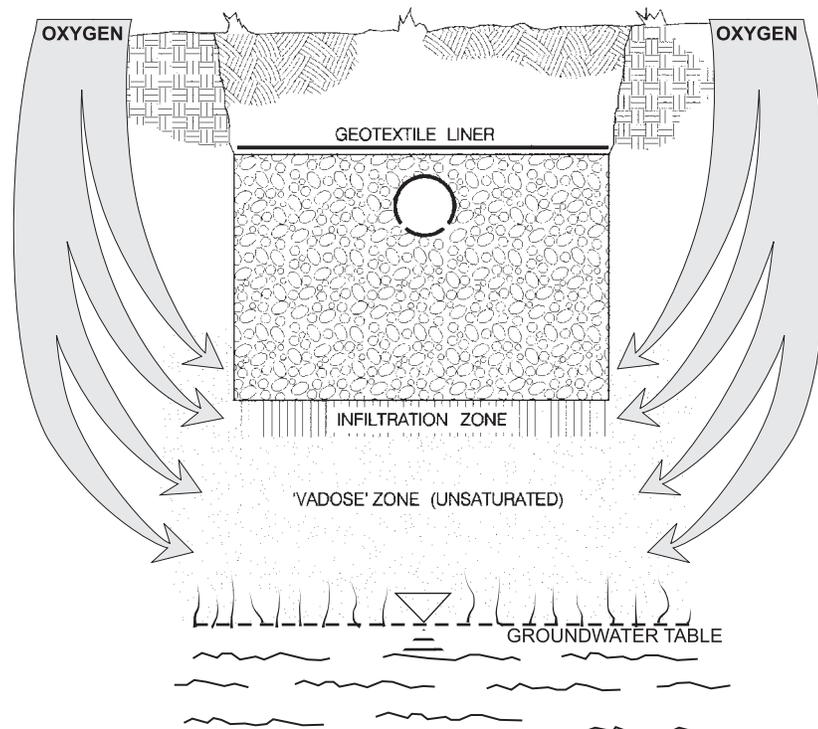
The trench length is important where downslope linear loadings are critical, ground water quality impacts are a concern, or the potential for ground

water mounding exists. In many jurisdictions, trench lengths have been limited to 100 feet. This restriction appeared in early codes written for gravity distribution systems and exists as an artifact with little or no practical basis when pressure distribution is used. Trench lengths longer than 100 feet might be necessary to minimize ground water impacts and to permit proper wastewater drainage from the site. Long trenches can be used to reduce the linear loadings on a site by spreading the

Table 4-4. Geometry, orientation, and configuration considerations for SWISs

Design type	Design considerations
Trench	
<i>Geometry</i>	
Width	Preferably less than 3 ft. Design width is affected by distribution method, constructability, and available area.
Length	Restricted by available length parallel to site contour, distribution method, and distribution network design.
Sidewall height	Sidewalls are not considered an active infiltration surface. Minimum height is that needed to encase the distribution piping or to meet peak flow storage requirements.
<i>Orientation/ configuration</i>	Should be constructed parallel to site contours and/or water table or restrictive layer contours. Should not exceed the site's maximum linear hydraulic loading rate per unit of length. Spacing of multiple, parallel trenches is also limited by the construction method and slow dispersion from the trenches.
Bed	
<i>Geometry</i>	
Width	Should be as narrow as possible. Beds wider than 10 to 15 feet should be avoided.
Length	Restricted by available length parallel to site contour, distribution method, and distribution network design.
Sidewall height	Sidewalls are not considered an active infiltration surface. Minimum height is that needed to encase the distribution piping or to meet peak flow storage requirements.
<i>Orientation/ configuration</i>	Should be constructed parallel to site contours and/or water table or restrictive layer contours. The loading over the total projected width should not exceed the estimated downslope maximum linear hydraulic loading.
Seepage pit	Not recommended because of limited treatment capability.

Figure 4-7. Pathway of subsoil reaeration



Source: Ayres Associates, 2000

wastewater loading parallel to and farther along the surface contour. With current distribution/dosing technology, materials, and construction methods, trench lengths need be limited only by what is practical or feasible on a given site. Also, use of standard trench lengths, e.g., X feet of trench/BR, is discouraged because it restricts the design options to optimize performance for a given site condition.

Height

The height of the sidewall is determined primarily by the type of porous medium used in the system, the depth of the medium needed to encase the distribution piping, and/or storage requirements for peak flows. Because the sidewall is not included as an active infiltration surface in sizing the infiltration area, the height of the sidewall can be minimized to keep the infiltration surface high in the soil profile. A height of 6 inches is usually sufficient for most porous aggregate applications. Use of a gravelless system requires a separate analysis to determine the height based on whether it is an aggregate-free (empty chamber) design or one that substitutes a lightweight aggregate for washed gravel or crushed stone.

Orientation

Orientation of the infiltration surface(s) becomes an important consideration on sloping sites, sites with shallow soils over a restrictive horizon or saturated zone, and small or irregularly shaped lots. The long axes of trenches should be aligned parallel to the ground surface contours to reduce linear contour hydraulic loadings and ground water mounding potential. In some cases, ground water or restrictive horizon contours may differ from surface contours because of surface grading or the soil's morphological history. Where this occurs, consideration should be given to aligning the trenches with the contours of the limiting condition rather than those of the surface. Extending the trenches perpendicular to the ground water gradient reduces the mass loadings per unit area by creating a "line" source rather than a "point" source along the contour. However, the designer must recognize that the depth of the trenches and the soil horizon in which the infiltration surface is placed will vary across the system. Any adverse impacts this might have on system performance should be mitigated through design adjustments.

Configuration

The spacing of multiple trenches constructed parallel to one another is determined by the soil characteristics and the method of construction. The sidewall-to-sidewall spacing must be sufficient to enable construction without damage to the adjacent trenches. Only in very tight soils will normally used spacings be inadequate because of high soil wetness and capillary fringe effects, which can limit oxygen transfer. It is important to note that the sum of the hydraulic loadings to one or more trenches or beds per each unit of contour length (when projected downslope) must not exceed the estimated maximum contour loading for the site. Also, the finer (tighter) the soil, the greater the trench spacing should be to provide sufficient oxygen transfer. Quantitative data are lacking, but Camp (1985) reported a lateral impact of more than 2.0 meters in a clay soil.

Given the advantages of lightweight gravelless systems in terms of potentially reduced damage to the site's hydraulic capacity, parallel trenches may physically be placed closer together, but the downslope hydraulic capacity of the site and the natural oxygen diffusion capacity of the soil cannot be exceeded.

4.4.7 Wastewater distribution onto the infiltration surface

The method and pattern of wastewater distribution in a subsurface infiltration system are important design elements. Uniform distribution aids in maintaining unsaturated flow below the infiltration surface, which results in wastewater retention times in the soil that are sufficiently long to effect treatment and promote subsoil reaeration. Uniform distribution design also results in more complete utilization of the infiltration surface.

Gravity flow and dosing are the two most commonly used distribution methods. For each method, various network designs are used (table 4-5). Gravity flow is the most commonly used method because it is simple and inexpensive. This method discharges effluent from the septic tank or other pretreatment tank directly to the infiltration surface as incoming wastewater displaces it from the tank(s). It is characterized by the term "trickle flow" because the effluent is slowly discharged over much of the day. Typically, tank discharges

are too low to flow throughout the distribution network. Thus, distribution is unequal and localized overloading of the infiltration surface occurs with concomitant poor treatment and soil clogging (Bouma, 1975; McGauhey and Winneberger, 1964; Otis, 1985; Robeck et al., 1964).

Dosing, on the other hand, accumulates the wastewater effluent in a dose tank from which the water is periodically discharged under pressure in “doses” to the infiltration system by a pump or siphon. The pretreated wastewater is allowed to accumulate in the dose tank and is discharged when a predetermined water level, water volume, or elapsed time is reached. The dose volumes and discharge rates are usually such that much of the distribution network is filled, resulting in more uniform distribution over the infiltration surface. Dosing outperforms gravity-flow systems because distribution is more uniform. In addition, the periods between doses provide opportunities for the subsoil to drain and reerate before the next dose (Bouma et al., 1974; Hargett et al., 1982; Otis et al., 1977). However, which method is most appropriate depends on the specific application.

Gravity flow

Gravity flow can be used where there is a sufficient elevation difference between the outlet of the pretreatment tank and the SWIS to allow flow to and through the SWIS by gravity. Gravity flow systems are simple and inexpensive to construct but

are the least efficient method of distribution. Distribution is very uneven over the infiltration surface, resulting in localized overloading (Converse, 1974; McGauhey and Winneberger, 1964; Otis et al., 1978; University of Wisconsin, 1978). Until a biomat forms on the infiltration surface to slow the rate of infiltration, the wastewater residence time in the soil might be too short to effect good treatment. As the biomat continues to form on the overloaded areas, the soil surface becomes clogged, forcing wastewater effluent to flow through the porous medium of the trench until it reaches an unclogged infiltration surface. This phenomenon, known as “progressive clogging,” occurs until the entire infiltration surface is ponded and the sidewalls become the more active infiltration surfaces. Without extended periods of little or no flow to allow the surface to dry, hydraulic failure becomes imminent. Although inefficient, these systems can work well for seasonal homes with intermittent use or for households with low occupancies. Seasonal use of SWISs allows the infiltration surface to dry and the biomat to oxidize, which rejuvenates the infiltration capacity. Low occupancies result in mass loadings of wastewater constituents that are lower and less likely to exceed the soil’s capacity to completely treat the effluent.

Perforated pipe

Four-inch-diameter perforated plastic pipe is the most commonly used distribution piping for

Table 4-5. Distribution methods and applications.

Method	Typical applications
Gravity flow	
4-inch perforated pipe	Single or looped trenches at the same elevation; beds.
Distribution box	Multiple independent trenches on flat or sloping sites.
Serial relief line	Multiple serially connected trenches on a sloping site.
Drop box	Multiple independent trenches on a sloping site.
Dosed distribution	
4-inch perforated pipe (with or without a distribution box)	Single (or multiple) trenches, looped trenches at the same elevation, and beds.
Pressure manifold	Multiple independent trenches on sloping sites.
Rigid pipe pressure network	Multiple independent trenches at the same elevation (a preferred method for larger SWISs)
Dripline pressure network	Multiple independent trenches on flat or sloping sites (a preferred method for larger SWISs)

gravity flow systems. The piping is generally smooth-walled rigid polyvinyl chloride (PVC), or flexible corrugated polyethylene (PE) or acrylonitrile-butadiene-styrene (ABS). One or two rows of holes or slots spaced 12 inches apart are cut into the pipe wall. Typically, the piping is laid level in gravel (figure 4-1) with the holes or slots at the bottom (ASTM, undated). One distribution line is used per trench. In bed systems, multiple lines are installed 3 to 6 feet apart.

Distribution box

Distribution boxes are used to divide the wastewater effluent flow among multiple distribution lines. They are shallow, flat bottomed, watertight structures with a single inlet and individual outlets provided at the same elevation for each distribution line. An above-grade cover allows access to the inside of the box. The “d-box” must be laid level on a sound, frost-proof footing to divide the flow evenly among the outlets. Uneven settlement or frost heaving results in unequal flow to the lateral lines because the outlet hole elevations cease to be level. If this occurs, adjustments must be made to reestablish equal division of flow. Several devices can be used. Adjustable weirs that can level the outlet inverts and maintain the same length of weir per outlet are one option. Other options include designs that allow for leveling of the entire box (figure 4-8). The box can also be used to take individual trenches out of service by blocking the outlet to the distribution lateral or raising the outlet weir above the weir elevations for the other outlets. Because of the inevitable movement of d-boxes, their use has been discouraged for many years (USPHS, 1957). However, under a managed care system with regular adjustment, the d-box is acceptable.

Serial relief line

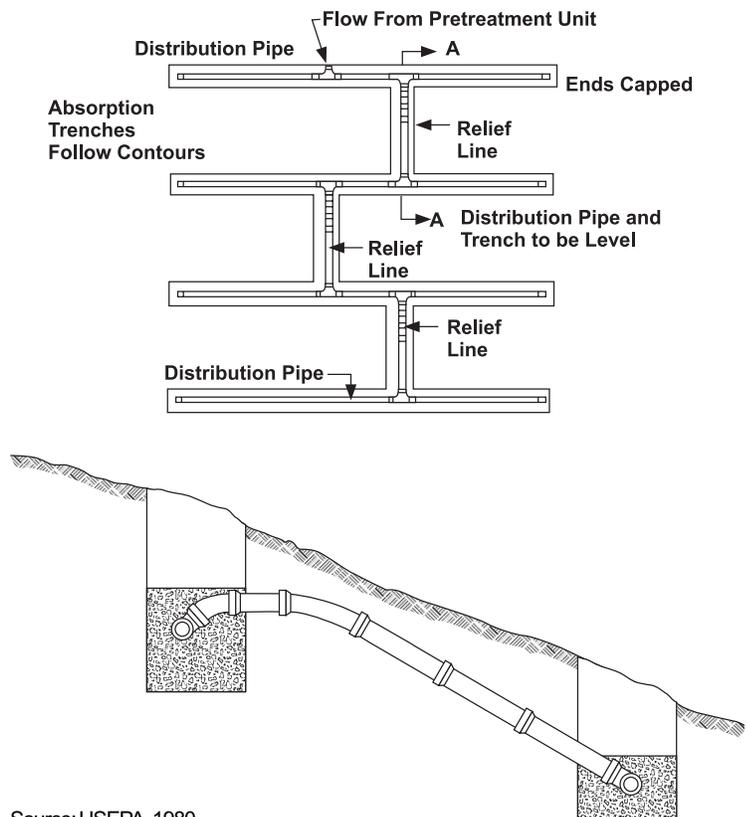
Serial relief lines distribute wastewater to a series of trenches constructed on a sloping site. Rather than dividing the flow equally among all trenches as with a distribution box, the uppermost trench is loaded until completely flooded before the next (lower) trench receives effluent. Similarly, that trench is loaded until flooded before discharge occurs to the next trench, and so on. This method of loading is accomplished by installing “relief lines” between successive trenches (figure 4-9).

Figure 4-8. Distribution box with adjustable weir outlets



Source: Ayres Associates.

Figure 4-9. Serial relief line distribution network and installation detail



Source: USEPA, 1980.

The relief lines are simple overflow lines that connect one trench to the adjacent lower trench. They are solid-wall pipes that connect the crown of the upper trench distribution pipe with the distribution pipe in the lower trench. Successive relief lines are separated by 5 to 10 feet to avoid short-circuiting. This method of distribution makes full hydraulic use of all bottom and sidewall infiltration surfaces, creates the maximum hydrostatic head over the infiltration surfaces to force the water into the surrounding soil, and eliminates the problem of dividing flows evenly among independent trenches. However, because continuous ponding of the infiltration surfaces is necessary for the system to function, the trenches suffer hydraulic failure more rapidly and progressively because the infiltration surfaces cannot regenerate their infiltrative capacity.

Drop box

Drop box distribution systems function similarly to relief line systems except that drop boxes are used in place of the relief lines. Drop boxes are installed for each trench. They are connected in manifolds to trenches above and below (figure 4-10). The outlet invert can be placed near the top of each trench to force the trench to fill completely before it discharges to the next trench if a serial distribution mode of operation is desired. Solid-wall pipe is used between the boxes.

The advantage of this method over serial relief lines is that individual trenches can be taken out of service by attaching 90 degree ells to the outlets that rise above the invert of the manifold connection to the next trench drop box. It is easier to add additional trenches to a drop box system than to a serial relief line network. Also, the drop box system may be operated as an alternating trench system by using the 90 degree ells on unused lines. With this and the serial distribution system, the designer must carefully evaluate the downslope capacity of the site to ensure that it will not be overloaded when the entire system or specific trench combinations are functioning.

Gravelless wastewater dispersal systems

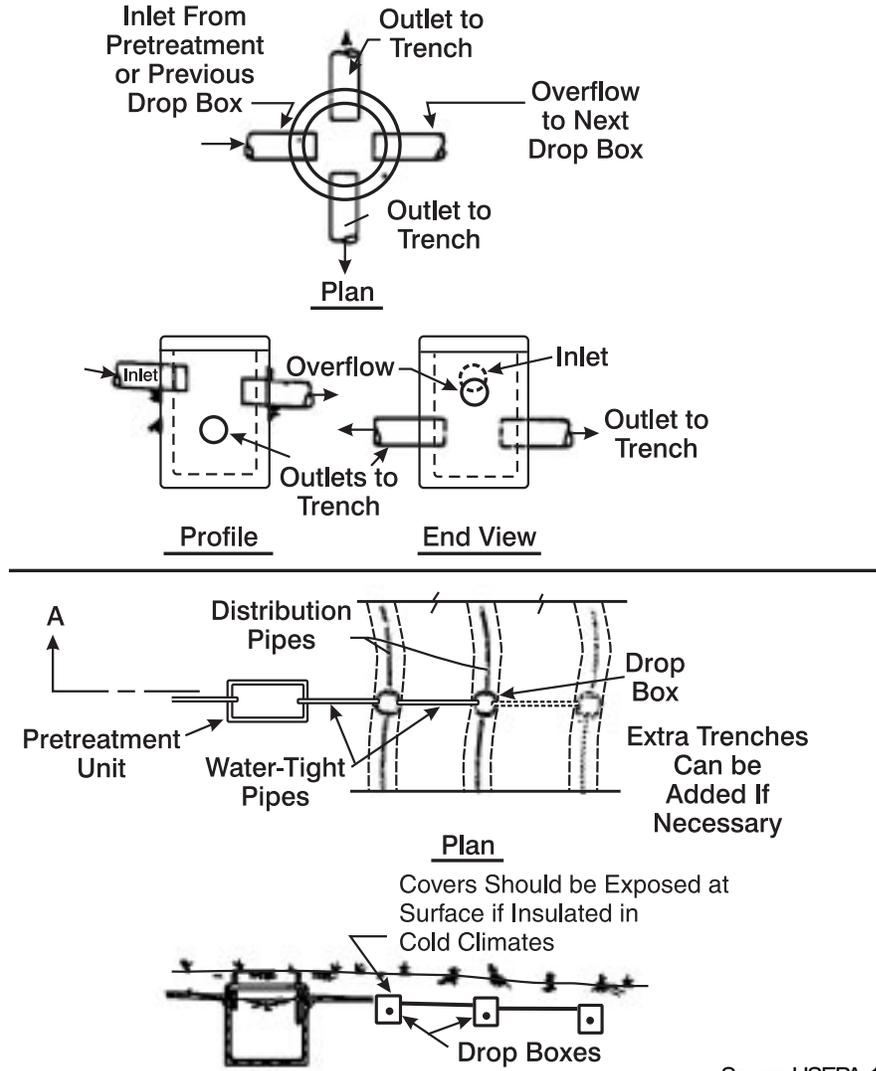
Gravelless systems have been widely used. They take many forms, including open-bottomed chambers, fabric-wrapped pipe, and synthetic materials such as expanded polystyrene foam chips (fig-

ure 4-11). Some gravelless drain field systems use large-diameter corrugated plastic tubing covered with permeable nylon filter fabric not surrounded by gravel or rock. The area of fabric in contact with the soil provides the surface for the septic tank effluent to infiltrate the soil. The pipe is a minimum of 10 to 12 inches (25.4 to 30.5 centimeters) in diameter covered with spun bonded nylon filter fabric to distribute water around the pipe. The pipe is placed in a 12- to 24-inch (30.5- to 61-centimeter)-wide trench. These systems can be installed in areas with steep slopes with small equipment and in hand-dug trenches where conventional gravel systems would not be possible.

Reduced sizing of the infiltration surface is often promoted as another advantage of the gravelless system. This is based primarily on the premise that gravelless systems do not “mask” the infiltration surface as gravel does where the gravel is in direct contact with the soil. Proponents of this theory claim that an infiltration surface area reduction of 50 percent is warranted. However, these reductions are not based on scientific evidence though they have been codified in some jurisdictions (Amerson et al., 1991; Anderson et al., 1985; Carlile and Osborne, 1982; Effert and Cashell, 1987). Although gravel masking might occur in porous medium applications, reducing the infiltration surface area for gravelless systems increases the BOD mass loading to the available infiltration surface. Many soils might not be able to support the higher organic loading and, as a result, more severe soil clogging and greater penetration of pollutants into the vadose zone and ground water can occur (University of Wisconsin, 1978), negating the benefits of the gravelless surface.

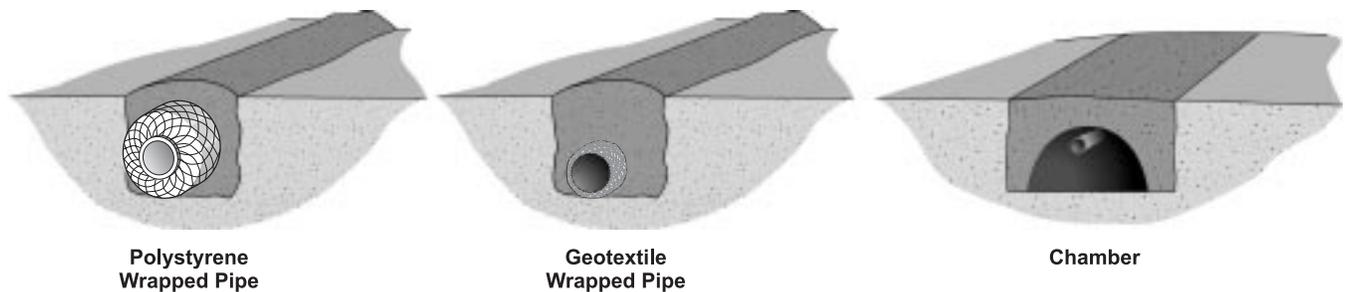
A similar approach must be taken with any contaminant in the pretreatment system effluent that must be removed before it reaches ground water or nearby surface waters. A 50 percent reduction in infiltrative surface area will likely result in less removal of BOD, pathogens, and other contaminants in the vadose zone and increase the presence and concentrations of contaminants in effluent plumes. The relatively confined travel path of a plume provides fewer adsorption sites for removal of adsorbable contaminants (e.g., metals, phosphorus, toxic organics). Because any potential reductions in infiltrative surface area must be analyzed in a similar comprehensive fashion, the use of

Figure 4-10. Drop box distribution network



Source: USEPA, 1980

Figure 4-11. Various gravelless systems



Source: National Small Flows Clearinghouse.

gravelless medium should be treated similarly to potential reductions from increased pretreatment and better distribution and dosing concepts.

Despite the cautions stated above, the overall inherent value of lightweight gravelless systems should not be ignored, especially in areas where gravel is expensive and at sites that have soils that are susceptible to smearing or other structural damage during construction due to the impacts of heavy machinery on the site. In all applications where gravel is used (see *SWIS Media* in the following section), it must be properly graded and washed. Improperly washed gravel can contribute fines and other material that can plug voids in the infiltrative surface and reduce hydraulic capability. Gravel that is embedded into clay or fine soils during placement can have the same effect.

Leaching chambers

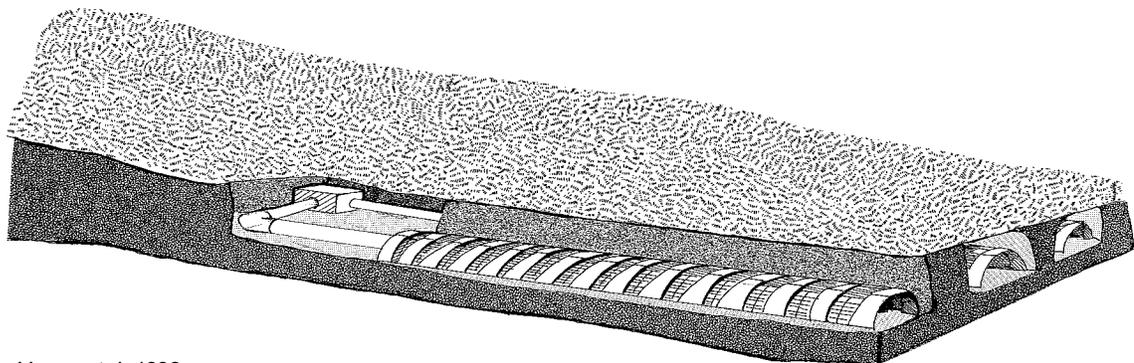
A leaching chamber is a wastewater treatment system that consists of trenches or beds and one or more distribution pipes or open-bottomed plastic chambers. Leaching chambers have two key functions: to disperse the effluent from septic tanks and to distribute this effluent throughout the trenches. A typical leaching chamber consists of several high-density polyethylene injection-molded arch-shaped chamber segments. A typical chamber has an average inside width of 15 to 40 inches (38 to 102 centimeters) and an overall length of 6 to 8 feet (1.8 to 2.4 meters). The chamber segments are usually 1-foot high, with wide slotted sidewalls. Depending on the drain field size requirements, one or more chambers are typically connected to form an underground drain field network.

Typical leaching chambers (figure 4-12) are gravelless systems that have drain field chambers with no bottoms and plastic chamber sidewalls, available in a variety of shapes and sizes. Use of these systems sometimes decreases overall drain field costs and may reduce the number of trees that must be removed from the drain field lot.

About 750,000 chamber systems have been installed over the past 15 years. Currently, a high percentage of new construction applications use lightweight plastic leaching chambers for new wastewater treatment systems in states like Colorado, Idaho, North Carolina, Georgia, Florida, and Oregon. The gravel aggregate traditionally used in drain fields can have large quantities of mineral fines that also clog or block soil pores. Use of leaching chambers avoids this problem. Recent research sponsored by manufacturers shows promising results to support reduction in sizing of drain fields through the use of leaching chambers without increased hydraulic and pollutant penetration failures (Colorado School of Mines, 2001; Siegrist and Vancuyk, 2001a, 2001b). These studies should be continued to eventually yield rational guidelines for proper sizing of these systems based on the type of pretreatment effluent to be received (septic tank effluent, effluent from filters or aerobic treatment units, etc.), as well as different soil types and hydrogeological conditions. Many states offer drain field sizing reduction allowances when leaching chambers are used instead of conventional gravel drain fields.

Because leaching chamber systems can be installed without heavy equipment, they are easy to install

Figure 4-12. Placement of leaching chambers in typical application



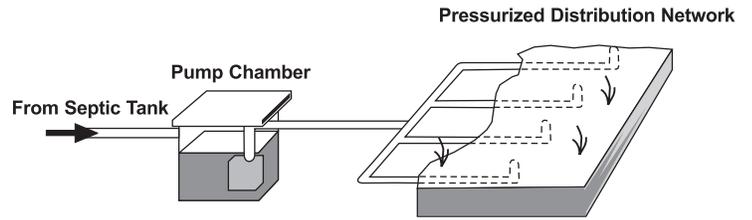
Source: Hoover et al., 1996.

and repair. These high-capacity, open-bottom drain field systems can provide greater storage than conventional gravel systems and can be used in areas appropriate for gravel aggregate drain fields. Leaching systems can operate independently and require little day-to-day maintenance. Their maintenance requirements are comparable to those of aggregate trench systems.

The lightweight chamber segments available on the market stack together compactly for efficient transport. Some chambers interlock with ribs without fasteners, cutting installation time by more than 50 percent reused and conventional gravel/pipe systems. Such systems can be reused and relocated if the site owner decides to build on another drain field site. A key disadvantage of leaching chambers compared to gravel drain fields is that they can be more expensive if a low-cost source of gravel is readily available.

Porous media should be placed along the chamber sidewall area to a minimum compacted height of 8 inches above the trench bottom. Additional backfill is placed to a minimum compacted height of 6 to 12 inches above the chamber, depending on the chamber strength. Individual chamber trench bottoms should be leveled in all directions and follow the contour of the ground surface elevation without any dams or other water stops. The manufacturer’s installation instructions should be followed, and systems should be installed by an authorized contractor.

Figure 4-13. Typical pressurized distribution system layout



Source: National Small Flows Clearinghouse

Dosed flow distribution

Dosed-flow distribution systems are a significant improvement over gravity-flow distribution systems. The design of dosed-flow systems (figure 4-13) includes both the distribution network and the dosing equipment (see table 4-6). Dosing achieves better distribution of the wastewater effluent over the infiltration surface than gravity flow systems and provides intervals between doses when no wastewater is applied. As a result, dosed-flow systems reduce the rate of soil clogging, more effectively maintain unsaturated conditions in the subsoil (to effect good treatment through extended residence times and increased reaeration potential), and provide a means to manage wastewater effluent applications to the infiltration system (Hargett et al., 1982). They can be used in any application and should be the method of choice. Unfortunately, they are commonly perceived to be less desirable because they add a mechanical

Table 4-6. Dosing methods and devices.

Dosing method	Typical application
On-Demand	Dosing occurs when a sufficient volume of wastewater has accumulated in the dose tank to activate the pump switch or siphon. Dosing continues until the preselected low water level is reached. Typically, there is no control on the daily volume of wastewater dosed.
Timed	Dosing is performed by pumps on a timed cycle, typically at equal intervals and for preset dose volumes so that the daily volume of wastewater dosed does not exceed the system’s design flow. Controls can be set so that only full doses occur. Peak flows are stored in the dose tank for dosing during low flow periods. Excessive flows are retained in the tank, and, if they persist, a high water alarm alerts the owner of the need for remedial action. This approach prevents unwanted and detrimental discharges to the SWIS.
Dosing device	
Pump	Pressure distribution networks are set at elevations that are typically higher than the dose tank. Multiple infiltration areas can be dosed from the same tank using multiple, alternating pumps or automatic valves.
Siphon	On-demand dosing of gravity or pressure distribution networks is used where the elevation between the siphon invert and the distribution pipe orifices is sufficient for the siphon to operate. Siphons cannot be used for timed dosing. Two siphons in the same dose tank can be used to alternate automatically between two infiltration areas.

component to an otherwise “passive” system and add cost because of the dosing equipment. The improved performance of dosed-flow systems over gravity flow systems should outweigh these perceived disadvantages, especially when a management entity is in place. It must be noted, however, that if dosed infiltration systems are allowed to pond, the advantages of dosing are lost because the bottom infiltration surface is continuously inundated and no longer allowed to rest and reaerate. Therefore, there is no value in using dosed-flow distribution in SWISs designed to operate ponded, such as systems that include sidewall area as an active infiltration surface or those using serial relief lines.

Perforated pipe

Four-inch perforated pipe networks (with or without d-boxes or pressure manifolds) that receive dosed-flow applications are designed no differently than gravity-flow systems. Many of the advantages of dosing are lost in such networks, however, because the distribution is only slightly better than that of gravity-flow systems (Converse, 1974).

Pressure manifold

A pressure manifold consists of a large-diameter pipe tapped with small outlet pipes that discharge to gravity laterals (figure 4-14). A pump pressurizes the manifold, which has a selected diameter to ensure that pressure inside the manifold is the same at each outlet. This method of flow division is more accurate and consistent than a distribution box, but it has the same shortcoming since flow after the manifold is by gravity along each distribu-

tion lateral. Its most common application is to divide flow among multiple trenches constructed at different elevations on a sloping site.

Table 4-7 can be used to size a pressure manifold for different applications (see sidebar). This table was developed by Berkowitz (1985) to size the manifold diameter based on the spacing between pressure lateral taps, the lateral tap diameter, and the number of lateral taps. The hydraulic computations made to develop the table set a maximum flow differential between laterals of 5 percent. The dosing rate is determined by calculating the flow in a single lateral tap assuming 1 to 4 feet of head at the manifold outlets and multiplying the result by the number of lateral taps. The Hazen-Williams equation for pipe flow can be used to make this calculation.

Pressure distribution is typically constructed of Schedule 40 PVC pipe (figure 4-15). The lateral taps are joined by tees. They also can be attached by tapping (threading) the manifold pipe, but the manifold pipe must be Schedule 80 to provide a thicker pipe wall for successful tapping. Valves on each pressure tap are recommended to enable each line to be taken out of service as needed by closing the appropriate valve. This allows an opportunity to manage, rest, or repair individual lines. To prevent freezing, the manifold can be drained back to the dose tank after each dose. If this is done, the volume of water that will drain from the manifold and forcemain must be added to the dose volume to achieve the desired dose.

Rigid pipe pressure network

Rigid pipe pressure distribution networks are used to provide relatively uniform distribution of

Figure 4-14. Pressure manifold detail

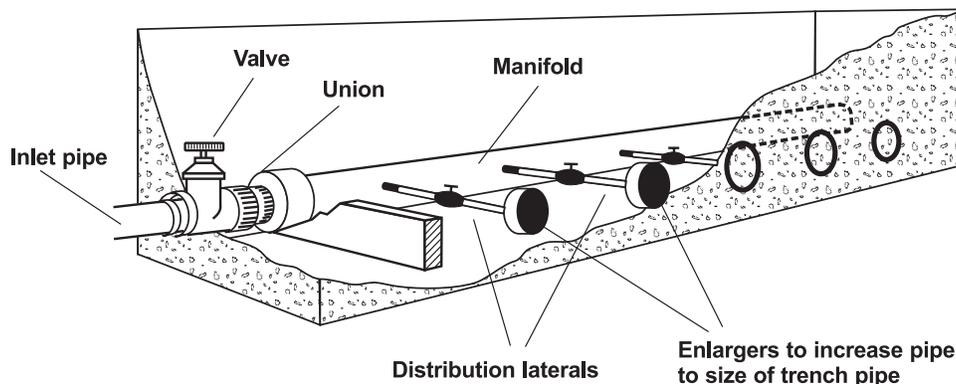
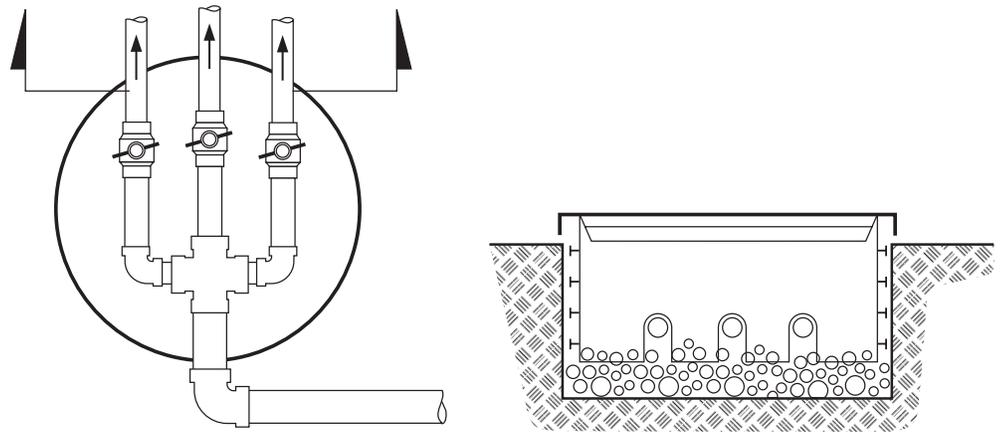


Table 4-7. Pressure manifold sizing

Tap spacing (feet)	Manifold size (inches)	Single-sided manifold						Double-sided manifold					
		Lateral tap diameter (inches)						Lateral tap diameter (inches)					
		0.50	0.75	1.00	1.25	1.50	2.00	0.50	0.75	1.00	1.25	1.50	2.00
		Maximum number of lateral taps						Maximum number of lateral taps					
0.5	2	4	2					2					
	3	9	5	3	2			4	2				
	4	16	9	5	3	2		7	4	2			
	6	>40	21	12	7	5	3	18	10	6	3	2	
	8		38	22	12	9	5		17	10	6	4	2
3.0	2	8	2					2					
	3	14	12	3	2			6	2				
	4	21	18	6	3	2		16	5	3			
	6	38	30	26	8	5	3	>20	19	7	3	2	
	8												
6.0	2	5	4					4					
	3	9	7	6	2			7	3	2			
	4	14	11	9	4	2		10	9	3			
	6	27	20	17	14	7	3	19	15	13	4	3	
	8												

Source: Adapted from Berkowitz, 1985.

Figure 4-15. Horizontal design for pressure distribution



Source: Washington Department of Health, 1998.

wastewater effluent over the entire infiltration surface simultaneously during each dose. They are well suited for all dosed systems. Because they deliver the same volume of wastewater effluent per linear length of lateral, they can be used to dose multiple trenches of unequal length. Although rigid pipe pressure networks can be designed to deliver equal volumes to trenches at different elevations (Mote, 1984; Mote et al., 1981; Otis, 1982), these situations should be avoided. Uniform distribution is achieved only when the network is fully pressurized. During filling and draining of the network,

the distribution lateral at the lowest elevation receives more water. This disparity increases with increasing dosing frequency. As an alternative on sloping sites, the SWIS could be divided into multiple cells, with the laterals in each cell at the same elevation. If this is not possible, other distribution designs should be considered.

The networks consist of solid PVC pipe manifolds that supply water to a series of smaller perforated PVC laterals (figure 4-16). The laterals are designed to discharge nearly equal volumes of

Pressure manifold design

A SWIS consisting of 12 trenches of equal length is to be constructed on a slope. To divide the septic tank effluent equally among the 12 trenches, a pressure manifold is to be used. The lateral taps are to be spaced 6 inches apart on one side of the manifold.

Table 4-7 can be used to size the manifold. Looking down the series of columns under the Single-sided manifold, up to sixteen ½-inch taps could be made to a 4-inch manifold. Therefore, a 4-inch manifold would be acceptable. If ¾- or 1-inch taps were used, a 6-inch manifold would be necessary.

Using the orifice equation, the flow from each lateral tap can be estimated by assuming an operating pressure in the manifold:

$$Q = Ca(2gh)^2$$

where Q is the lateral discharge rate, C is a dimensionless coefficient that varies with the characteristics of the orifice (0.6 for a sharp-edged orifice), a is the area of the orifice, g is the acceleration due to gravity, and h is the operating pressure within the manifold. In English units using a 0.6 orifice coefficient, this equation becomes

$$Q = 11.79 d^2 h_d^{1/2}$$

where Q is the discharge rate in gallons per minute, d is the orifice diameter in inches, and h is the operating pressure in feet of water.

Assuming ½-inch taps with a operating pressure of 3 feet of water, the discharge rate from each outlet is

$$Q = 11.79 (1/2)^2 3^{1/2} = 5.1 \text{ gpm}$$

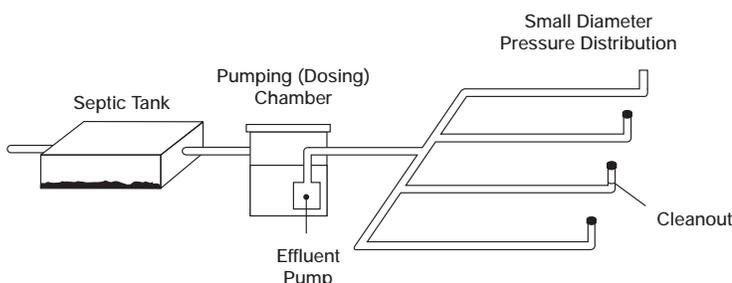
Thus, the pump must be capable of delivering 12×5.1 gpm or approximately 60 gpm against an operating pressure of 3 feet of water plus the static lift and friction losses incurred in the forcemain to the pressure manifold.

wastewater from each orifice in the network when fully pressurized. This is accomplished by maintaining a uniform pressure throughout the network during dosing. The manifolds and laterals are sized relative to the selected orifice size and spacing to achieve uniform pressure. A manual flushing mechanism should be included to enable periodic flushing of slimes and other solids that accumulate in the laterals.

Design of dosed flow systems

A simplified method of network design has been developed (Otis, 1982). Lateral and manifold sizing is determined using a series of graphs and tables after the designer has selected the desired orifice size and spacing and the distal pressure in the network (typically 1 to 2 feet of head). These graphs and tables were derived by calculating the change in flow and pressure at each orifice between the distal and proximal ends of the network. The method is meant to result in discharge rates from the first and last orifices that differ by no more than 10 percent in any lateral and 15 percent across the entire network. However, subsequent testing of field installations indicated that the design model overestimates the maximum lateral length by as much as 25 percent (Converse and Otis, 1982). Therefore, if the graphs and tables are used, the maximum lateral length for any given orifice size and spacing should not exceed 80 percent of the maximum design length suggested by the lateral sizing graphs. In lieu of using the graphs and tables, a spreadsheet could be written using the equations presented and adjusting the orifice discharge coefficient.

Figure 4-16. Rigid pipe pressure distribution networks with flushing cleanouts



Design procedure for rigid pipe pressure distribution network

The simplified design procedure for rigid pipe pressure networks as presented by Otis (1982) includes the following steps:

1. Lay out the proposed network.
2. Select the desired orifice size and spacing. Maximize the density of orifices over the infiltration surface, keeping in mind that the dosing rate increases as the orifice size increases and the orifice spacing decreases.
3. Determine the appropriate lateral pipe diameter compatible with the selected orifice size and spacing using a spreadsheet or sizing charts from Otis (1982).
4. Calculate the lateral discharge rate using the orifice discharge equation (0.48 discharge coefficient or 80 percent of 0.6).
5. Determine the appropriate manifold size based on the number, spacing, and discharge rate of the laterals using a spreadsheet or sizing table from Otis (1982).
6. Determine the dose volume required. Use either the minimum dose volume equal to 5 times the network volume or the expected daily flow divided by the desired dosing frequency, whichever is larger.
7. Calculate the minimum dosing rate (the lateral discharge times the number of laterals).
8. Select the pump based on the required dosing rate and the total dynamic head (sum of the static lift, friction losses in the forcemain to the network, and the network losses, which are equal to 1.3 times the network operating pressure).

To achieve uniform distribution, the density of orifices over the infiltration surface should be as high as possible. However, the greater the number of orifices used, the larger the pump must be to provide the necessary dosing rate. To reduce the dosing rate, the orifice size can be reduced, but the smaller the orifice diameter, the greater the risk of orifice clogging. Orifice diameters as small as 1/8 inch have been used successfully with septic tank effluent when an effluent screen is used at the septic tank outlet. Orifice spacings typically are 1.5 to 4 feet, but the greater the spacing, the less uniform the distribution because each orifice represents a point load. It is up to the designer to achieve the optimum balance between orifice density and pump size.

The dose volume is determined by the desired frequency of dosing and the size of the network. Often, the size of the network will control design. During filling and draining of the network at the start and end of each dose, the distribution is less uniform. The first holes in the network discharge more during initial pressurization of the network, and the holes at the lowest elevation discharge more as the network drains after each dose. To

minimize the relative difference in discharge volumes, the dose volume should be greater than five times the volume of the distribution network (Otis, 1982). A pump or siphon can be used to pressurize the network.

Dripline pressure network

Drip distribution, which was derived from drip irrigation technology, was recently introduced as a method of wastewater distribution. It is a method of pressure distribution capable of delivering small, precise volumes of wastewater effluent to the infiltration surface. It is the most efficient of the distribution methods and is well suited for all types of SWIS applications. A dripline pressure network consists of several components:

- Dose tank
- Pump
- Prefilter
- Supply manifold
- Pressure regulator (when turbulent, flow emitters are used)

- Dripline
- Emitters
- Vacuum release valve
- Return manifold
- Flush valve
- Controller

The pump draws wastewater effluent from the dose tank, preferably on a timed cycle, to dose the distribution system. Before entering the network, the effluent must be prefiltered through mechanical or granular medium filters. The former are used primarily for large SWIS systems. The backflush water generated from a self-cleaning filter should be returned to the headworks of the treatment system. The effluent enters the supply manifold that feeds each dripline (figure 4-17). If turbulent flow emitters are used, the filtered wastewater must first pass through a pressure regulator to control the

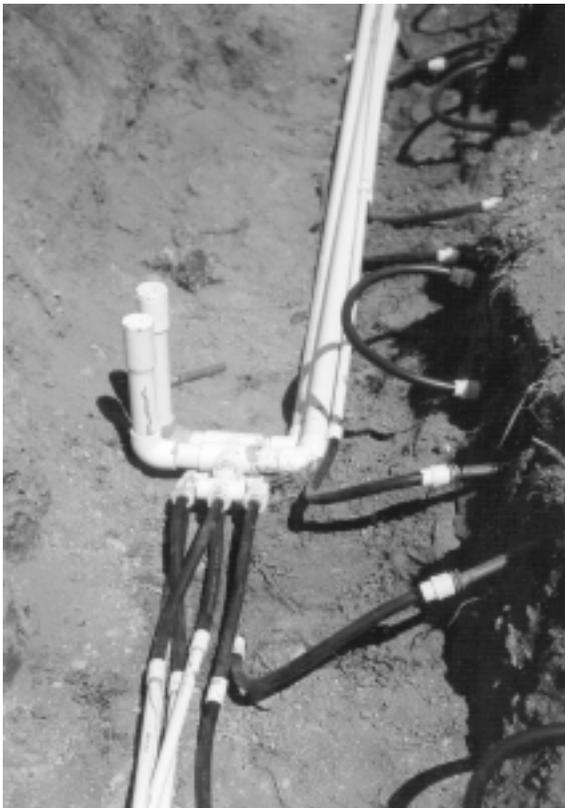
maximum pressure in the dripline. Usually, the dripline is installed in shallow, narrow trenches 1 to 2 feet apart and only as wide as necessary to insert the dripline using a trenching machine or vibratory plow. The trench is backfilled without any porous medium so that the emitter orifices are in direct contact with the soil. The distal ends of each dripline are connected to a return manifold. The return manifold is used to regularly flush the dripline. To flush, a valve on the manifold is opened and the effluent is flushed through the driplines and returned to the treatment system headworks.

Because of the unique construction of drip distribution systems, they cause less site disruption during installation, are adaptable to irregularly shaped lots or other difficult site constraints, and use more of the soil mantle for treatment because of the shallow depth of placement. Also, because the installed cost per linear foot of dripline is usually less than the cost of conventional trench construction, dripline can be added to decrease mass loadings to the infiltration surface at lower costs than other distribution methods. Because of the equipment required, however, drip distribution tends to be more costly to construct and requires regular operation and maintenance by knowledgeable individuals. Therefore, it should be considered for use only where operation and maintenance support is ensured.

The dripline is normally a ½-inch-diameter flexible polyethylene tube with emitters attached to the inside wall spaced 1 to 2 feet apart along its length. Because the emitter passageways are small, friction losses are large and the rate of discharge is low (typically from 0.5 to nearly 2 gallons per hour).

Two types of emitters are used. One is a “turbulent-flow” emitter, which has a very long labyrinth. Flow through the labyrinth reduces the discharge pressure nearly to atmospheric rates. With increasing in-line pressure, more wastewater can be forced through the labyrinth. Thus, the discharges from turbulent flow emitters are greater at higher pressures (figure 4-18). To more accurately control the rate of discharge, a pressure regulator is installed in the supply manifold upstream of the dripline. Inlet pressures from a minimum of 10 psi to a maximum of 45 psi are recommended. The second emitter type is the pressure-compensating

Figure 4-17. Pressure manifold and flexible drip lines prior to trench filling



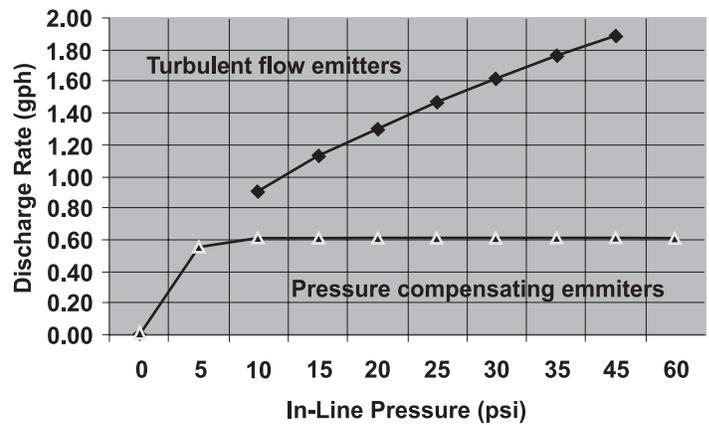
Source: Ayres Associates.

emitter. This emitter discharges at nearly a constant rate over a wide range of in-line pressures (figure 4-18).

Head losses through driplines are high because of the small diameter of the tubing and its in-line emitters, and therefore dripline lengths must be limited. Manufacturers limit lengths at various emitter spacings. With turbulent flow emitters, the discharge from each successive emitter diminishes in response to pressure loss created by friction or by elevation changes along the length of the dripline. With pressure-compensating emitters, the in-line pressure should not drop below 7 to 10 psi at the final emitter. The designer is urged to work with manufacturers to ensure that the system meets their requirements.

Pressure-compensating emitters are somewhat more expensive but offer some important advantages over turbulent-flow emitters for use in onsite wastewater systems. Pressure-compensating dripline is better suited for sloping sites or sites with rolling topography where the dripline cannot be laid on contour. Turbulent-flow emitters discharge more liquid at lower elevations than the same emitters at higher elevations. The designer should limit the difference in discharge rates between emitters to no more than 10 percent. Also, because the discharge rates are equal when under pressure, monitoring flow rates during dosing of a pressure-compensating dripline network can provide an effective way to determine whether leaks or obstructions are present in the network or emitters. Early detection is important so that simple and effective corrective actions can be taken. Usually, injection of a mild bleach solution into the dripline is effective in restoring emitter performance if clogging is due to biofilms. If this action proves to be unsuccessful, other corrective actions are more difficult and costly. An additional advantage of pressure-compensating emitters is that pressure regulators are not required. Finally, when operating in their normal pressure range, pressure-compensating emitters are not affected by soil water pressure in structured soils, which can cause turbulent-flow emitters to suffer reduced dosing volumes.

Figure 4-18. Turbulent-flow and pressure-compensating emitter discharge rates versus in-line pressure



Controlling clogging in drip systems

With small orifices, emitters are susceptible to clogging. Particulate materials in the wastewater, soil particulates drawn into an emitter when the dripline drains following a dose, and biological slimes that grow within the dripline pose potential clogging problems. Also, the moisture and nutrients discharged from the emitters may invite root intrusion through the emitter. Solutions to these problems lie in both the design of the dripline and the design of the distribution network. Emitter hydrodynamic design and biocide impregnation of the dripline and emitters help to minimize some of these problems. Careful network design is also necessary to provide adequate safeguards. Monitoring allows the operator to identify other problems such as destruction from burrowing animals.

To control emitter clogging, appropriate engineering controls must be provided. These include prefiltration of the wastewater, regular dripline flushing, and vacuum release valves on the network. Prefiltration of the effluent through granular or mechanical filters is necessary. These filters should be capable of removing all particulates that could plug the emitter orifices. Dripline manufacturers recommend that self-cleaning filters be designed to remove particles larger than 100 to 115 microns. Despite this disparate experience, pretreatment with filters is recommended in light of the potential cost of replacing plugged emitters. Regular cleaning of the filters is necessary to maintain satisfactory performance. The backflush water should be returned to the head of the treatment works.

The dripline must be flushed on a regular schedule to keep it scoured of solids. Flushing is accomplished by opening the flush valve on the return manifold and increasing the pumping rate to achieve scouring velocity. Each supplier recommends a velocity and procedure for this process. The flushing rate and volume must include water losses (discharge) through the emitters during the flushing event. Both continuous flushing and timed flushing are used. However, flushing can add a significant hydraulic load to the treatment system and must be considered in the design. If intermittent flushing is practiced, flushing should be performed at least monthly.

Aspiration of soil particles is another potential emitter clogging hazard. Draining of the network following a dosing cycle can create a vacuum in the network. The vacuum can cause soil particles to be aspirated into the emitter orifices. To prevent this from occurring, vacuum relief valves are used. It is best to install these at the high points of both the supply and return manifolds.

Placement and layout of drip systems

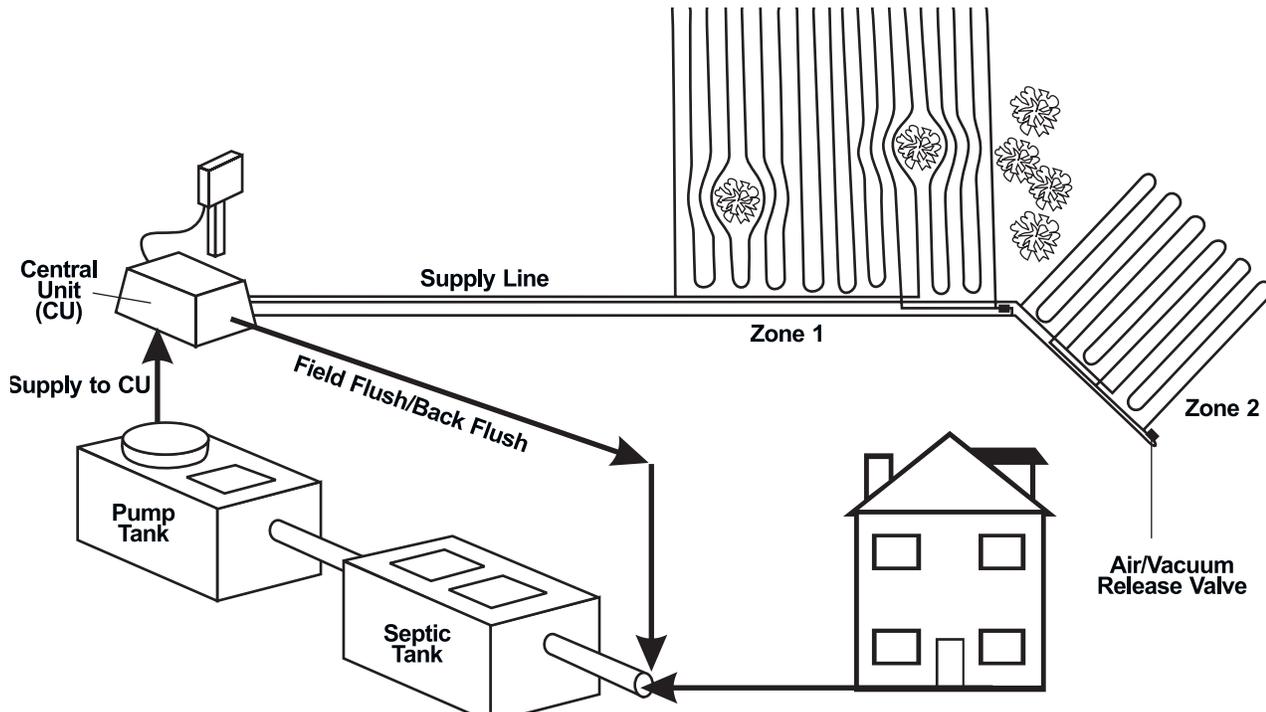
When drip distribution was introduced, the approach to sizing SWISs using this distribution method was substantially different from that for SWISs using other distribution methods. Manufacturer-recommended hydraulic loading rates were expressed in terms of gallons per day per square foot of drip distribution footprint area. Typically, the recommended rates were based on 2-foot emitter and dripline spacing. Therefore, each emitter would serve 4 square feet of footprint area. Because the dripline is commonly plowed into the soil without surrounding it with porous medium, the soil around the dripline becomes the actual infiltration surface. The amount of infiltration surface provided is approximately $2/3$ to 1 square foot per 5 linear feet of dripline. As a result, the wastewater loading rate is considerably greater than the hydraulic loadings recommended for traditional SWISs. Experience has shown however, that the hydraulic loading on this surface can be as much as seven times higher than that of traditional SWIS designs (Ayres Associates, 1994). This is probably due to the very narrow geometry, higher levels of pretreatment, shallow placement, and intermittent loadings of the trenches, all of which help to enhance reaeration of the infiltration surface.

The designer must be aware of the differences between the recommended hydraulic loadings for drip distribution and those customarily used for traditional SWISs. The recommended drip distribution loadings are a function of the soil, dripline spacing, and applied effluent quality. It is necessary to express the hydraulic loading in terms of the footprint area because the individual dripline trenches are not isolated infiltration surfaces. If the emitter and/or dripline spacing is reduced, the wetting fronts emanating from each emitter could overlap and significantly reduce hydraulic performance. Therefore, reducing the emitter and/or dripline spacing should not reduce the overall required system footprint. Reducing the spacing might be beneficial for irrigating small areas of turf grass, but the maximum daily emitter discharge must be reduced proportionately by adding more dripline to maintain the same footprint size. Using higher hydraulic loading rates must be carefully considered in light of secondary boundary loadings, which could result in excessive ground water mounding (see chapter 5). Further, the instantaneous hydraulic loading during a dose must be controlled because storage is not provided in the dripline trench. If the dose volume is too high, the wastewater can erupt at the ground surface.

Layout of the drip distribution network must be considered carefully. Two important consequences of the network layout are the impacts on dose pump sizing necessary to achieve adequate flushing flows and the extent of localized overloading due to internal dripline drainage. Flushing flow rates are a function of the number of manifold/dripline connections: More connections create a need for greater flushing flows, which require a larger pump. To minimize the flushing flow rate, the length of each dripline should be made as long as possible in accordance with the manufacturer's recommendations. To fit the landscape, the dripline can be looped between the supply and return manifolds (figure 4-19). Consideration should also be given to dividing the network into more than one cell to reduce the number of connections in an individual network. A computer program has been developed to evaluate and optimize the hydraulic design for adequate flushing flows of dripline networks that use pressure-compensating emitters (Berkowitz and Harman, 1994).

Internal drainage that occurs following each dose or when the soils around the dripline are saturated

Figure 4-19. Dripline layout on a site with trees



Source: Adapted from American Manufacturing, 2001.

can cause significant hydraulic overloading to lower portions of the SWIS. Following a dose cycle, the dripline drains through the emitters. On sloping sites, the upper driplines drain to the lower driplines, where hydraulic overloading can occur. Any free water around the dripline can enter through an emitter and drain to the lowest elevation. Each of these events needs to be avoided as much as possible through design. The designer can minimize internal drainage problems by isolating the driplines from each other in a cell, by aligning the supply and return manifolds with the site's contours. A further safeguard is to limit the number of doses per day while keeping the instantaneous hydraulic loadings to a minimum so the dripline trench is not flooded following a dose. This trade-off is best addressed by determining the maximum hydraulic loading and adjusting the number of doses to fit this dosing volume.

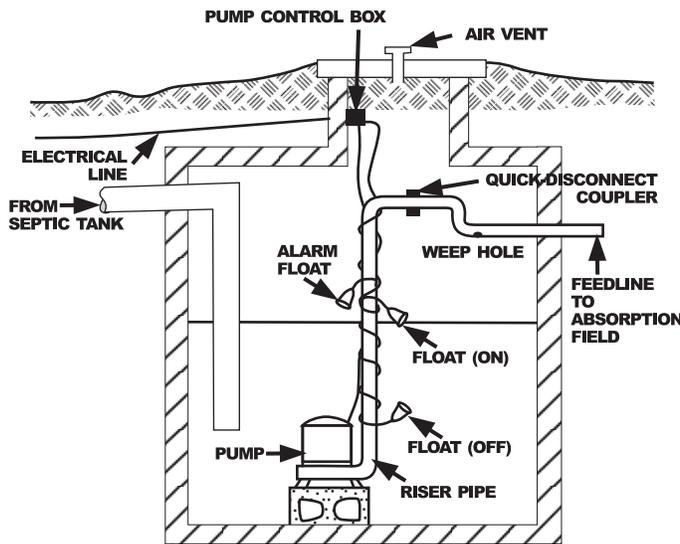
Freezing of dripline networks has occurred in severe winter climates. Limited experience indicates that shallow burial depths together with a lack of uncompacted snow cover or other insulating materials might lead to freezing. In severe winter

climates, the burial depth of dripline should be increased appropriately and a good turf grass established over the network. Mulching the area the winter after construction or every winter should be considered. Also, it is good practice to install the vacuum release valves below grade and insulate the air space around them. Although experience with drip distribution in cold climates is limited, these safeguards should provide adequate protection.

Dosing methods

Two methods of dosing have been used (table 4-6). With on-demand dosing, the wastewater effluent rises to a preset level in the dose tank and the pump or siphon is activated by a float switch or other mechanism to initiate discharge (figure 4-20). During peak-flow periods, dosing is frequent with little time between doses for the infiltration system to drain and the subsoil to reaerate. During low-flow periods, dosing intervals are long, which can be beneficial in controlling biomat development but is inefficient in using the hydraulic capacity of the system.

Figure 4-20. Pumping tank (generic)



Source: Purdue University, 1990

Timed dosing overcomes some of the shortcomings of on-demand dosing. Timers are used to turn the pump on and off at specified intervals so that only a predetermined volume of wastewater is discharged with each dose. Timed dosing has two distinct advantages over on-demand dosing. First, the doses can be spaced evenly over the entire 24-hour day to optimize the use of the soil's treatment capacity. Second, the infiltration system receives no more than its design flow each day. Clear water infiltration, leaking plumbing fixtures, or excessive water use are detected before the excess flow is discharged to the infiltration system because the dose tank will eventually fill to its high water alarm level. At that point, the owner has the option of calling a septage pumper to empty the tanks or activating the pump to dose the system until the problem is diagnosed and corrected. Unlike on-demand dosing, timed dosing requires that the dose tank be sized to store peak flows until they can be pumped (see sidebar).

Dosing frequency and volume are two important design considerations. Frequent, small doses are preferred over large doses one or two times per day. However, doses should not be so frequent that distribution is poor. This is particularly true with either of the pressure distribution networks. With pressure networks, uniform distribution does not occur until the entire network is pressurized. To ensure pressurization and to minimize unequal discharges from the orifices during filling and draining, a dose volume equal to five times the

network volume is a good rule of thumb. Thus, doses can be smaller and more frequent with dripline networks than with rigid pipe networks because the volume of drip distribution networks is smaller.

4.4.8 SWISmedia

A porous medium is placed below and around SWIS distribution piping to expand the infiltration surface area of the excavation exposed to the applied wastewater. This approach is similar in most SWIS designs, except when drip distribution or aggregate-free designs are used. In addition, the medium also supports the excavation sidewalls, provides storage of peak wastewater flows, minimizes erosion of the infiltration surface by dissipating the energy of the influent flow, and provides some protection for the piping from freezing and root penetration.

Traditionally, washed gravel or crushed rock, typically ranging from $\frac{3}{4}$ to $2\frac{1}{2}$ inches in diameter, has been used as the porous medium. The rock should be durable, resistant to slaking and dissolution, and free of fine particles. A hardness of at least 3 on the Moh's scale of hardness is suggested. Rock that can scratch a copper penny without leaving any residual meets this criterion. It is important that the medium be washed to remove fine particles. Fines from insufficiently washed rock have been shown to result in significant reductions in infiltration rates (Amerson et al., 1991). In all applications where gravel is used, it must be properly graded and washed. Improperly washed gravel can contribute fines and other material that can plug voids in the infiltrative surface and reduce hydraulic capability. Gravel that is embedded into clay or fine soils during placement can have the same effect.

In addition to natural aggregates, gravelless systems have been widely used as alternative SWIS medium (see preceding section). These systems take many forms, including open-bottomed chambers, fabric-wrapped pipe, and synthetic materials such as expanded polystyrene foam chips, as described in the preceding section. Systems that provide an open chamber are sometimes referred to as "aggregate-free" systems, to distinguish them from others that substitute lightweight medium for gravel or stone. These systems provide a suitable substitute in locales where gravel is not available or affordable. Some systems (polyethylene chambers and light-

Dose tank sizing for timed dosing

Timed dosing to a SWIS is to be used in an onsite system serving a restaurant in a summer resort area. Timed dosing will equalize the flows, enhancing treatment in the soil and reducing the required size of the SWIS.

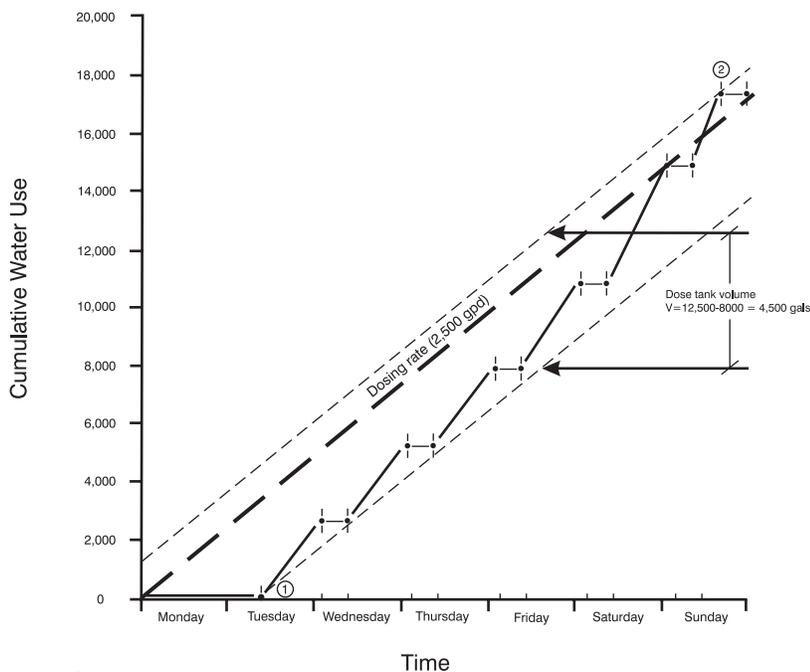
The restaurant serves meals from 11 a.m. to 12 midnight Tuesday through Saturday and from 9 a.m. to 2 p.m. Sundays. The largest number of meals is served during the summer weekends. The restaurant is closed on Mondays. The metered water use is as follows:

Average weekly water use (summer)	17,500 gal
Peak weekend water use (4 p.m. Friday to 2 p.m. Sunday)	9,500 gal

The dose tank will be sized to equalize flows over a 7-day period. The dosing frequency is to be six times daily or one dose every 4 hours. Therefore, the dose volume will be

$$\text{Dose volume} = 17,500 \text{ gal/wk} \div (7 \text{ d/wk} \times 6 \text{ doses/day}) = 417 \text{ gal/dose}$$

The necessary volume of the dose tank to store the peak flows and equalize the flow to the SWIS over the 7-day week can be determined graphically.



Source: Ayres Associates.

The accumulated water use over the week and the daily dosing rate (6 doses/day x 417 gal/dose = 2,500 gpd) is plotted on the graph. Lines parallel to the dosing rate are drawn tangent to points 1 and 2 representing the maximum deviations of the water use line above and below the dosing rate line. The volume represented by the difference between the two parallel lines is the tank volume needed to achieve flow equalization. A 4,500-gallon tank would be required.

Both siphons and pumps can be used for dosing distribution networks. Only drip distribution networks cannot be dosed by siphons because of the higher required operating pressures and the need to control instantaneous hydraulic loadings (dose volume). Siphons can be used where power is not available and elevation is adequate to install the siphon sufficiently above the distribution network to overcome friction losses in the forcemain and network. Care must be taken in their selection and installation to ensure proper performance. Also, owners must be aware that siphon systems require routine monitoring and occasional maintenance. “Dribbling” can occur when the siphon bell becomes saturated, suspending dosing and allowing the wastewater effluent to trickle out under the bell. Dribbling can occur because of leaks in the bell or a siphon out of adjustment. Today, pumps are favored over siphons because of the greater flexibility in site selection and dosing regime.

weight aggregate systems) can also offer substantial advantages in terms of reduced site disruption over the traditional gravel because their light weight makes them easy to handle without the use of heavy equipment. These advantages reduce labor costs, limit damage to the property by machinery, and allow construction on difficult sites where conventional medium could not reasonably be used.

4.5 Construction management and contingency options

Onsite wastewater systems can and do fail to perform at times. To avoid threats to public health and the environment during periods when a system malfunctions hydraulically, contingency plans should be made to permit continued use of the system until appropriate remedial actions can be taken. Contingency options should be considered during design so that the appropriate measures are designed into the original system. Table 4-8 lists common contingency options.

4.5.1 Construction considerations

Construction practices are critical to the performance of SWISs. Satisfactory SWIS performance depends on maintaining soil porosity. Construction activities can significantly reduce the porosity and cause SWISs to hydraulically fail soon after being brought into service. Good construction practices should carefully consider site protection before and during construction, site preparation, and construction equipment selection and use. Good construction practices for at-grade and mound systems can be found elsewhere (Converse and Tyler, 2000; Converse et al., 1990). Many of them, however, are similar to those described in the following subsections.

Site protection

Construction of the onsite wastewater system is often only one of many construction activities that occur on a property. If not protected against intrusion, the site designated for the onsite system can be damaged by other, unrelated construction

Table 4-8. Contingency options for SWIS malfunctions

Contingency option	Description	Comments
Reserve area	Unencumbered area of suitable soils set aside for a future replacement system.	Does not provide immediate relief from performance problems because the replacement system must be constructed. The replacement system should be constructed such that use can be alternated with use of the original system.
Multiple cells	Two or more infiltration cells with a total hydraulic capacity of 100% to 200% of the required area that are alternated into service.	Provide immediate relief from performance problems by providing stand-by capacity. Rotating cells in and out of service on an annual or other regular schedule helps to maintain system capacity. Alternating valves are commercially available to implement this option. The risk from performance problems is reduced because the malfunction of a single cell involves a smaller proportion of the daily flow.
Water conservation	Water-conserving actions taken to reduce the hydraulic load to the system, which may alleviate the problem.	A temporary solution that may necessitate a significant lifestyle change by the residents, which creates a disincentive for continued implementation. The organic loading will remain the same unless specific water uses or waste inputs are eliminated from the building or the wastewaters are removed from the site.
Pump and haul	Conversion of the septic tank to a holding tank that must be periodically pumped. The raw waste must be hauled to a suitable treatment and/or disposal site.	Holding tanks are a temporary or permanent solution that can be effective but costly, creating a disincentive for long-term use.

activities. Therefore, the site should be staked and roped off before any construction activities begin to make others aware of the site and to keep traffic and materials stockpiles off the site.

The designer should anticipate what activities will be necessary during construction and designate acceptable areas for them to occur. Site access points and areas for traffic lanes, material stockpiling, and equipment parking should be designated on the drawings for the contractor.

Site preparation

Site preparation activities include clearing and surface preparation for filling. Before these activities are begun, the soil moisture should be determined. In nongranular soils, compaction will occur if the soil is near its plastic limit. This can be tested by removing a sample of soil and rolling it between the palms of the hands. If the soil fails to form a “rope” the soil is sufficiently dry to proceed. However, constant care should be taken to avoid soil disturbance as much as possible.

Clearing

Clearing should be limited to mowing and raking because the surface should be only minimally disturbed. If trees must be removed, they should be cut at the base of the trunk and removed without heavy machinery. If it is necessary to remove the stumps, they should be ground out. Grubbing of the site (mechanically raking away roots) should be avoided. If the site is to be filled, the surface should be moldboard- or chisel-plowed parallel to the contour (usually to a depth of 7 to 10 inches) when the soil is sufficiently dry to ensure maximum vertical permeability. The organic layer should not be removed. Scarifying the surface with the teeth of a backhoe bucket is not sufficient.

Excavation

Excavation activities can cause significant reductions in soil porosity and permeability (Tyler et al., 1985). Compaction and smearing of the soil infiltrative surface occur from equipment traffic and vibration, scraping actions of the equipment, and placement of the SWIS medium on the infiltration surface. Lightweight backhoes are most commonly used. Front-end loaders and blades should not be used

because of their scraping action. All efforts should be made to avoid any disturbance to the exposed infiltration surface. Equipment should be kept off the infiltration field. Before the SWIS medium is installed, any smeared areas should be scarified and the surface gently raked. If gravel or crushed rock is to be used for SWIS medium, the rock should be placed in the trench by using the backhoe bucket rather than dumping it directly from the truck. If damage occurs, it might be possible to restore the area, but only by removing the compacted layer. It might be necessary to remove as much as 4 inches of soil to regain the natural soil porosity and permeability (Tyler et al., 1985). Consequences of the removal of this amount of soil over the entire infiltration surface can be significant. It will reduce the separation distance to the restrictive horizon and could place the infiltration surface in an unacceptable soil horizon.

To avoid potential soil damage during construction, the soil below the proposed infiltration surface elevation must be below its plastic limit. This should be tested before excavation begins. Also, excavation should be scheduled only when the infiltration surface can be covered the same day to avoid loss of permeability from wind-blown silt or raindrop impact. Another solution is to use lightweight gravelless systems, which reduce the damage and speed the construction process.

Before leaving the site, the area around the site should be graded to divert surface runoff from the SWIS area. The backfill over the infiltration surface should be mounded slightly to account for settling and eliminate depressions over the system that can pond water. Finally, the area should be seeded and mulched.

4.5.2 Operation, maintenance, and monitoring

Subsurface wastewater infiltration systems require little operator intervention. Table 4-9 lists typical operation, maintenance, and monitoring activities that should be performed. However, more complex pretreatment, larger and more variable flows, and higher-risk installations increase the need for maintenance and monitoring. More information is provided in the USEPA draft *Guidelines for Onsite/Decentralized Wastewater Systems* (2000) and in the chapter 4 fact sheets.

Table 4-9. Operation, maintenance, and monitoring activities

Task	Description	Frequency
Water meter reading	Recommended for large, commercial systems	Daily
Dosing tank controls	Check function of pump, switches, and timers for pressure-dosed systems	Monthly
Pump calibration	Check pumping rate and adjust dose timers as appropriate for pressure-dosed systems	Annually
Infiltration cell rotation	Direct wastewater to standby cells to rest operating cells	Annually (optimally in the spring)
Infiltration surface ponding	Record wastewater ponding depths over the infiltration surface and switch to standby cell when ponding persists for more than a month	Monthly
Inspect surface and perimeter of SWIS	Walk over SWIS area to observe surface ponding or other signs of stress or damage	Monthly
Tank solids levels and integrity assessment	Check for sludge and scum accumulation, condition of baffles and inlet and outlet appurtenances, and potential leaks	Varies with tank size and management program

4.5.3 Considerations for large and commercial systems

Designs for systems treating larger flows follow the same guidelines used for residential systems, but they must address characteristics of the wastewater to be treated, site characteristics, infiltration surface sizing, and contingency planning more comprehensively.

Wastewater characteristics

Wastewaters from cluster systems serving multiple homes or commercial establishments can differ substantially in flow pattern and waste strength from wastewaters generated by single family residences. The ratio of peak to average daily flow from residential clusters is typically much lower than what is typical from single residences. This is because the moderating effect associated with combining multiple water use patterns reduces the daily variation in flow. Commercial systems, on the other hand, can vary significantly in wastewater strength. Typically, restaurants have high concentrations of grease and BOD, laundromats have high sodium and suspended solids concentrations, and toilet facilities at parks and rest areas have higher concentrations of BOD, TSS, and nitrogen. These differences in daily flow patterns and waste strengths must be dealt with in the design of SWISs. Therefore, it is important to

characterize the wastewater fully before initiating design (see chapter 3).

Site characteristics

The proposed site for a SWIS that will treat wastewater from a cluster of homes or a commercial establishment must be evaluated more rigorously than a single-residence site because of the larger volume of water that is to be applied and the greater need to determine hydraulic gradients and direction. SWIS discharges can be from 10 to more than 100 times the amount of water that the soil infiltration surface typically receives from precipitation. For example, assume that an area receives an average of 40 inches of rainfall per year. Of that, less than 25 percent (about 10 inches annually) infiltrates and even less percolates to the water table. A wastewater infiltration system is designed to infiltrate 0.4 to 1.6 inches per day, or 146 to 584 inches per year. Assuming actual system flows are 30 percent of design flows, this is reduced to 44 to 175 inches per year even under this conservative approach.

The soils associated with small systems can usually accommodate these additional flows. However, systems that treat larger flows load wastewaters to the soil over a greater area and might exceed the site’s capacity to accept the wastewater. Restrictive horizons that may inhibit deep percolation need to

be identified before design. Ground water mounding analysis should be performed to determine whether the hydraulic loading to the saturated zone (secondary design boundary), rather than the loading to the infiltration surface, controls system sizing (see Chapter 5). If the secondary boundary controls design, the size of the infiltration surface, its geometry, and even how wastewater is applied will be affected.

Infiltration surface sizing

Selection of the design flow is a very important consideration in infiltration surface sizing. State codified design flows for residential systems typically are 2 to 5 times greater than the average daily flow actually generated in the home. This occurs because the design flow is usually based on the number of bedrooms rather than the number of occupants. As a result, the actual daily flow is often a small fraction of the design flow.

This is not the case when the per capita flows for the population served or metered flows are used as the design flow. In such instances, the ratio of design flow to actual daily flow can approach unity. This is because the same factors of safety are typically not used to determine the design flow. In itself, this is not a problem. The problem arises when the metered or averaged hydraulic loading rates are used to size the infiltration surface. These rates can be more than two times what the soil below the undersized system is actually able to accept. As a result, SWISs would be significantly undersized. This problem is exacerbated where the waste strength is high.

To avoid the problem of undersizing the infiltration surface, designs must compensate in some way. Factors of safety of up to 2 or more could be applied to accurate flow estimates, but the more common practice is to design multiple cells that provide 150 to 200 percent of the total estimated infiltration surface needed. Multiple cells are a good approach because the cells can be rotated into service on a regular schedule that allows the cells taken out of service to rest and rejuvenate their hydraulic capacity. Further, the system provides standby capacity that can be used when malfunctions occur, and distribution networks are smaller to permit smaller and more frequent dosing, thereby maximizing oxygen transfer and the hydraulic capacity of the site. For high-strength wastewaters, advanced pretreatment can be speci-

fied or the infiltration surface loadings can be adjusted (see *Special Issue Fact Sheet 4*).

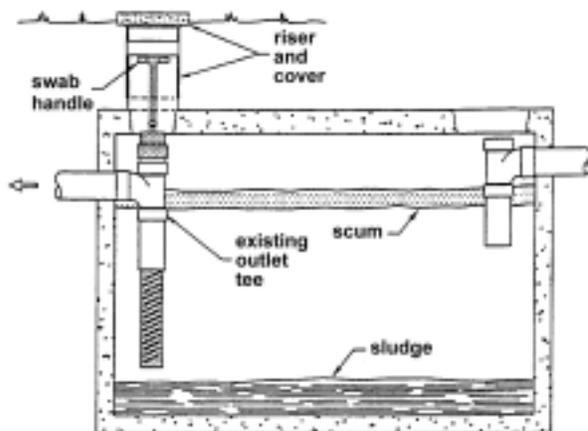
Contingency planning

Malfunctions of systems that treat larger flows can create significant public health and environmental hazards. Therefore, adequate contingency planning is more critical for these systems than for residential systems. Standby infiltration cells, timed dosing, and flow monitoring are key design elements that should be included. Also, professional management should be required.

4.6 Septictanks

The septic tank is the most commonly used wastewater pretreatment unit for onsite wastewater systems. Tanks may be used alone or in combination with other processes to treat raw wastewater before it is discharged to a subsurface infiltration system. The tank provides primary treatment by creating quiescent conditions inside a covered, watertight rectangular, oval, or cylindrical vessel, which is typically buried. In addition to primary treatment, the septic tank stores and partially digests settled and floating organic solids in sludge and scum layers. This can reduce the sludge and scum volumes by as much as 40 percent, and it conditions the wastewater by hydrolyzing organic molecules for subsequent treatment in the soil or by other unit processes (Baumann et al., 1978). Gases generated from digestion of the organics are vented back through the building sewer and out of the house plumbing stack vent. Inlet structures are designed to limit short circuiting of incoming wastewater across the tank to the outlet, while outlet structures (e.g., a sanitary “tee” fitting) retain the sludge and scum layers in the tank and draw effluent only from the clarified zone between the sludge and scum layers. The outlet should be fitted with an effluent screen (commonly called a septic tank filter) to retain larger solids that might be carried in the effluent to the SWIS, where it could contribute to clogging and eventual system failure. Inspection ports and manways are provided in the tank cover to allow access for periodically removing the tank contents, including the accumulated scum and sludge (figure 4-21). A diagram of a two-compartment tank is shown later in this section.

Septic tanks are used as the first or only pretreatment step in nearly all onsite systems regardless of

Figure 4-21. Profile of a single-compartment septic tank with outlet screen

daily wastewater flow rate or strength. Other mechanical pretreatment units may be substituted for septic tanks, but even when these are used septic tanks often precede them. The tanks passively provide suspended solids removal, solids storage and digestion, and some peak flow attenuation.

4.6.1 Treatment

A septic tank removes many of the settleable solids, oils, greases, and floating debris in the raw wastewater, achieving 60 to 80 percent removal (Baumann et al., 1978; Boyer and Rock, 1992; University of Wisconsin, 1978). The solids removed are stored in sludge and scum layers, where they undergo liquefaction. During liquefaction, the first step in the digestion process, acid-forming bacteria

partially digest the solids by hydrolyzing the proteins and converting them to volatile fatty acids, most of which are dissolved in the water phase. The volatile fatty acids still exert much of the biochemical oxygen demand that was originally in the organic suspended solids. Because these acids are in the dissolved form, they are able to pass from the tank in the effluent stream, reducing the BOD removal efficiency of septic tanks compared to primary sedimentation. Typical septic tank BOD removal efficiencies are 30 to 50 percent (Boyer and Rock, 1992; University of Wisconsin, 1978; see table 4-10). Complete digestion, in which the volatile fatty acids are converted to methane, could reduce the amount of BOD released by the tank, but it usually does not occur to a significant extent because wastewater temperatures in septic tanks are typically well below the optimum temperature for methane-producing bacteria.

Gases that form from the microbial action in the tank rise in the wastewater column. The rising gas bubbles disturb the quiescent wastewater column, which can reduce the settling efficiency of the tank. They also dislodge colloidal particles in the sludge blanket so they can escape in the water column. At the same time, however, they can carry active anaerobic and facultative microorganisms that might help to treat colloidal and dissolved solids present in the wastewater column (Baumann and Babbitt, 1953).

Septic tank effluent varies naturally in quality depending on the characteristics of the wastewater and condition of the tank. Documented effluent quality from single-family homes, small communities and cluster systems, and various commercial septic tanks is presented in tables 4-10 through 4-12.

Table 4-10. Characteristics of domestic septic tank effluent

Parameter	University of Wis. (1978)	Harkin, et al. (1979)	Ronayne, et al. (1982)	Ayres Associates (1993)	Ayres Associates (1996)
No. tanks sampled	7	33	8	8	1
Location (No. samples)	Wisconsin (150)	Wisconsin (140 - 215)	Oregon (56)	Florida (36)	Florida (3)
BOD ₅ (mg/L)	138	132	217	141	179
COD (mg/L)	327	445	—	—	—
TSS (mg/L)	49	87	146	161	59
TKN (mgN/L)	45	82	57.1	39	66
TP (mgP/l)	13	21.8	—	11	17
Oil/Grease (mg/L)	—	—	—	36	37
Fecal coliforms (log#/L)	4.6	6.5	6.4	5.1-8.2	7.0

Table 4-11. Average septic tank effluent concentrations for selected parameters from small community and cluster systems

Parameter	Westboro, WI ^a	Bend, OR ^b	Glide, OR ^c	Manila, CA ^d	College Sta., TX ^e
BOD ₅ (mg/L)	168	157	118	189	--
COD (mg/L)	338	276	228	284	266
TSS (mg/L)	85	36	52	75	--
TN (mgN/L)	63.4	41	50	--	29.5
TP (mgP/L)	8.1	--	--	--	8.2
Oil/Grease (mg/L)	--	65	16	22	--
Fecal coliforms (log#/L)	7.3	--	--	--	6.0
pH	6.9–7.4	6.4–7.2	6.4–7.2	6.5–7.8	7.4
Flow (gpcd)	36	40–60	48	40–57	--

^a Small-diameter gravity sewer serving a small community collecting septic tank effluent from 90 connections (Otis, 1978).

^b Pressure sewer collecting septic tank effluent from eleven homes (Bowne, 1982).

^c Pressure sewer collecting septic tank effluent from a small community (Bowne, 1982).

^d Pressure sewer serving a small community collecting septic tank effluent from 330 connections (Bowne, 1982).

^e Effluent from one septic tank accepting wastewater from nine homes (Brown et al., 1977).

Table 4-12. Average septic tank effluent concentrations of selected parameters from various commercial establishments^a

Wastewater Type	BOD ₅ (mg/L)	COD (mg/L)	TSS (mg/L)	TKN (mgN/L)	TP (mgP/L)	Oil/Grease (mg/L)	Temp (°C)	pH
Restaurant A	582	1196	187	82	24	101	8–22	5.6–6.4
Restaurant B	245	622	65	64	14	40	8–22	6.6–7.0
Restaurant C	880	1667	372	71	23	144	13–23	5.8–6.3
Restaurant D	377	772	247	30	15	101	16–21	5.7–6.8
Restaurant E	693	1321	125	78	28	65	4–26	5.5–6.9
Restaurant F	261	586	66	73	19	47	7–25	5.8–7.0
Motel	171	381	66	34	20	45	20–28	6.5–7.1
Country Club A	197	416	56	36	13	24	6–20	6.5–6.8
Country Club B	333	620	121	63	17	46	13–26	6.2–6.8
Country Club C	101	227	44	36	10	33	10–23	6.2–7.4
Bar/Grill	179	449	79	61	7	49	8–22	6.0–7.0

^a Averages based on 2 to 9 grab samples depending on the parameter taken between March and September 1983.

Source: Siegrist et al., 1985.

Volume

4.6.2 Design considerations

The primary purpose of a septic tank is to provide suspended solids and oil/grease removal through sedimentation and flotation. The important factor to achieving good sedimentation is maintaining quiescent conditions. This is accomplished by providing a long wastewater residence time in the septic tank. Tank volume, geometry, and compartmentalization affect the residence time.

Septic tanks must have sufficient volume to provide an adequate hydraulic residence time for sedimentation. Hydraulic residence times of 6 to 24 hours have been recommended (Baumann and Babbitt, 1953; Kinnicutt et al., 1910). However, actual hydraulic residence times can vary significantly from tank to tank because of differences in geometry, depth, and inlet and outlet configurations (Baumann and Babbitt, 1953). Sludge and scum also affect the residence time, reducing it as the solids accumulate.

Table 4-13. Septic tank capacities for one- and two-family dwellings (ICC, 1995).

Number of bedrooms	Septic tank volume (gallons)
1	750 ^a
2	750 ^a
3	1,000
4	1,200
5	1,425
6	1,650
7	1,875
8	2,100

^a Many states have established 1,000 gallons or more as the minimum size.

Most state and national plumbing codes specify the tank volume to be used based on the building size or estimated peak daily flow of wastewater. Table 4-13 presents the tank volumes recommended in the International Private Sewage Disposal Code specified for one- and two-family residences (ICC, 1995). The volumes specified are typical of most local codes, but in many jurisdictions the minimum tank volume has been increased to 1,000 gallons or more. For buildings other than one- or two-family residential homes, the rule of thumb often used for sizing tanks is to use two to three times the esti-

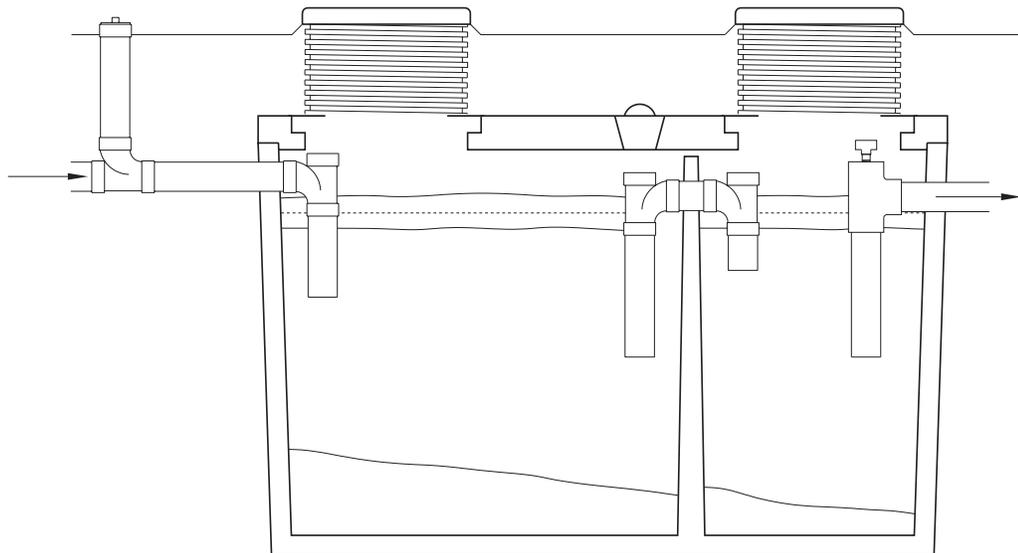
mated design flow. This conservative rule of thumb is based on maintaining a 24-hour minimum hydraulic retention time when the tank is ready for pumping, for example, when the tank is one-half to two-thirds full of sludge and scum.

Geometry

Tank geometry affects the hydraulic residence time in the tank. The length-to-width ratio and liquid depth are important considerations. Elongated tanks with length-to-width ratios of 3:1 and greater have been shown to reduce short-circuiting of the raw wastewater across the tank and improve suspended solids removal (Ludwig, 1950). Prefabricated tanks generally are available in rectangular, oval, and cylindrical (horizontal or vertical) shapes. Vertical cylindrical tanks can be the least effective because of the shorter distance between the inlets and outlets. Baffles are recommended.

Among tanks of equal liquid volumes, the tank with shallower liquid depths better reduces peak outflow rates and velocities, so solids are less likely to remain in suspension and be carried out of the tank in the effluent. This is because the shallow tank has a larger surface area. Inflows to the tank cause less of a liquid rise because of the larger surface area. The rate of flow exiting the tank (over a weir or through a pipe invert) is propor-

Figure 4-22. Two-compartment tank with effluent screen and surface risers



Source: Washington Department of Health, 1998.

tional to the height of the water surface over the invert (Baumann et al., 1978; Jones, 1975). Also, the depth of excavation necessary is reduced with shallow tanks, which helps to avoid saturated horizons and lessens the potential for ground water infiltration or tank flotation. A typically specified minimum liquid depth below the outlet invert is 36 inches. Shallower depths can disturb the sludge blanket and, therefore, require more frequent pumping.

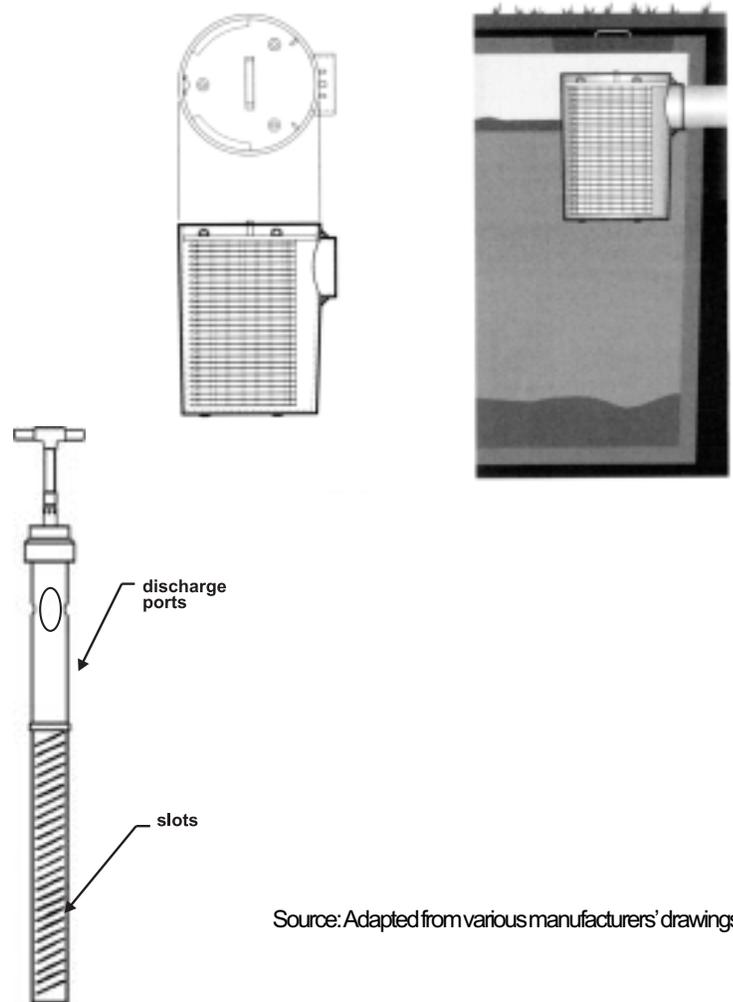
Compartmentalization

Compartmentalized tanks (figure 4-23) or tanks placed in series provide better suspended solids removal than single-compartment tanks alone, although results from different studies vary (Baumann and Babbitt, 1953; Boyer and Rock, 1992; Weibel et al., 1949, 1954; University of Wisconsin, 1978). If two compartments are used, better suspended solids removal rates are achieved if the first compartment is equal to one-half to two-thirds the total tank volume (Weibel et al., 1949, 1954). An air vent between compartments must be provided to allow both compartments to vent. The primary advantage of these configurations is when gas generated from organic solids digestion in the first compartment is separated from subsequent compartments.

Inlets and outlets

The inlet and outlet of a septic tank are designed to enhance tank performance. Their respective invert elevations should provide at least a 2- to 3-inch drop across the tank to ensure that the building sewer does not become flooded and obstructed during high wastewater flows (figure 4-24). A clear space of at least 9 inches should be provided above the liquid depth (outlet invert) to allow for scum storage and ventilation. Both the inlet and outlet are commonly baffled. Plastic sanitary tees are the most commonly used baffles. Curtain baffles (concrete baffles cast to the tank wall and fiberglass or plastic baffles bolted to the tank wall) have also been used. The use of gasket materials that achieve a watertight joint with the tank wall makes plastic sanitary tees easy to adjust, repair, or equip with effluent screens or filters. The use of a removable, cleanable effluent screen connected to the outlet is strongly recommended.

Figure 4-23. Examples of septic tank effluent screens/filters



Source: Adapted from various manufacturers' drawings.

The inlet baffle is designed to prevent short-circuiting of the flow to the outlet by dissipating the energy of the influent flow and deflecting it downward into the tank. The rising leg of the tee should extend at least 6 inches above the liquid level to prevent the scum layer from plugging the inlet. It should be open at the top to allow venting of the tank through the building sewer and out the plumbing stack vent. The descending leg should extend well into the clear space between the sludge and scum layers, but not more than about 30 to 40 percent of the liquid depth. The volume of the descending leg should not be larger than 2 to 3 gallons so that it is completely flushed to expel floating materials that could cake the inlet. For this reason, curtain baffles should be avoided.

The outlet baffle is designed to draw effluent from the clear zone between the sludge and scum layers. The rising leg of the tee should extend 6 inches above the liquid level to prevent the scum layer from escaping the tank. The descending leg should extend to 30 or 40 percent of the liquid depth. Effluent screens (commonly called septic tank filters), which can be fitted to septic tank outlets, are commercially available. Screens prevent solids that either are buoyant or are resuspended from the scum or sludge layers from passing out of the tank (figures 4-22 and 4-23). Mesh, slotted screens, and stacked plates with openings from 1/32 to 1/8 inch are available. Usually, the screens can be fitted into the existing outlet tee or retrofitted directly into the outlet. An access port directly above the outlet is required so the screen can be removed for inspection and cleaning.

Quality-assured, reliable test results have not shown conclusively that effluent screens result in effluents with significantly lower suspended solids and BOD concentrations. However, they provide an excellent, low-cost safeguard against neutral-buoyancy solids and high suspended solids in the tank effluent resulting from solids digestion or other upsets. Also, as the effluent screens clog over time, slower draining and flushing of home fixtures may alert homeowners of the need for maintenance before complete blockage occurs.

Tank access

Access to the septic tank is necessary for pumping septage, observing the inlet and outlet baffles, and servicing the effluent screen. Both manways and inspection ports are used. Manways are large openings, 18 to 24 inches in diameter or square. At least one that can provide access to the entire tank for septage removal is needed. If the system is compartmentalized, each compartment requires a manway. They are located over the inlet, the outlet, or the center of the tank. Typically, in the past manway covers were required to be buried under state and local codes. However, they should be above grade and fitted with an airtight, lockable cover so they can be accessed quickly and easily. Inspection ports are 8 inches or larger in diameter and located over both the inlet and the outlet unless a manway is used. They should be extended above grade and securely capped.

(CAUTION: The screen should not be removed for inspection or cleaning without first plugging the outlet or pumping the tank to lower the liquid level below the outlet invert. Solids retained on the screen can slough off as the screen is removed. These solids will pass through the outlet and into the SWIS unless precautions are taken. This caution should be made clear in homeowner instructions and on notices posted at the access port.)

Septic tank designs for large wastewater flows do not differ from designs for small systems. However, it is suggested that multiple compartments or tanks in series be used and that effluent screens be attached to the tank outlet. Access ports and manways should be brought to grade and provided with locking covers for all large systems.

Construction materials

Septic tanks smaller than 6,000 gallons are typically premanufactured; larger tanks are constructed in place. The materials used in premanufactured tanks include concrete, fiberglass, polyethylene, and coated steel. Precast concrete tanks are by far the most common, but fiberglass and plastic tanks are gaining popularity. The lighter weight fiberglass and plastic tanks can be shipped longer distances and set in place without cranes. Concrete tanks, on the other hand, are less susceptible to collapse and flotation. Coated steel tanks are no longer widely used because they corrode easily. Tanks constructed in place are typically made of concrete.

Tanks constructed of fiberglass-reinforced polyester (FRP) usually have a wall thickness of about 1/4 inch (6 millimeters). Most are gel- or resin-coated to provide a smooth finish and prevent glass fibers from becoming exposed, which can cause wicking. Polyethylene tanks are more flexible than FRP tanks and can deform to a shape of structural weakness if not properly designed. Concrete tank walls are usually about 4 inches thick and reinforced with no. 5 rods on 8-inch (20-centimeter) centers. Sulfuric acid and hydrogen sulfide, both of which are present in varying concentrations in septic tank effluent, can corrode exposed rods and the concrete itself over time. Some plastics (e.g., polyvinyl chloride, polyethylene, but not nylon) are virtually unaffected by acids and hydrogen sulfide (USEPA, 1991).

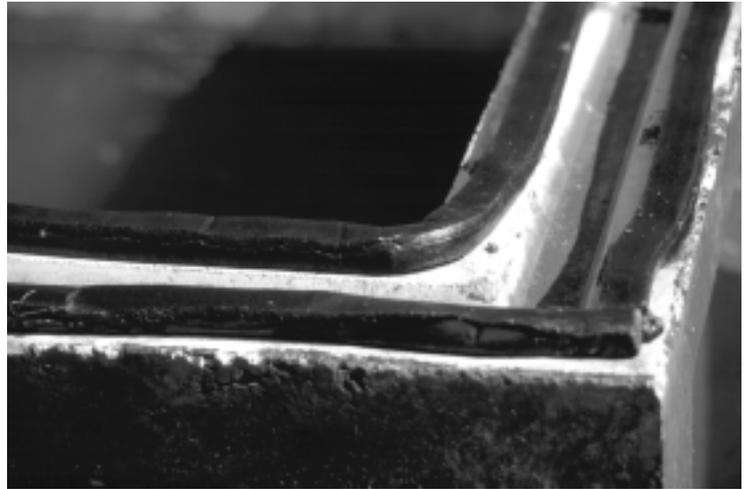
Quality construction is critical to proper performance. Tanks must be properly designed, reinforced, and constructed of the proper mix of materials so they can meet anticipated loads without cracking or collapsing. All joints must be watertight and flexible to accommodate soil conditions. For concrete tank manufacturing, a “best practices manual” can be purchased from the National Pre-Cast Concrete Association (NPCA, 1998). Also, a *Standard Specification for Precast Concrete Septic Tanks (C 1227)* has been published by the American Society for Testing and Materials (ASTM, 1998).

Watertightness

Watertightness of the septic tank is critical to the performance of the entire onsite wastewater system. Leaks, whether exfiltrating or infiltrating, are serious. Infiltration of clear water to the tank from the building storm sewer or ground water adds to the hydraulic load of the system and can upset subsequent treatment processes. Exfiltration can threaten ground water quality with partially treated wastewater and can lower the liquid level below the outlet baffle so it and subsequent processes can become fouled with scum. Also, leaks can cause the tank to collapse.

Tank joints should be designed for watertightness. Two-piece tanks and tanks with separate covers should be designed with tongue and groove or lap joints (figure 4-24). Manway covers should have similar joints. High-quality, preformed joint sealers should be used to achieve a watertight seal. They should be workable over a wide temperature range and should adhere to clean, dry surfaces; they must not shrink, harden, or oxidize. Seals should meet the minimum compression and other requirements prescribed by the seal manufacturer. Pipe and

Figure 4-24. Tongue and groove joint and sealer



Source: Ayres Associates

inspection port joints should have cast-in rubber boots or compression seals.

Septic tanks should be tested for watertightness using hydrostatic or vacuum tests, and manway risers and inspection ports should be included in the test. The professional association representing the materials industry of the type of tank construction (e.g., the National Pre-cast Concrete Association) should be contacted to establish the appropriate testing criteria and procedures. Test criteria for precast concrete are presented in table 4-14.

4.6.3 Construction considerations

Important construction considerations include tank location, bedding and backfilling, watertightness, and flotation prevention, especially with non-concrete tanks. Roof drains, surface water runoff, and other clear water sources must not be routed to the septic tank. Attention to these considerations

Table 4-14. Watertightness testing procedure/criteria for precast concrete tanks

Standard	Hydrostatic test		Vacuum test	
	Preparation	Pass/fail criterion	Preparation	Pass/fail criterion
C 1227, ASTM (1993)	Seal tank, fill with water, and let stand for 24 hours. Refill tank.	Approved if water level is held for 1 hour	Seal tank and apply a vacuum of 2 in. Hg.	Approved if 90% of vacuum is held for 2 minutes.
NPCA (1998)	Seal tank, fill with water, and let stand for 8 to 10 hours. Refill tank and let stand for another 8 to 10 hours.	Approved if no further measurable water level drop occurs	Seal tank and apply a vacuum of 4 in. Hg. Hold vacuum for 5 minutes. Bring vacuum back to 4 in. Hg.	Approved if vacuum can be held for 5 minutes without a loss of vacuum.

will help to ensure that the tank performs as intended.

Location

The tank should be located where it can be accessed easily for septage removal and sited away from drainage swales or depressions where water can collect. Local codes must be consulted regarding minimum horizontal setback distances from buildings, property boundaries, wells, water lines, and the like.

Bedding and backfilling

The tank should rest on a uniform bearing surface. It is good practice to provide a level, granular base for the tank. The underlying soils must be capable of bearing the weight of the tank and its contents. Soils with a high organic content or containing large boulders or massive rock edges are not suitable.

After setting the tank, leveling, and joining the building sewer and effluent line, the tank can be backfilled. The backfill material should be free-flowing and free of stones larger than 3 inches in diameter, debris, ice, or snow. It should be added in lifts and each lift compacted. In fine-textured soils such as silts, silt loams, clay loams, and clay, imported granular material should be used. This is a must where freeze and thaw cycles are common because the soil movement during such cycles can work tank joints open. This is a significant concern when using plastic and fiberglass tanks.

The specific bedding and backfilling requirements vary with the shape and material of the tank. The manufacturer should be consulted for acceptable materials and procedures.

Watertightness

All joints must be sealed properly, including tank joints (sections and covers if not a monolithic tank), inlets, outlets, manways, and risers (ASTM, 1993; NPCA, 1998). The joints should be clean and dry before applying the joint sealer. Only high-quality joint sealers should be used (see previous section). Backfilling should not proceed until the sealant setup period is completed. After all joints have been made and have cured, a watertightness

test should be performed (see table 4-14 for precast concrete tanks). Risers should be tested.

Flotation prevention

If the tank is set where the soil can be saturated, tank flotation may occur, particularly when the tank is empty (e.g., recently pumped dose tanks or septic tank after septage removal). Tank manufacturers should be consulted for appropriate antiflotation devices.

4.6.4 Operation and maintenance

The septic tank is a passive treatment unit that typically requires little operator intervention. Regular inspections, septage pumping, and periodic cleaning of the effluent filter or screen are the only operation and maintenance requirements. Commercially available microbiological and enzyme additives are promoted to reduce sludge and scum accumulations in septic tanks. They are not necessary for the septic tank to function properly when treating domestic wastewaters. Results from studies to evaluate their effectiveness have failed to prove their cost-effectiveness for residential application. For most products, concentrations of suspended solids and BOD in the septic tank effluent increase upon their use, posing a threat to SWIS performance. No additive made up of organic solvents or strong alkali chemicals should be used because they pose a potential threat to soil structure and ground water.

Inspections

Inspections are performed to observe sludge and scum accumulations, structural soundness, watertightness, and condition of the inlet and outlet baffles and screens. *(Warning: In performing inspections or other maintenance, the tank should not be entered. The septic tank is a confined space and entering can be extremely hazardous because of toxic gases and/or insufficient oxygen.)*

Sludge and scum accumulations

As wastewater passes through and is partially treated in the septic tank over the years, the layers of floatable material (scum) and settleable material (sludge) increase in thickness and gradually reduce the amount of space available for clarified waste-

water. If the sludge layer rises to the bottom of the effluent T-pipe, solids can be drawn through the effluent port and transported into the infiltration field, increasing the risk of clogging. Likewise, if the bottom of the thickening scum layer moves lower than the bottom of the effluent T-pipe, oils and other scum material can be drawn into the piping that discharges to the infiltration field. Various devices are commercially available to measure sludge and scum depths. The scum layer should not extend above the top or below the bottom of either the inlet or outlet tees. The top of the sludge layer should be at least 1 foot below the bottom of either tee or baffle. Usually, the sludge depth is greatest below the inlet baffle. The scum layer bottom must not be less than 3 inches above the bottom of the outlet tee or baffle. If any of these conditions are present, there is a risk that wastewater solids will plug the tank inlet or be carried out in the tank effluent and begin to clog the SWIS.

Structural soundness and watertightness

Structural soundness and watertightness are best observed after the septage has been pumped from the tank. The interior tank surfaces should be inspected for deterioration, such as pitting, spalling, delamination, and so forth and for cracks and holes. The presence of roots, for example, indicates tank cracks or open joints. These observations should be made with a mirror and bright light. Watertightness can be checked by observing the liquid level (before pumping), observing all joints for seeping water or roots, and listening for running or dripping water. Before pumping, the liquid level of the tank should be at the outlet invert level. If the liquid level is below the outlet invert, exfiltration is occurring. If it is above, the outlet is obstructed or the SWIS is flooded. A constant trickle from the inlet is an indication that plumbing fixtures in the building are leaking and need to be inspected.

Baffles and screens

The baffles should be observed to confirm that they are in the proper position, secured well to the piping or tank wall, clear of debris, and not cracked or broken. If an effluent screen is fitted to the outlet baffle, it should be removed, cleaned, inspected for irregularities, and replaced. Note that

effluent screens should not be removed until the tank has been pumped or the outlet is first plugged.

Septic tank pumping

Tanks should be pumped when sludge and scum accumulations exceed 30 percent of the tank volume or are encroaching on the inlet and outlet baffle entrances. Periodic pumping of septic tanks is recommended to ensure proper system performance and reduce the risk of hydraulic failure. If systems are not inspected, septic tanks should be pumped every 3 to 5 years depending on the size of the tank, the number of building occupants, and household appliances and habits (see Special Issues Fact Sheets). Commercial systems should be inspected and/or pumped more frequently, typically annually. There is a system available that provides continuous monitoring and data storage of changes in the sludge depth, scum or grease layer thickness, liquid level, and temperature in the tank. Long-term verification studies of this system are under way. Accumulated sludge and scum material stored in the tank should be removed by a certified, licensed, or trained service provider and reused or disposed of in accordance with applicable federal, state, and local codes. (Also see section 4.5.5.)

4.6.5 Septage

Septage is an odoriferous slurry (solids content of only 3 to 10 percent) of organic and inorganic material that typically contains high levels of grit, hair, nutrients, pathogenic microorganisms, oil, and grease (table 4-15). Septage is defined as the entire contents of the septic tank—the scum, the sludge, and the partially clarified liquid that lies between them—and also includes pumpings from aerobic treatment unit tanks, holding tanks, biological (“composting”) toilets, chemical or vault toilets, and other systems that receive domestic wastewaters. Septage is controlled under the federal regulations at 40 CFR Part 503. Publications and other information on compliance with these regulations can be found at <http://www.epa.gov/oia/tips/scws.htm>.

Septage also may harbor potentially toxic levels of metals and organic and inorganic chemicals. The exact composition of septage from a particular treatment system is highly dependent upon the type of facility and the activities and habits of its users.

Table 4-15. Chemical and physical characteristics of domestic septage

Parameter	Concentration (mg/L)	
	Average	Range
Total solids	34,106	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	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	970	522–4,190
Grease	5,600	208–23,368
pH	—	1.5–12.6

Source: USEPA, 1994.

For example, oil and grease levels in septage from food service or processing facilities might be many times higher than oil and grease concentrations in septage from residences (see Special Issues Fact Sheets). Campgrounds that have separate graywater treatment systems for showers will likely have much higher levels of solids in the septage from the blackwater (i.e., toilet waste) treatment system. Septage from portable toilets might have been treated with disinfectants, deodorizers, or other chemicals.

Septage management programs

The primary objective of a septage management program is to establish procedures and rules for handling and disposing of septage in an affordable manner that protects public health and ecological resources. When planning a program it is important to have a thorough knowledge of legal and regulatory requirements regarding handling and disposal. USEPA (1994) has issued regulations and guidance that contain the type of information required for developing, implementing, and maintaining a septage management program. Detailed guidance for identifying, selecting, developing, and operating reuse or disposal sites for septage is provided in *Process Design Manual: Surface Disposal of Sewage Sludge and Domestic Septage* (USEPA,

1995^b), which is on the Internet at <http://www.epa.gov/ORD/WebPubs/sludge.pdf>. Additional information can be found in *Domestic Septage Regulatory Guidance* (USEPA, 1993), at <http://www.epa.gov/oia/tips/scws.htm>.

States and municipalities typically establish public health and environmental protection regulations for septage management (pumping, handling, transport, treatment, and reuse/disposal). Key components of septage management programs include tracking or manifest systems that identify acceptable septage sources, pumpers, transport equipment, final destination, and treatment, as well as procedures for controlling human exposure to septage, including vector control, wet weather runoff, and access to disposal sites.

Septage treatment/disposal: land application

The ultimate fate of septage generally falls into three basic categories—land application, treatment at a wastewater treatment plant, or treatment at a special septage treatment plant. Land application is the most commonly used method for disposing of septage in the United States. Simple and cost-effective, land application approaches use minimal energy and recycle organic material and nutrients back to the land. Topography, soils, drainage patterns, and agricultural crops determine which type of land disposal practice works best for a given situation. Some common alternatives are surface application, subsurface incorporation, and burial. Disposal of portable toilet wastes mixed with disinfectants, deodorizers, or other chemicals at land application sites is not recommended. If possible, these wastes should be delivered to the collection system of a wastewater treatment plant to avoid potential chemical contamination risks at septage land application sites. Treatment plant operators should be consulted so they can determine when and where the septage should be added to the collection system.

When disposing of septage by land application, appropriate buffers and setbacks should be provided between application areas and water resources (e.g., streams, lakes, sinkholes). Other considerations include vegetation type and density, slopes, soils, sensitivity of water resources, climate,

and application rates. Agricultural products from the site must not be directly consumed by humans. Land application practices include the following:

Spreading by hauler truck or farm equipment

In the simplest method, the truck that pumps the septage takes it to a field and spreads it on the soil. Alternatively, the hauler truck can transfer its septage load into a wagon spreader or other specialized spreading equipment or into a holding facility at the site for spreading later.

Spray irrigation

Spray irrigation is an alternative that eliminates the problem of soil compaction by tires. Pretreated septage is pumped at 80 to 100 psi through nozzles and sprayed directly onto the land. This method allows for septage disposal on fields with rough terrain.

Ridge and furrow irrigation

Pretreated septage can be transferred directly into furrows or row crops. The land should be relatively level.

Subsurface incorporation of septage

This alternative to surface application involves placing untreated septage just below the surface. This approach reduces odors and health risks while still fertilizing and conditioning the soil. The method can be applied only on relatively flat land (less than 8 percent slope) in areas where the seasonally high water table is at least 20 inches. Because soil compaction is a concern, no vehicles should be allowed to drive on the field for 1 to 2 weeks after application. Subsurface application practices include the following:

- *Plow and furrow irrigation:* In this simple method, a plow creates a narrow furrow 6 to 8 inches (15 to 20 centimeters) deep. Liquid septage is discharged from a tank into the furrow, and a second plow covers the furrow.
- *Subsurface injection:* A tillage tool is used to create a narrow cavity 4 to 6 inches (10 to 15 centimeters) deep. Liquid septage is injected into the cavity, and the hole is covered.

Codisposal of septage in sanitary landfills

Because of the pollution risks associated with runoff and effluent leaching into ground water, landfill disposal of septage is not usually a viable option. However, some jurisdictions may allow disposal of septage/soil mixtures or permit other special disposal options for dewatered septage (sludge with at least 20 percent solids). Septage or sludge deposited in a landfill should be covered immediately with at least 6 inches of soil to control odors and vector access (USEPA, 1995b). (*Note: Codisposal of sewage sludge or domestic septage at a municipal landfill is considered surface disposal and is regulated under 40 CFR Part 258.*)

Septage treatment/disposal: treatment plants

Disposal of septage at a wastewater treatment plant is often a convenient and cost-effective option. Addition of septage requires special care and handling because by nature septage is more concentrated than the influent wastewater stream at the treatment plant. Therefore, there must be adequate capacity at the plant to handle and perhaps temporarily store delivered septage until it can be fed into the treatment process units. Sites that typically serve as the input point for septage to be treated at a wastewater treatment plant include the following:

Upstream sewer manhole

This alternative is viable for larger sewer systems and treatment plants. Septage is added to the normal influent wastewater flow at a receiving station fitted with an access manhole.

Treatment plant headworks

The septage is added at the treatment plant upstream of the inlet screens and grit chambers. The primary concern associated with this option is the impact of the introduced wastes on treatment unit processes in the plant. A thorough analysis should be conducted to ensure that plant processes can accept and treat the wastes while maintaining appropriate effluent pollutant concentrations and meeting other treatment requirements. In any event, the treatment plant operator should be consulted before disposal.

Sludge-handling process

To reduce loading to the liquid stream, the septage can be sent directly to the sludge-handling process. Like the headworks option, the impact on the sludge treatment processes must be carefully analyzed to ensure that the final product meets treatment and other requirements.

Treatment at a special septage treatment plant

This method of septage disposal is usually employed in areas where land disposal or treatment at a wastewater treatment plant is not a feasible option. There are few of these facilities, which vary from simple lagoons to sophisticated plants that mechanically and/or chemically treat septage. Treatment processes used include lime stabilization, chlorine oxidation, aerobic and anaerobic digestion, composting, and dewatering using pressure or vacuum filtration or centrifugation. This is the most expensive option for septage management and should be considered only as a last resort.

Public outreach and involvement

Developing septage treatment units or land application sites requires an effective public outreach program. Opposition to locating these facilities in the service area is sometimes based about incomplete or inaccurate information, fear of the unknown, and a lack of knowledge on potential impacts. Without an effective community-based program of involvement, even the most reasonable plan can be difficult to implement. Traditional guidance on obtaining public input in the development of disposal or reuse facilities can be found in *Process Design Manual: Surface Disposal of Sewage Sludge and Domestic Septage* (USEPA, 1995b), which is on the Internet at <http://www.epa.gov/ORD/WebPubs/sludge.pdf>.

Additional information can be found in *Domestic Septage Regulatory Guidance* (USEPA, 1993), posted at <http://www.epa.gov/oia/tips/scws.htm>. General guidance on developing and implementing a public outreach strategy is available in *Getting In Step: A Guide to Effective Outreach in Your Watershed*, published by the Council of State Governments (see chapter 2) and available at <http://www.epa.gov/owow/watershed/outreach/documents/>.

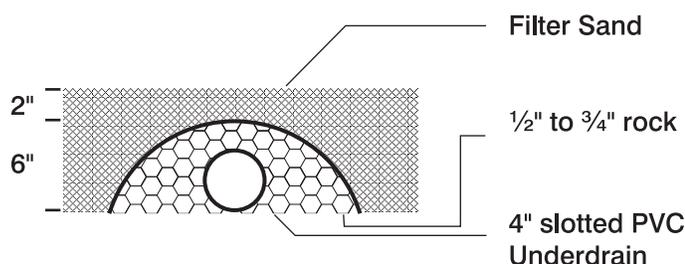
4.7 Sand/media filters

Sand (or other media) filters are used to provide advanced treatment of settled wastewater or septic tank effluent. They consist of a lined (lined with impervious PVC liner on sand bedding) excavation or watertight structure filled with uniformly sized washed sand (the medium) that is normally placed over an underdrain system (figure 4-25). These contained media filters are also known as packed bed filters. The wastewater is dosed onto the surface of the sand through a distribution network and is allowed to percolate through the sand to the underdrain system. The underdrain collects the filtrate for further processing, recycling, or discharging to a SWIS. Some “bottomless” designs directly infiltrate the filtered effluent into the soil below.

4.7.1 Treatment mechanisms and filter design

Sand filters are essentially aerobic, fixed-film bioreactors used to treat septic tank effluent. Other very important treatment mechanisms that occur in sand filters include physical processes such as straining and sedimentation, which remove suspended solids within the pores of the media, and chemical adsorption of dissolved pollutants (e.g., phosphorus) to media surfaces. The latter phenomenon tends to be finite because adsorption sites become saturated with the adsorbed compound, and it is specific to the medium chosen. Bioslimes from the growth of microorganisms develop as attached films on the sand particle surfaces. The microorganisms in the slimes absorb soluble and colloidal waste materials in the wastewater as it percolates around the sand surfaces. The absorbed materials are incorporated into new cell mass or degraded under aerobic conditions to carbon dioxide and water.

Figure 4-25. Underdrain system detail for sand filters



Most of the biochemical treatment occurs within approximately 6 inches (15 centimeters) of the filter surface. As the wastewater percolates through this active layer, carbonaceous BOD and ammonium-nitrogen are removed. Most of the suspended solids are strained out at the filter surface. The BOD is nearly completely removed if the wastewater retention time in the sand media is sufficiently long for the microorganisms to absorb and react with waste constituents. With depleting carbonaceous BOD in the percolating wastewater, nitrifying microorganisms are able to thrive deeper in this active surface layer, where nitrification will readily occur.

To achieve acceptable treatment, the wastewater retention time in the filter must be sufficiently long and reaeration of the media must occur to meet the oxygen demand of the applied wastewater. The pore size distribution and continuity of the filter medium, the dose volume, and the dosing frequency are key design and operating considerations for achieving these conditions. As the effective size and uniformity of the media increases, the reaeration rate increases, but the retention time decreases. Treatment performance might decline if the retention time is too short. If so, it may be necessary to recirculate the wastewater through the filter several times to achieve the desired retention time and concomitant treatment performance. Multiple small dose volumes that do not create a saturated wetting front on the medium can be used to extend residence times. If saturated conditions are avoided, moisture tensions within the medium will remain high, which will redistribute the applied wastewater throughout the medium, enhancing its contact with the bioslimes on the medium. The interval between doses provides time for reaeration of the medium to replenish the oxygen depleted during the previous dose.

Filter surface clogging can occur with finer media in response to excessive organic loadings. Biomass increases can partially fill the pores in the surface layer of the sand. If the organic loadings are too great, the biomass will increase to a point where the surface layer becomes clogged and is unable to accept further wastewater applications. However, if the applied food supply is less than that required by resident microorganisms, the microorganisms are forced into endogenous respiration; that is, they begin to draw on their stored metabolites or

surrounding dead cells for food. If the microorganisms are maintained in this growth phase, net increases of biomass do not occur and clogging can be minimized.

Chemical adsorption can occur throughout the medium bed, but adsorption sites in the medium are usually limited. The capacity of the medium to retain ions depends on the target constituent, the pH, and the mineralogy of the medium. Phosphorus is one element of concern in wastewater that can be removed in this manner, but the number of available adsorption sites is limited by the characteristics of the medium. Higher aluminum, iron, or calcium concentrations can be used to increase the effectiveness of the medium in removing phosphorus. Typical packed bed sand filters are not efficient units for chemical adsorption over an extended period of time. However, use of special media can lengthen the service (phosphorus removal) life of such filters beyond the normal, finite period of effective removal.

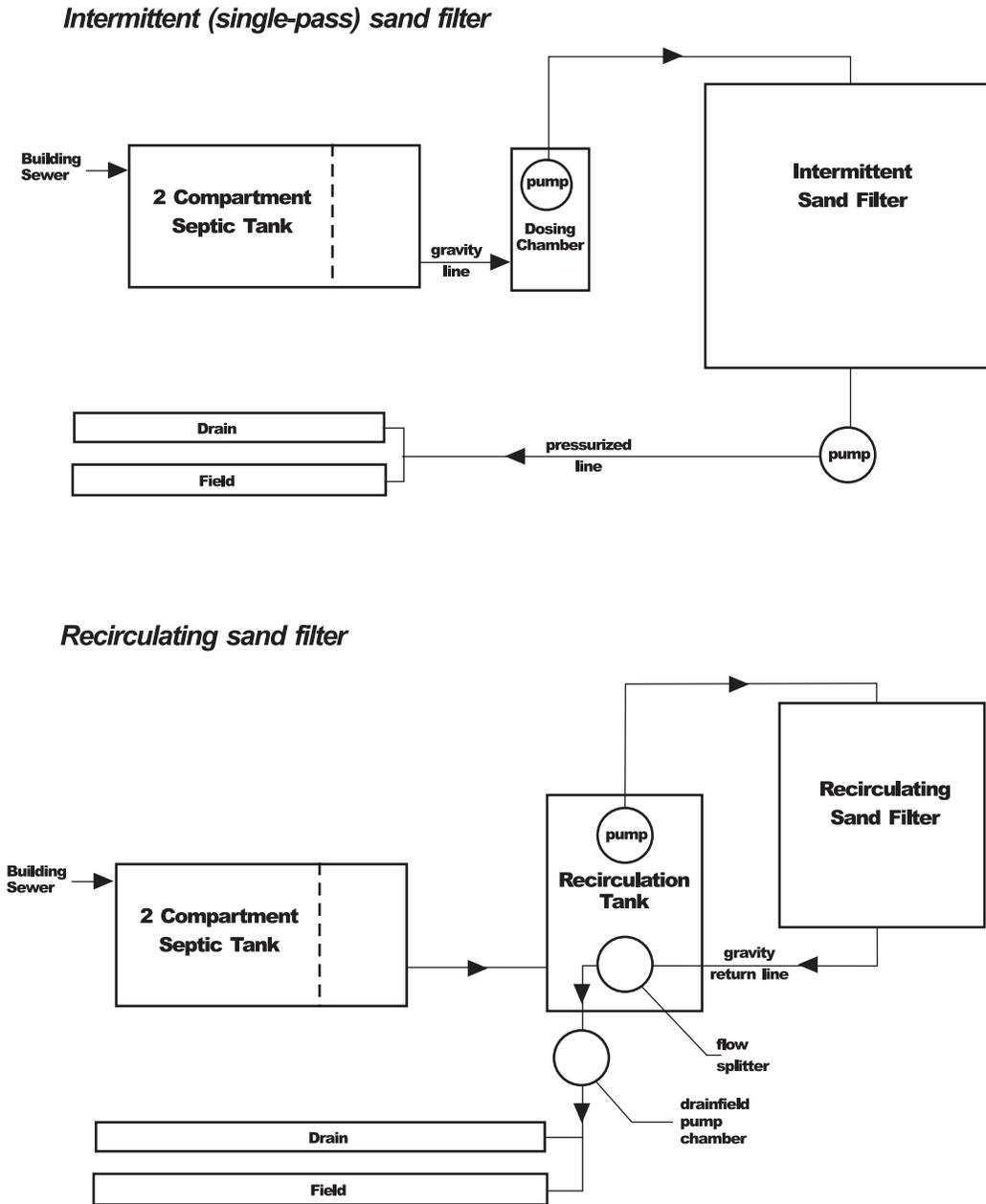
Filter designs

Sand filters are simple in design and relatively passive to operate because the fixed-film process is very stable and few mechanical components are used. Two types of filter designs are common, “single-pass” and “recirculating” (figure 4-26). They are similar in treatment mechanisms and performance, but they operate differently. Single-pass filters, historically called “intermittent” filters, discharge treated septic tank effluent after one pass through the filter medium (see Fact Sheet 10). Recirculating filters collect and recirculate the filtrate through the filter medium several times before discharging it (see Fact Sheet 11). Each has advantages for different applications.

Single-pass filters

The basic components of single-pass filters (see Fact Sheet 10) include a dose tank, pump and controls (or siphon), distribution network, and the filter bed with an underdrain system (figure 4-25). The wastewater is intermittently dosed from the dose tank onto the filter through the distribution network. From there, it percolates through the sand medium to the underdrain and is discharged. On-demand dosing has often been used, but timed dosing is becoming common.

Figure 4-26. Schematics of the two most common types of sand media filters



To create the wastewater retention times necessary for achieving desired treatment results, single-pass filters must use finer media than that typically used in recirculating filters. Finely sized media results in longer residence times and greater contact between the wastewater and the media surfaces and their attached bioslimes. BOD removals of greater than 90 percent and nearly complete ammonia removal are typical (Darby et al., 1996; Emerick et al., 1997;

University of Wisconsin, 1978). Single-pass filters typically achieve greater fecal coliform removals than recirculating filters because of the finer media and the lower hydraulic loading. Daily hydraulic loadings are typically limited to 1 to 2 gpd/ft², depending on sand size, organic loading, and especially the number of doses per day (Darby et al., 1996).

Recirculating filters

The basic components of recirculating filters (see Fact Sheet 11) are a recirculation/dosing tank, pump and controls, a distribution network, a filter bed with an underdrain system, and a return line fitted with a flow-splitting device to return a portion of the filtrate to the recirculation/dosing tank (figure 4-26). The wastewater is dosed to the filter surface on a timed cycle 1 to 3 times per hour. The returned filtrate mixes with fresh septic tank effluent before being returned to the filter.

Media types

Many types of media are used in packed bed filters. Washed, graded sand is the most common medium. Other granular media used include gravel, anthracite, crushed glass, expanded shale, and bottom ash from coal-fired power plants. Bottom ash has been studied successfully by Swanson and Dix (1987). Crushed glass has been studied (Darby et al., 1996; and Emerick et al., 1997), and it was found to perform similarly to sand of similar size and uniformity. Expanded shale appears to have been successful in some field trials in Maryland, but the data are currently incomplete in relation to long-term durability of the medium.

Foam chips, peat, and nonwoven coarse-fiber synthetic textile materials have also been used. These are generally restricted to proprietary units. Probably the most studied of these is the peat filter, which has become fairly common in recent years. Depending on the type of peat used, the early performance of these systems will produce an effluent with

a low pH and a yellowish color. This is accompanied by some excellent removal of organics and microbes, but would generally not be acceptable as a surface discharge (because of low pH and visible color). However, as a pretreatment for a SWIS, low pH and color are not a problem. Peat must meet the same hydraulic requirements as sand (see Fact Sheets 10 and 11). The primary advantage of the proprietary materials, the expanded shale, and to some degree the peat is their light weight, which makes them easy to transport and use at any site. Some short-term studies of nonwoven fabric filters have shown promise (Roy and Dube, 1994). System manufacturers should be contacted for application and design using these materials.

4.7.2 Applications

Sand media filters may be used for a broad range of applications, including single-family residences, large commercial establishments, and small communities. They are frequently used to pretreat wastewater prior to subsurface infiltration on sites where the soil has insufficient unsaturated depth above ground water or bedrock to achieve adequate treatment. They are also used to meet water quality requirements before direct discharge to a surface water. They are used primarily to treat domestic wastewater, but they have been used successfully in treatment trains to treat wastewaters high in organic materials such as those from restaurants and supermarkets. Single pass filters are most frequently used for smaller applications and sites where nitrogen removal is not required. Recirculating filters are used for both large and small flows

Performance of sand and other filters

Twelve innovative treatment technologies were installed to replace failed septic systems in the Narragansett Bay watershed, which is both pathogen- and nitrogen-sensitive. The technologies installed consisted of an at-grade recirculating sand filter, single pass sand filters, Maryland-style recirculating sand filters, foam biofilters, and a recirculating textile filter. The treatment performance of these systems was monitored over an 18-month period. In the field study, TSS and BOD₅ concentrations were typically less than 5 mg/L for all sand filter effluent and less than 20 mg/L for both the foam biofilter and textile filter effluents. Single pass sand filters achieved substantial fecal coliform reductions, reaching mean discharge levels ranging from 200 to 520 colonies per 100 mL for all 31 observations. The at-grade recirculating sand filter achieved the highest total nitrogen reductions of any technology investigated and consistently met the Rhode Island state nitrogen removal standard (a TN reduction of 50 percent or more and a TN concentration of 19 mg/L or less) throughout the study.

Source: Loomis et al., 2001.

and are frequently used where nitrogen removal is necessary. Nitrogen removal of up to 70 to 80 percent can be achieved if an anoxic reactor is used ahead of the recirculation tank, where the nitrified return filtrate can be mixed with the carbon-rich septic tank effluent (Anderson et al., 1998; Boyle et al., 1994; Piluk and Peters, 1994).

4.7.3 Performance

The treatment performance of single-pass and recirculating filters is presented in table 4-16. The medium used was sand or gravel as noted. Recirculating sand filters generally match or outperform single-pass filters in removal of BOD, TSS, and nitrogen. Typical effluent concentrations for domestic wastewater treatment are less than 10 mg/L for both BOD and TSS, and nitrogen removal is approximately 50 percent. Single-pass sand filters can also typically produce an effluent of less than 10 mg/L for both BOD and TSS. Effluent is nearly completely nitrified, but some variability can be expected in nitrogen removal capability. Pell and Nyberg (1989) found typical nitrogen removals of 18 to 33 percent with their intermittent sand filter. Fecal coliform removal is somewhat better in single pass filters. Removals range from 2 to 4 logs in both types of filters. Intermittent sand filter fecal coliform removal is a function of hydraulic loading; removals decrease as the loading rate increases above 1 gpm/ft² (Emerick et al., 1997).

Effluent suspended solids from sand filters are typically low. The medium retains the solids. Most of the organic solids are ultimately digested. Gravel filters, on the other hand, do not retain solids as well.

excessive solids buildup due to the lack of periodic sludge pumping and removal. In such cases, the solids storage capacity of the final settling compartment might be exceeded, which results in the discharge of solids into the effluent. ATU performance and effluent quality can also be negatively affected by the excessive use of toxic household chemicals. ATUs must be properly operated and maintained to ensure acceptable performance.

4.8 Aerobic treatment units

Aerobic treatment units (ATUs) refer to a broad category of pre-engineered wastewater treatment

devices for residential and commercial use. ATUs are designed to oxidize both organic material and ammonium-nitrogen (to nitrate nitrogen), decrease suspended solids concentrations and reduce pathogen concentrations.

A properly designed treatment train that incorporates an ATU and a disinfection process can provide a level of treatment that is equivalent to that level provided by a conventional municipal biological treatment facility. The ATU, however, must be properly designed, installed, operated and maintained.

Although most ATUs are suspended growth devices, some units are designed to include both suspended growth mechanisms combined with fixed-growth elements. A third category of ATU is designed to provide treatment entirely through the use of fixed-growth elements such as trickling filters or rotating biological contactors (refer to sheets 1 through 3). Typical ATU's are designed using the principles developed for municipal-scale wastewater treatment and scaled down for residential or commercial use.

Most ATUs are designed with compressors or aerators to oxygenate and mix the wastewater. Partial pathogen reduction is achieved. Additional disinfection can be achieved through chlorination, UV treatment, ozonation or soil filtration. Increased nutrient removal (denitrification) can be achieved by modifying the treatment process to provide an anaerobic/anoxic step or by adding treatment processes to the treatment train.

4.8.1 Treatment mechanisms

ATUs may be designed as continuous or batch flow systems (refer to fact sheets 1 through 3). The simplest continuous flow units are designed with no flow equalization and depend upon aeration tank volume and/or baffles to reduce the impact of hydraulic surges. Some units are designed with flow-dampening devices, including air lift or float-controlled mechanical pumps to transfer the wastewater from the aeration tank to a clarifier. Other units are designed with multiple-chambered tanks to attenuate flow. The batch (fill and draw) flow system design eliminates the problem of hydraulic variation. Batch systems are designed to collect and treat wastewater over a period of time.

Table 4-16. Single pass and recirculating filter performance.

Reference	BOD			TSS			TKN			TN			Fecal Coliforms			Comments ^{d,e}
	Inf.	Eff.	% Rem.	Inf.	Eff.	% Rem.	Inf.	Eff.	% Rem.	Inf.	Eff.	% Rem.	Inf.	Eff.	% Rem.	
	(mg/L)	(mg/L)		(mg/L)	(mg-N/L)		(mg-N/L)	(mg-N/L)		(mg-N/L)	(#/100mL)		(#/100mL)			
Single Pass Filters																
Cagle & Johnson (1994) ^a California	160	2	98.75	73	16	78.08	61.8	5.9	90.45	61.8	37.4	39.48	1.14E+05	1.11E+02	99.90	Sand media: es=0.25-0.65 mm; uc=3-4. Design hydraulic loadings=1.2 gpd/ft ² based on 150 gpd/bedroom. Actual flows not measured.
Effert, et al. (1985) ^a Ohio	127	4	96.85	53	17	67.92	--	--	--	41.5	37.5	9.64	2.19E+05	1.60E+03	99.27	Sand media: es=0.4 mm, uc=2.5. Average loadings=0.4 gpd/ft ² / 0.42 lb BOD/1000ft ² Doses per day=3.3.
Ronayne, et al. (1982) ^a Oregon	217	3	98.62	146	10	93.15	57.1	1.7	97.02	57.5	30.3	47.30	2.60E+05	4.07E+02	99.84	Sand media: es=0.14-0.30 mm; uc=1.5-4.0. Average loadings=0.33-0.70 gpd/ft ² BOD/1000ft ² -day.
Sievers (1998) ^a Missouri	297	3	98.99	44	3	93.18	37	0.5	98.65	37.1	27.5	25.88	4.56E+05	7.30E+01	99.98	Sand media: not reported. Design hydraulic loading=1 gpd/ft ² . Daily flows not reported.
Recirculating Filters																
Louden, et al. (1985) ^a Michigan	150	6	96.00	42	6	85.71	55	2.3	95.82	55	26	52.73	3.40E+03	1.40E+01	99.59	Sand media: es=0.3 mm, uc=4.0. Average loadings=0.9 gpd/ft ² (forward flow) / 1.13 lb BOD/1000ft ² -day. Recirculation ratio=3:1. Dosed 4-6 times per hour. Open surface, sprinkler
Piluk & Peters (1994) ^a Maryland	235	5	97.87	75	8	89.33	--	--	--	57	20	64.91	1.80E+06	9.20E+03	99.49	Sand media: es=1 mm, uc=<2.5. Design hydraulic loading=3.54 gpd/ft ² (forward flow). Actual flows not measured. Recirculation ratio=3:1. Doses per day=24.
Ronayne, et al. (1982) ^a Oregon	217	3	98.62	146	4	97.26	57.1	1.1	98.07	57.5	31.5	45.22	2.60E+05	8.50E+03	96.73	Sand media: es=1.2 mm, uc=2.0. Maximum hydraulic loading (forward flow)=3.1 gpd/ft ² Recirculation ratio=3:1-4:1. Doses/day=48.
Roy & Dube (1994) ^a Quebec	101	6	94.06	77	3	96.10	37.7	7.9	79.05	37.7	20.1	46.68	4.80E+05	1.30E+04	97.29	Gravel media: es=4.0, uc=<2/5. Design hydraulic loading (forward flow)=23.4 gpd/ft ² ratio=5:1. Doses per day=48. Open surface, winter operation.
Ayres Assoc. (1998a) ^b Wisconsin	601	10	98.34	546	9	98.35	65.9	3	95.45	65.9	16	75.72	>2500	6.20E+01	>98	Gravel media: pea gravel (3/8-in. dia.). Design hydraulic loading=15 gpd/ft ² (forward flow). Recirculation ratio=3:1-5:1. Doses per day=72. Open surface, seasonal operation.
Owen & Bobb (1994) ^c Wisconsin	80	8	90.00	36	6	83.33	--	--	>95	--	--	--	--	--	--	Sand media: es=1.5 mm, uc=4-5. Design hydraulic loading=2.74 gpd/ft ² (forward flow). Recirculation ratio=1:1 to 4:1. Open surface, winter operation.

^a Single-family home filters. ^b Restaurant (grease and oil inf/eff = 119 < 1 mg/L respectively). ^c Small community treating average 15,000 gpd of septic tank effluent. ^d 1 gpd/ft² = 4 cm/day = 0.04m³/m²×day. ^e 1 lb BOD/1000ft²×day = 0.00455 kg/m²×day

Pumps are used to discharge the settled effluent at the end of the cycle (usually one day). Fixed film treatment plants typically are operated as continuous flow systems.

Oxygen is transferred by diffused air, sparged turbine, or surface entrainment devices. When diffused air systems are used, blowers or compressors are used to force the air through diffusers near the bottom of the tank. The sparged turbine is typically designed with a diffused air source and an external mixer, e.g., a submerged flat-bladed turbine. The sparged turbine is more complex than the simple diffused air system. A variety of surface entrainment devices aerate and mix the wastewater. Air is entrained and circulated in the mixed liquor through violent agitation from mixing or pumping.

The separation of process-generated solids by clarification or filtration is a critical design factor for successful ATU performance. Most ATUs are designed to rely on the process of simple gravity separation to remove most of the solids. Some systems include effluent filters within the clarifier to further screen and retain solids in the treatment plant. Gas deflection barriers and scum baffles are a part of some designs and are a simple way to keep floating solids away from the weir area. Properly managed upflow clarifiers can improve separation.

4.8.2 Design Considerations

ATU's are typically rated by hydraulic capacity and organic and solids loadings. ATU daily treatment volumes may range from 400 gpd to a maximum of 1,500 gpd. ATUs typically can be used to treat residential wastewaters with influent concentrations which have 100 mg/L to 300 mg/L total organic compounds and 100 mg/L to 350 mg/L total suspended solids. Design flows are generally set by local sanitary codes for residential and commercial dwellings using methods described in Section 3.3.

ATU's should be equipped with audio and visual alarms to warn of compressor/aerator failure and high water. These alarms alert the owner and/or service provider of service issues that require immediate attention.

ATU's should be constructed of noncorrosive materials, including reinforced plastics and

fiberglass, coated steel, and reinforced concrete. Buried ATU's must be designed to provide easy access to mechanical parts, electrical control systems, and appurtenances requiring maintenance such as weirs, air lift pump lines, etc. ATU's installed above ground should be properly housed to protect against severe climatic conditions. Installation should be in accordance with manufacturers' specifications.

Appurtenances should be constructed of corrosion-free materials including polyethylene plastics. Air diffusers are usually constructed of PVC or ceramic stone. Mechanical components must be either waterproofed and/or protected from the elements. Because blowers, pumps, and other prime movers can be subject to harsh environments and continuous operation, they should be designed for heavy duty use. Proper housing can reduce blower noise.

4.8.3 Applications

ATUs are typically integrated in a treatment train to provide additional treatment before the effluent is discharged to a SWIS. ATU-treatment trains can also be designed to discharge to land and surface waters; ATU discharge is suitable for drip irrigation if high quality effluent is consistently maintained through proper management. Although some jurisdictions allow reductions in vertical separation distances and/or higher soil infiltration rates when ATUs are used, consideration must be given to the potential impacts of higher hydraulic and pollutant loadings. Increased flow through the soil may allow deeper penetration of pathogens and decreased treatment efficiency of other pollutants (see sections 4.4.2 and 4.4.5).

4.8.4 Performance

Managed ATU effluent quality is typically characterized as 25 mg/L or less CBOD₅ and 30 mg/L or less TSS. Fecal coliform counts are typically 3-4 log # / 100 ml (Table 3-19) when the ATUs are operated at or below their design flows and the influent is typical domestic sewage. Effluent nutrient levels are dependent on influent concentrations, climate, and operating conditions.

Other wastewater characteristics may influence performance. Cleaning agents, bleach, caustic

agents, floating matter, and other detritus can plug or damage equipment. Temperature will affect process efficiency, i.e., treatment efficiency generally will improve as the temperature increases.

Owners should be required by local sanitary codes or management program requirements to maintain ongoing service agreements for the life of the system. ATU's should be inspected every three months to help ensure proper operation and treatment effectiveness. Many ATU manufacturers offer a two-year warranty with an optional service agreement after the warranty expires. Inspections generally include visual checks of hoses, wires, leads and contacts, testing of alarms, examination of the mixed liquor, cleaning of filters, removal of detritus, and inspection of the effluent. ATU's should be pumped when the mixed-liquor (aerator) solids are above 6,000 mg/L or the final settler is more than 1/3 full of settled solids.

4.8.5 Risk management

ATU's should be designed to protect the treatment capability of the soil dispersal system and also to sound alarms or send signals to the management entity (owners and/or service providers) when inspection or maintenance is needed. All biological systems are sensitive to temperature, power interruptions, influent variability, and shock loadings of toxic chemicals. Successful operation of ATUs depends on adherence to manufacturers' design and installation requirements and good management that employs meaningful measurements of system performance at sufficiently frequent intervals to ascertain changes in system function. Consistent performance depends on a stable power supply, an intact system as designed, and routine maintenance to ensure that components and appurtenances are in good order. ATU's, like all other onsite wastewater treatment technologies, will fail if they are not designed, installed, or operated properly. Vigilance on the part of owners and service providers is essential to ensure ATUs are operated and maintained to function as designed.

4.8.6 Costs

Installed ATU costs range from \$2500 to \$9000 installed. Pumping may be necessary at any time due to process upsets, or every eight to twelve months, depending on influent quality, temperature and type of process. Pumping could cost from \$100-to-\$300, depending on local requirements. Aerators/compressors last about three to five years and cost from \$300 to \$500 to replace.

Many communities require service contracts. These contracts typically range in cost between \$100 and \$400 per year, depending on the options and features the owners choose. The high end includes pumping costs. Power requirements are generally quoted at around \$200/year.

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