Robert L. Siegrist

Decentralized Water Reclamation Engineering

A Curriculum Workbook

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Preface

About the Topic

Water and sanitation can underpin a healthy society when solutions are effective in protecting public health and preserving environmental quality while being affordable, socially acceptable, and sustainable. In the United States, water and sanitation infrastructure evolved during the 20th century in response to a growing recognition that providing safe drinking water and adequate treatment of wastewaters were needed to protect public health and preserve water quality. During this evolution, there was always a mix of onsite systems serving individual homes and businesses in rural and peri-urban areas, decentralized systems serving suburban residential and mixed-use developments, and larger centralized systems serving densely populated urban areas. However, the relative proportion of the population and development served by different types of infrastructure has varied and evolved over time.

During much of the 20th century, some viewed onsite and decentralized wastewater systems as a means of providing temporary service until sewers and a centralized treatment plant became available to provide permanent service. Early versions of onsite systems (e.g., pit privy and cesspool) were often designed with simple and short-term goals of human waste disposal to prevent human exposure to infectious waste materials and to achieve basic public health and environmental protection. As water-using fixtures and appliances became commonplace, system designs evolved to include raw wastewater treatment through solids separation and anaerobic digestion in a tank-based unit (e.g., a septic tank) followed by effluent disposal to the land (e.g., a soil drainfield). Continuing to evolve, onsite and decentralized systems were increasingly designed and implemented to achieve wastewater treatment as well as disposal and even considered for beneficial water reuse. But many designers and regulatory officials continued to view onsite and decentralized systems as inherently deficient compared to centralized systems. As a result, during the latter half of the 20th century, there were major Federal and State programs that provided funding for construction of wastewater collection systems and centralized treatment plants. But the push to expand service areas and increase the percentage of the population connected to centralized wastewater systems eventually faded for a number of reasons. The construction grants program that provided funding for centralization ended plus there was a growing realization that large centralized systems were not appropriate for all rural and many suburban and peri-urban areas and there were growing concerns about the sustainability of large infrastructure. By the end of the 20th century, about 25 % of the US population was served by onsite and decentralized wastewater systems and approximately one-third of new development was being supported by such systems. This amounted to roughly 25 million existing systems with about 200,000 new systems being installed each year.

Near the end of the 20th century and into the 21st century, a series of activities and events in the United States helped catalyze a reevaluation of how water and wastewater infrastructure could be made more sustainable. There was growing interest in how onsite and decentralized systems could help provide more sustainable infrastructure by:

- Reducing the use of drinking water to flush toilets and transport waste to remote wastewater treatment plants.
- Preventing pollutant discharges from large centralized systems including sanitary sewer overflows, combined sewer overflows, and leaking sewers.
- Recharging water near the point of water extraction and avoiding water export and depletion of local water resources.
- Enabling recovery and reuse of wastewater resources including water, organic matter, nutrients, and energy.
- Lowering consumption of energy and chemicals and reducing greenhouse gas emissions.
- Providing infrastructure that is more robust and resilient to natural disasters and climate change.

During this period, there was also a growing recognition that the capabilities of 21st century onsite and decentralized systems should not be judged based on the performance of older 20th century systems. The early versions of onsite and decentralized systems (e.g., cesspools, seepage pits, leachfields, and septic systems) were designed to be simple and cheap but not to achieve long-term treatment or reuse goals. During the latter decades of the 20th century, increased water use and wastewater generation and more widespread use of disposal-based systems in a growing suburban America led to hydraulic malfunctions, groundwater contamination, and surface water quality deterioration. As a result, these older disposal-based systems became known as “legacy systems.”

Based on major research and development efforts over the past two decades or more, 21st century onsite and decentralized systems (hereafter referred to as decentralized systems) have evolved and modern systems can include a growing array of approaches, devices, and technologies that can achieve wastewater treatment and enable resource conservation and reuse. Ultraefficient fixtures and source separation plumbing can minimize water and energy demands, enable resource recovery and reuse, and reduce wastewater flows and loadings. Wastewater treatment can be achieved using engineered reactor-based unit operations (e.g., aerobic bioreactors, porous media biofilters, and membrane bioreactors) or engineered natural system unit operations (e.g., constructed wetlands, subsurface soil infiltration, and landscape dispersal). Nutrient reduction strategies and technologies can remove and, in some cases, recover nitrogen and phosphorus. Reuse of reclaimed water can occur through garden and landscape irrigation, toilet flushing, and other functions. Sensors and monitoring devices can be used to verify performance and enable remote monitoring and process control to correct a system malfunction.

Today, decentralized systems involving wastewater treatment and water reclamation can be used to serve buildings and developments with design flows of less than 100 to 100,000 gal/day or more. Common and emerging applications within the United States and similar industrialized countries include approaches, technologies, and systems that are deployed for one or more of the following purposes:

- To provide effective wastewater treatment for homes and businesses in rural and peri-urban areas and residential, commercial, and mixed-use developments in suburban areas.
- To provide effective wastewater treatment for buildings and developments while also producing reclaimed water for nonpotable reuse purposes such as toilet flushing, cooling, or irrigation.
- To recover valuable wastewater resources including nutrients, organic matter, and energy.
- To earn points for a green building or sustainability rating through the low impact water and wastewater management options enabled by decentralized systems.
- To provide appropriate treatment and recovery of stormwater runoff in suburban and urbanized areas.

Decentralized systems are also critical to providing safe drinking water and adequate sanitation in developing regions of the world. In developing regions worldwide, concerns about sustainability of large water and wastewater infrastructure are not yet paramount. Rather, concerns are still focused on how best to provide solutions for safe drinking water and effective sanitation—solutions that are effective, affordable, and socially acceptable.

For nearly a generation now, the virtues and varied benefits of decentralized systems have been increasingly recognized and approaches, technologies, and systems have been advocated as critical components for a 21st century water infrastructure in the United States and worldwide. Translating this recognition and advocacy into meaningful impacts requires a portfolio of education and training activities that target different audiences to achieve different outcomes.

About This Workbook

Decentralized Water Reclamation Engineering—A Curriculum Workbook was developed to present technical information and materials concerning the engineering of decentralized infrastructure for wastewater treatment and water reclamation in a form suitable for classroom lectures or self-study. The approaches, technologies, and systems are targeted for sustainable infrastructure across the United States and similar industrialized nations, but they are also applicable in developing regions around the world.

The intended audience for this Workbook includes educators and students engaged in curriculum concerned with water and sanitation and the scientists and engineers seeking to improve the state of the art and standard of practice. This Workbook should also be highly informative for design professionals, contractors, technology developers, regulators, policy makers, and others involved in, or just interested in, the subject of sustainable infrastructure for water and sanitation.

The subject of decentralized water reclamation engineering spans a wealth of approaches, technologies, and systems too numerous to properly cover in a single curriculum workbook. This Workbook was intentionally crafted to provide in-depth information about a selected number of key topics. The presentation is intentionally concise so the information can be efficiently conveyed through course lectures or self-study. The intended outcome is for the reader to increase their understanding and know-how such that they are able to complete an engineering design of a decentralized system for a particular project. Topics covered in this Workbook include:

- Introduction to decentralized infrastructure for wastewater treatment and water reclamation and reuse (Chap. 1).
- Selection, design, and implementation of decentralized systems to satisfy project goals and requirements including sustainability (Chap. 2).
- Characteristics of contemporary water use and wastewater generation and methods to predict flow and composition data for use in design (Chap. 3).

- Water use efficiency and source separation as a means to reduce water use, energy consumption, and greenhouse gas emissions and enhance treatment and enable resource recovery (Chap. 4).
- Alternative methods of wastewater collection and conveyance that are well suited to decentralized system applications (Chap. 5).
- Tank-based treatment operations including septic tanks, aerobic treatment units, porous media biofilters, and membrane bioreactors that can be used to produce primary to tertiary quality effluents (Chaps. 6–9).
- Wetland-based treatment operations including free water surface and vegetated subsurface bed constructed wetlands that can be used to produce advanced secondary quality effluents (Chap. 10).
- Land-based treatment operations involving subsurface soil infiltration that can be used to treat wastewater and assimilate the reclaimed water into a local hydrologic system (Chap. 11).
- Land-based treatment operations involving landscape drip dispersal that can be used to treat wastewater and, in many cases, beneficially recover the water and nutrients for their fertilizer value (Chap. 12).
- Approaches and technologies that can be used as needed to achieve nutrient reduction (and resource recovery in some cases) and pathogen destruction to enable a particular discharge or reuse plan (Chaps. 13 and 14).
- Management requirements and methods for process solids, sludges, and residuals that are generated during decentralized wastewater treatment and water reclamation (Chap. 15).

The Workbook contains 15 chapters, each of which comprises a summary section and a conceptual and technical details section. The summary section presents the scope and key concepts of the chapter topic and provides definitions of terminology and acronyms abbreviations and symbols and a list of references. There are also short-answer questions and calculation problems relevant to the topic of the chapter. The conceptual and technical details section is presented in a slide format that was developed for teaching and then embellished and expanded to provide detailed coverage of a topic. The slides section of each treatment technology chapter (Chaps. 6–14) is divided into major parts that consist of a technology description, treatment performance, principles and processes, design and implementation, summary, and example problems. The Workbook contains over 300 figures and illustrations of technologies and systems and over 150 tables of design and performance data. There are also more than 200 questions and problems relevant to the topics covered including more than 50 example problems that have solutions to illustrate decentralized system assessment and design.

The author developed and refined the contents of the Workbook over the past decade to support delivery of a 15-week long course focused on engineering design for decentralized water reclamation and reuse. This university level course was developed for education of upper level undergraduate and graduate students at the Colorado School of Mines in Golden, Colorado, in the United States. The contents of the Workbook have also been used for delivery of seminars, guest lectures, and professional development workshops.

Boulder, CO Robert L. Siegrist

Acknowledgments

The author is grateful to many individuals and organizations who have served as colleagues, students, research sponsors, collaborators, and supporters over a 40-year career, most of which has been focused on education, research, and practice in areas of decentralized wastewater treatment and water reclamation and reuse. A number of these individuals and organizations are represented by those listed below who reviewed and commented on the contents of a draft version of this Workbook during summer and fall 2015. Each chapter in this Workbook was reviewed by three to six of those listed below based on their expertise in the subject area covered. Several individuals reviewed the entire Workbook. The Workbook was finalized based on the general and specific review comments provided by individuals representing academia, design professionals, and technology companies. The author gratefully acknowledges and appreciates the review efforts of those listed below—their contributions have greatly improved the content and quality of the first edition of this Workbook. The author also acknowledges the valuable contributions of numerous individuals and organizations that shared photographs of technologies and systems and field installations.[1]

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[1] *Note*: The inclusion of a specific company or technology in the Workbook does not necessarily reflect a general positive endorsement and conversely, the lack of inclusion of a specific company or technology should not be interpreted as a negative endorsement.

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About the Author

Robert L. Siegrist is University Professor Emeritus of Environmental Science and Engineering at the Colorado School of Mines where he is also a Research Professor of Civil and Environmental Engineering. He earned his B.S. (with High Honors) and M.S. degrees in Civil Engineering and his Ph.D. degree in Environmental Engineering at the University of Wisconsin. While at the University of Wisconsin, Dr. Siegrist was a graduate student and then research staff member with the Small Scale Waste Management Project, a large multidisciplinary research program focused on the development and application of small systems for water and wastewater treatment and disposal/reuse. Before joining the Colorado School of Mines in 1995, Dr. Siegrist held academic and research positions with the University of Wisconsin, Norwegian Institute for Georesources and Pollution Research, and Oak Ridge National Laboratory. Dr. Siegrist was also a founding partner and senior engineer with RSE, Inc., an environmental consulting company that specialized in the engineering of decentralized systems for water and sanitation.

Dr. Siegrist is an internationally recognized expert in decentralized systems for wastewater treatment and water reclamation as well as in situ remediation of contaminated soil and groundwater. He has published 300 technical papers and authored two reference books. Dr. Siegrist has given invited lectures at more than 100 workshops and conferences in more than 30 countries worldwide. He has served as a science and engineering advisor to the U.S. Environmental Protection Agency, the Department of Energy, the Department of Defense, the National Research Council, and the Government Accountability Office and was appointed a Fellow with the North Atlantic Treaty Organization Committee for Challenges to Modern Society.

At the Colorado School of Mines, Dr. Siegrist served as Director of the Environmental Science and Engineering Division from 2001 to 2010. He was also Program Director for the Small Flows Program, which he established in 1998 to advance the science and engineering of sustainable decentralized approaches to water and sanitation. The Program was designed to enhance the quantitative understanding of processes important to system design and performance and to develop decision-support tools for applications involving individual houses and buildings up to those involving large developments, communities, and watershed-scale situations. Research and educational activities were carried out by a team of faculty, staff, and students from several departments in collaboration with other institutions in the United States and abroad. To support research and teaching, a unique field test facility known as the Mines Park Test Site was established on the university campus. Dr. Siegrist developed an undergraduate/graduate course covering the principles and practices of decentralized water reclamation and reuse.

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Chapter 1

Introduction to Decentralized Infrastructure for Wastewater Treatment and Water Reclamation

1-1. Scope

This chapter highlights the development of wastewater infrastructure in the United States and describes how and why decentralized infrastructure has evolved to become a critical component of a 21st century infrastructure. Decentralized infrastructure consists of approaches, technologies, and systems that can be used at buildings and developments with indoor water use and wastewater flows that span from less than 100 to 100,000 gal/day or more. Several examples are provided to illustrate the characteristics and applications of decentralized approaches, technologies and systems that can be used to achieve effective treatment and disposal of wastewaters, provide a source of reclaimed water, and/or to minimize resource consumption and enable resource recovery.

1-2. Key Concepts

- Water and wastewater infrastructure are inextricably linked through the actions of humans and are crucial for a healthy society with a high standard of living.
  - Modern solutions for water and wastewater infrastructure need to be effective while being affordable, socially acceptable, and sustainable.
  - Modern solutions also need to recognize two distinct perspectives concerning wastewater.
    - Wastewater has long been recognized for the risks it poses. Wastewater can pose inherent risks to human health or the environment due to its chemical and microbial constituents. Fundamentally, the challenge is to assess the magnitude of the risks in a

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given situation and decide on the most appropriate way to manage those risks.

- ○ Wastewater is increasingly being recognized for the resources it contains. Wastewater represents a resource by virtue of the water, organic matter, nutrients, and energy it contains. The challenge is to select, design and implement approaches, technologies and systems that can recover resources of value in a given situation while also mitigating risks to human health and environmental quality.

- ■ In the United States during the 20th century, major investments were made leading to knowledge, laws, and regulations, modernized fixtures and appliances and plumbing systems, and construction of water and wastewater infrastructure including new and improved conveyance and treatment systems and expansion of service areas and increased accessibility.

- ■ At the close of the 20th century, most of the U.S. population had acceptable and affordable access to safe drinking water and adequate wastewater management.
  - • Approximately 75 % of the nation's population was served by larger centralized infrastructure with 25 % served by smaller decentralized infrastructure. Basic features of this 20th century infrastructure can be described as follows:
    - ○ Centralized infrastructure—Extensive collection system piping for long-distance transport of wastewaters for remote treatment at energy consuming, mechanical plants with discharge of treated effluents to surface waters. Engineered plants can have high operation and maintenance requirements but they can yield a high capacity per unit of land area, which is often needed in densely populated areas.
    - ○ Decentralized infrastructure (including onsite systems)—Local treatment at, or very near, the building(s) where wastewater is generated. Treatment using lower energy, reactor-based or landscape-based systems is common with discharge of treated effluents to the land or surface waters. Treatment systems can have low to high operation and maintenance requirements while providing a low to high capacity per unit of land area, which can yield compatibility for areas with low to high population densities. Recovery of wastewater resources such as water, organic matter, nutrients and energy can also be enabled using decentralized infrastructure.

- During the latter part of the 20th century, major Federal and State programs in the United States provided funding for construction of new and expanded centralized infrastructure including wastewater collection systems and treatment and disposal facilities.
  - This was done to improve the quality of life in urbanized areas—often located near rivers, lakes, and coastal zones—where population densities were high and the risks associated with wastewater were also high.
  - During this time there was little funding available for construction of decentralized systems serving homes and businesses. This could be attributed to the fact that the wastewater-related risks were lower due to low population densities and locations in rural areas. In addition some viewed decentralized systems as temporary and only needed until they were replaced by a centralized system.
- During much of the 20th century, decentralized wastewater systems were used in rural areas and other areas with low population densities. Many of these systems were not designed or implemented to achieve explicit treatment and reuse objectives over long-term permanent use.
  - Not surprisingly, such systems often suffered performance deficiencies ranging from hydraulic failures to localized contamination of groundwaters and surface waters. These were attributed to varied causes including poor system siting, improper design, faulty installation, and/or inadequate operation and maintenance.
  - Research and educational initiatives, along with changes in regulatory requirements and advancements in management and performance assurance, helped to improve the standard-of-practice and mitigate performance deficiencies.
- During the latter part of the 20th century, growth in centralized infrastructure for wastewater management eventually leveled off.
  - By the end of the 1970s many urbanized areas of the United States had new and expanded centralized infrastructure for wastewater management and it was increasingly clear that larger centralized systems were not technically feasible or affordable to serve lower density populations located in most rural areas and many small towns.
  - During the 1990s concerns grew about the sustainability of large centralized infrastructure due to:
    - Wasteful use of clean drinking water (up to 20 % lost during delivery plus about 30 % used for flushing toilets and waste carriage),
    - Public health and ecosystem impacts due to sewer overflows and treatment plant failures,

  - High energy use and material and chemical requirements, and
  - Barriers to recycling caused by co-mingling of domestic and industrial wastes.

- Near the end of the 20th century and into the twenty-first, a series of activities and events helped promote the development and deployment of decentralized approaches, technologies, and systems for wastewater treatment and water reclamation and resource recovery. As the United States entered the 21st century, there was growing interest in modern decentralized infrastructure due to the potential benefits it might offer such as:
  - Avoiding large capital costs and reducing operation and maintenance costs.
  - Reducing the use of drinking water to flush toilets and transport waste to remote wastewater treatment plants.
  - Preventing pollutant discharges from large centralized systems by reducing or eliminating sanitary sewer overflows (SSOs), combined sewer overflows (CSOs), and leakage from conventional gravity sewers.
  - Preserving water in a watershed by eliminating inflow and infiltration into sewers and protecting water quality by eliminating leaking sewers.
  - Recharging water near the point of water extraction and avoiding water export and depletion of local water resources (e.g., declining groundwater levels or stream flows).
  - Enabling recovery and reuse of water, organic matter, and nutrients (N, P, K) in domestic wastewaters.
  - Lowering consumption of energy and chemicals, and reducing release of greenhouse gases through the use of water efficient fixtures and appliances and natural treatment system technologies.

- Today, decentralized infrastructure involving wastewater treatment and water reclamation can be applied under different circumstances to achieve different goals. Common and emerging applications within the United States and similar industrialized countries include approaches, technologies and systems that are deployed at buildings and developments located in rural, peri-urban, suburban and even urban areas for one or more of the following purposes:
  - To provide effective wastewater treatment and disposal.
  - To provide effective wastewater treatment and produce a reclaimed water for nonpotable reuse purposes such as toilet flushing, cooling, or irrigation.
  - To recover valuable wastewater resources including nutrients, organic matter, and energy.

- To earn points for a green building or sustainability rating through the low impact water and wastewater management options enabled by decentralized systems.
- To provide appropriate treatment and recovery of stormwater runoff in suburban and urbanized areas.

Decentralized systems are also critical to providing safe drinking water and adequate sanitation in developing regions of the world.

- Modern decentralized infrastructure encompasses approaches, technologies and systems that include:
  - Ultra high water use efficiency fixtures and appliances and in-building waste stream source separation.
  - Small diameter sewers for wastewater collection and conveyance networks.
  - Reactor-based and landscape-based treatment unit operations.
  - System monitoring and performance assurance methods.
- Selection and design of decentralized infrastructure for a particular application can now benefit from modern decision aids and mathematical models.
- Management of decentralized infrastructure is critical to achieving and sustaining a performance outcome.
  - Management involves public and/or private entities and a set of activities, often organized within a jurisdiction, to ensure:
    - Decentralized systems are properly considered during infrastructure and land use planning.
    - If selected they are properly designed, constructed, and operated so the desired performance capabilities are sustainably achieved.
  - As decentralized systems have evolved to become a permanent part of the wastewater infrastructure in the United States the need for, and critical role of, approaches for effective management have also evolved.
- In summary, modern decentralized infrastructure can help support a more sustainable 21st century water and wastewater infrastructure in the United States and other industrialized nations and also aid water and sanitation improvements in the developing world.

1-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 1 are presented in the Slides section.

1-4. Terminology

Terminology introduced and used in Chap. 1 is defined below.

Cluster—Term that refers to combining the wastewater flows from more than one building (e.g., multiple houses or several businesses) using a collection system so the combined flow can be treated for a chosen discharge or water reuse option.

Cluster system—A term used to describe decentralized infrastructure that is used to serve a group of buildings or other sources. A cluster system is often comprised of an alternative sewer system connected to a decentralized treatment system for effluent discharge or water reuse.

Combined sewer overflows (**CSO**)—Discharge of untreated wastewater combined with stormwater to a surface water or land surface. CSOs typically can occur during storm events when hydraulic overloads occur in collection systems or treatment plants that handle wastewater plus stormwater.

Constituent of concern (**COC**)—Constituents of concern include dissolved and suspended inorganic and organic substances and biological organisms that can cause undesirable human health effects or degraded environmental conditions under a given water reclamation plan for discharge or reuse.

Decentralized water reclamation—Wastewater treatment and discharge or reuse occurs on the same or nearby property close to the location(s) where the source(s) of wastewater generation is (are) located.

Disinfection—Refers to the process of destroying pathogenic microorganisms in a media like water so that the risk of infectious disease transmission through human contact with that media is reduced. Example processes include chlorination, ultraviolet light irradiation, and membrane filtration.

Effluent—The liquid that is discharged from a treatment unit. For example, the effluent from a biofilter is the filtrate that is discharged (not recycled) and transported to a next treatment unit or discharged to the environment. Effluent can become the influent to another treatment unit operation. For example, in the context of landscape drip dispersal (LDU) effluent is produced by an upstream treatment unit (e.g., aerobic unit) and becomes the influent to the LDU.

Human Development Index (**HDI**)—A statistical tool developed by the United Nations used to measure a country's overall achievement in its social and economic dimensions. The social and economic dimensions of a country are based on the health of people, their level of education attainment and their standard of living.

Impaired water—Refers to water that has been used or impacted in a manner as to have quality characteristics that make it unsuited for one or more uses. Examples of impaired waters include: residential and commercial wastewater, municipal wastewater, graywater, stormwater, acid mine drainage, etc.

Infrastructure—The basic physical and organizational structures and facilities needed for a given function such as water treatment and supply or wastewater treatment and discharge or water reuse.

Nonpotable—Water that has a quality that makes it unsafe for use as a source of safe drinking water but suitable for other purposes such as toilet flushing or landscape irrigation.

Onsite water reclamation—In the context of decentralized infrastructure, onsite refers to wastewater treatment and discharge or reuse that occurs on the same property as the source of the wastewater generation (e.g., house, business, institution).

Peri-urban—A term that is used to refer to a residential area or mixed-use area that exists between suburban areas and the countryside. See Suburban.

Potable—Water that has a quality that makes it safe to use as a source of safe drinking water.

Primary treatment—A term used to encompass processes and unit operations that remove suspended solids (organic and inorganic) from wastewater by sedimentation or flotation processes. Advanced primary treatment includes some treatment of the separated solids (e.g., by anaerobic biodegradation of settled organic solids). Examples of primary treatment operations include settling basins, septic tanks, and upflow anaerobic sludge blanket reactors.

Reclaimed water—Reclaimed water is wastewater that has been treated to remove inorganic and organic substances and pathogenic microorganisms to a degree that the effluent can be considered reclaimed water with a quality that is fit for the purpose (i.e., appropriate for and of a necessary standard) of an intended discharge or water reuse plan.

Sanitation—A term that refers to the processes, systems and services used to prevent human contact with the hazards of wastes and wastewaters and provide for effective treatment and proper disposal of wastewater. According to the World Health Organization, inadequate sanitation is a major cause of disease worldwide and improving sanitation is known to have a significant beneficial impact on health both in households and across communities.

Secondary treatment—A term used to encompass processes and unit operations that follow primary treatment and are designed to remove biodegradable dissolved and colloidal organic matter by aerobic biological processes. Examples of secondary treatment operations include extended aeration bioreactors, porous media biofilters, and constructed wetlands.

Suburban—A term that is used to refer to a residential area or mixed use area that is geographically separated from a city or highly urbanized area but within commuting distance of it. Peri-urban is another term that is used to refer to residential or mixed-use development between suburban areas and the countryside.

Tertiary treatment (Advanced treatment)—A term used to encompass processes and unit operations that typically follow secondary treatment and are designed to remove specific constituents such as nutrients, trace organic compounds, heavy metals or dissolved salts. Examples of tertiary treatment operations include: denitrifying porous media biofilters, adsorptive media packed bed reactors, and ion exchange columns.

Treatment train—Within a decentralized system a treatment train consists of a sequence of compatible unit operations that connect the source to an intended discharge or reuse option.

Unit operation—A physical facility (e.g., basin, column, reactor, landscape) in which a physical, chemical, and/or biological process is made to occur for the purpose of removing or destroying constituents of potential concern in wastewater or other impaired waters.

Wastewater—Wastewater consists of water plus materials added during water use. The types and concentrations of materials depend on the characteristics of the source (e.g., house, restaurant, school, veterinary clinic). Materials can include human excreta, foodstuffs, consumer products, pharmaceuticals and personal care products, heavy metals, silt, etc.

Wastewater management—A set of elements and activities that can encompass wastewater generation, collection and conveyance, treatment, effluent discharge and recovery of resources (e.g., water, organic matter, nutrients, energy).

Water reclamation (wastewater treatment and discharge or reuse)—A modern term that refers to treatment of wastewaters or other impaired waters to improve the water quality by removing inorganic and organic substances and pathogenic microorganisms to the extent needed to permit safe release of the treated effluent (reclaimed water) to the natural or built environment by a chosen discharge or water reuse option.

Water reuse—Use of reclaimed water for an intended beneficial purpose. Nonpotable water reuse includes landscape irrigation, ornamental uses, and toilet flushing. Potable water reuse includes using reclaimed water to augment sources of drinking water supplies (indirect potable reuse) or direct delivery into a drinking water supply (direct potable reuse).

1-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 1 are listed below.

BOD	Biochemical oxygen demand
CGP	Construction Grants Program in the U.S.
CIDWT	Consortium of Institutions for Decentralized Wastewater Treatment

COC	Constituent of concern
CSO	Combined sewer overflows
DWRC	Decentralized Water Resources Collaborative
ERC	Engineering Research Center (NSF)
FAQ	Frequently asked questions
FOG	Fats, oils, and greases
HDI	Human Development Index
K	Potassium
LEED	Leadership in energy and environmental design
N	Nitrogen
NDWRCDP	National Decentralized Water Resources Capacity Development Project
NSF	National Science Foundation (U.S.)
P	Phosphorus
ReNUWIt	Reinventing the Nation's Urban Water Infrastructure
SSO	Sanitary sewer overflows
STUMOD	Soil treatment unit model
TSS	Total suspended solids
UN	United Nations
U.S.	United States of America
USEPA	U.S. Environmental Protection Agency
UV	Ultraviolet light
WARMF	Watershed analysis risk management framework model
WERF	Water Environment Research Foundation

1-6. Problems

1.1. Water supply and wastewater management are inextricably linked by the actions of humans. Give an example of an adverse outcome to: (1) human health and another to (2) water quality if wastewater is not properly treated and disposed of or reused.

1.2. In the United States and other developed countries, major achievements were made in the 20th century to protect public health and preserve environmental quality by establishing regulations and investing in new and expanded infrastructure to provide safe drinking water, properly treat and dispose of wastewaters, and maintain clean water resources. In many underdeveloped countries in Asia, Africa, and elsewhere this is not the case yet. Briefly explain why this is still true.

1.3. During the 20th century early versions of onsite wastewater systems in the United States were installed at homes and businesses as cesspools, seepage pits, and leachfields. What is meant by the term legacy systems in the context of decentralized wastewater treatment? Why is it important to differentiate the performance of 20th century legacy systems from 21st century modern systems?

1.4. As the United States entered the 21st century, there was growing interest to use decentralized systems to help provide more sustainable solutions by achieving several goals. Complete the following two phrases, which represent two major goals: reducing the use of drinking water for: __________ and preventing pollutant discharges from large centralized systems that result from: __________.

1.5. As of the early 21st century, what fraction of the U.S. population relies on decentralized infrastructure for wastewater treatment and disposal or reuse?

1.6. What is the difference between onsite and decentralized in the context of decentralized infrastructure for wastewater treatment and water reclamation?

1.7. In the 21st century, water and wastewater infrastructure is increasingly driven by sustainability concerns. Approaches and technologies are increasing judged by which of the following (check all that apply): human and environmental effects, resilience to natural or human influenced upsets, ability to deal with climate change?

1.8. Modern decentralized systems can be designed and implemented as long-term solutions for sustainable infrastructure that can achieve which of the following goals (select all that apply): (1) provide effective wastewater treatment and safe discharge, (2) provide treatment of wastewater to provide a reclaimed water source, (3) minimize resource consumption and enable recovery?

1.9. Give an example of a decentralized approach, technology or system you might implement in a high-rise condominium in an urban setting to help achieve water reuse for toilet flushing and turf irrigation.

1.10. Describe how you might use a decentralized approach, technology or system in an office building in a city center like Denver, Colorado to reduce the building demand for water, wastewater, and energy services.

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Consortium of Institutes for Decentralized Wastewater Treatment (CIDWT). Consortium of more than 20 universities and organizations with curriculum development and research efforts. http://www.onsiteconsortium.org/

Decentralized Water Resources Collaborative (DWRC). Collaborative research effort (evolved out of the NDWRCDP) concerning decentralized systems for water, wastewater, storm water. http://www.decentralizedwater.org

European Commission Water Initiative (ECWI). Safe water and sanitation—cost-effective approaches that work. http://ec.europa.eu/research/water-initiative/safewater_en.html

European Union Water Initiative (EUWI). Advancing water and sanitation through strategic partnerships in specific regions of the world. http://www.euwi.net/

International Water Association (IWA). International professional society with a specialty group focus on small water and wastewater systems. http://www.iwahq.org/templates/ld_templates/layout_633184.aspx?ObjectId=633932

National Association of Home Builders Research Center (NAHB). NAHB Green Home Building Guidelines, including water and resource efficiency approaches. http://www.nahbrc.com/technical/index.aspx

National Decentralized Water Resources Capacity Development (NDWRCDP). National research and education program to overcome barriers to use of onsite and decentralized systems. http://www.ndwrcdp.org/

National Environmental Health Association (NEHA). National professional organization with projects, conferences, and certification related to onsite systems. http://www.neha.org/research/onsite_wastewater.htm

National Environmental Services Center (NESC). National information dissemination related to small community water and wastewater. http://www.nesc.wvu.edu/

National Onsite Wastewater Recycling Association (NOWRA). U.S. based professional organization with annual conferences and exhibitions and related project activities. http://www.nowra.org/

NSF International (NSF). Independent accredited organization that tests, audits, and certifies products and systems that protect food, water, consumer products and the environment. http://www.nsf.org

Small community wastewater management, small water systems. http://www.epa.gov/owm/mab/smcomm/index.htm; http://www.epa.gov/safewater/smallsys/ssinfo.htm

U.S. Environmental Protection Agency (USEPA). Septic systems and onsite wastewater treatment and management. http://cfpub.epa.gov/owm/septic/home.cfm

United Nations Children's Fund (UNICEF). Water, sanitation and hygiene program (WASH) helps governments involve women in planning water and sanitation projects worldwide. http://www.unicef.org/wes/index_3951.html

United Nations Development Program (UNDP). Technical assistance and support for water and sanitation to serve the developing world. http://www.undp.org/water/priorityareas/supply.html

Water Environment Federation (WEF). International professional society covering all aspects of water quality and pollution control including small flows. http://www.wef.org/

Water Environment Research Foundation (WERF). Foundation for water research including decentralized systems for water, wastewater, storm water. http://www.werf.org/

Water for People (WFP). International non-profit organization that facilitates and supports community projects for water and sanitation. http://www.waterforpeople.org/

Water is Life. Program to communicate the value of water and wastewater infrastructure. http://waterislife.com/

Water Research Foundation (WRF). Foundation for water research focused on water sources and water supply. http://www.waterresearchfoundation.org/

WaterReuse Association (WRA). Non-profit organization focused on water reuse through research, technical support, and policy development. http://www.watereuse.org/information-resources/reuse/resources

WaterSense. A partnership program by the U.S. Environmental Protection Agency that promotes water efficiency through product labeling and information dissemination. http://www3.epa.gov/watersense/

World Health Organization (WHO). Technical assistance and policy support for water and sanitation to serve the developing world. http://www.who.int/water_sanitation_health/en/

Slides of Chapter 1

Decentralized Water Reclamation

Chapter 1: Introduction to Decentralized Infrastructure for Wastewater Treatment and Water Reclamation

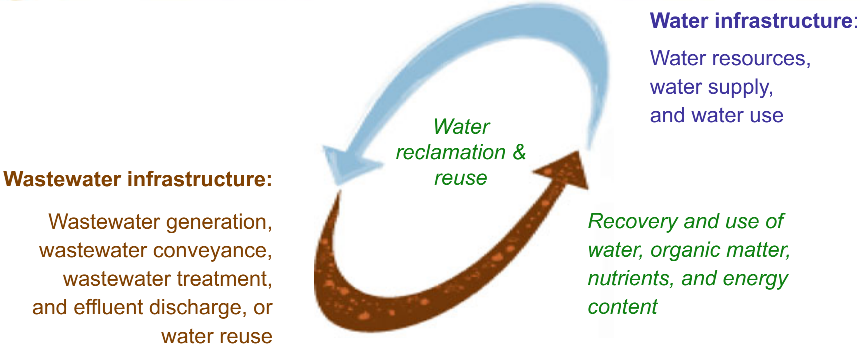
Contents

1.1

1-1. Introduction

- Water use generates wastewaters
  - Humans use water for various purposes including drinking, bathing, fishing, swimming, food production, etc.
    - Water use often requires water treatment and supply, which typically involves use of energy, chemicals, and materials
  - The use of water by humans generates wastewaters
    - Wastewaters contain chemical and microbial constituents and management is needed to mitigate public health and environmental risks
    - Wastewaters also contain water, organic matter, nutrients, and energy, which can have sufficient value to warrant recovery and use
  - Water and wastewater infrastructure are inextricably linked
    - Wastewater treatment for water reclamation and reuse is a natural or engineered outcome (Fig. 1.1)

1.2



Note: Infrastructure encompasses the basic physical and organizational structures and facilities needed for a given function such as water treatment and supply or wastewater treatment and discharge or water reuse.

Fig. 1.1 Water and wastewater infrastructure are inextricably linked by the actions of humans

1.3

- Water and wastewater infrastructure are critical to achieving and sustaining a healthy society
  - United Nations Human Development Index (HDI) is a measure of the quality of life and prosperity in a society (Fig. 1.2)
  - Along a nation's timeline of evolving to a high HDI:
    - Efforts are initially focused on providing safe drinking water
    - As safe drinking water becomes widely available, health and well-being increase
    - The population can become more affluent, which leads to increased water use and wastewater generation
    - Efforts then move to include effective management of wastewaters

Fig. 1.2 Illustration of the UN human development index (after UN 2010)

1.4

- Modern solutions for water and wastewater infrastructure need to be *effective* while being *affordable*, *socially acceptable*, and *sustainable*
  - Modern solutions for safe drinking water should:
    - Protect our raw water supply sources
    - Minimize chemicals and energy used in water treatment and delivery
    - Minimize drinking water used for cleaning and waste carriage
  - Modern solutions for wastewater management should:
    - Minimize wastewater volumes and reduce pollutant loads
    - Minimize the use of chemicals and energy in treating waste-water to reclaim and clean the water
    - Maximize the beneficial recovery and reuse of wastewater resources including water, nutrients, organic matter and energy

1.5

- So, where are we today. . .? Where are we going. . .?
  - Answering this question depends on the context
    - In the United States and similar industrialized countries
    - In developing countries and regions of the world
  - An assessment of where we are and where we are going should include consideration of attributes such as shown in Fig. 1.3
    - Acceptability and sustainability attributes can be particularly difficult to assess

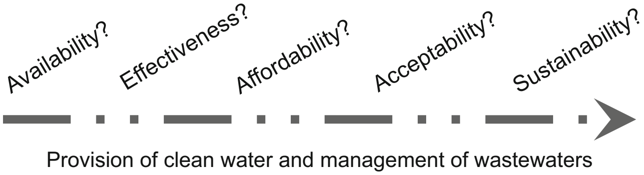


Fig. 1.3 Timeline attributes important to assessing the status and future of water and wastewater infrastructure in a particular situation

1.6

1-2. Wastewater Perspectives

- Wastewater has long been recognized for the risks it poses
  - Wastewater can pose inherent risks to human health or the environment due to its physical, chemical and biological constituents
  - Fundamentally, the challenge is to assess the magnitude of the risks in a given situation and decide on the most appropriate way to manage those risks
    - For example, pathogenic bacteria, virus, and protozoa are present in wastewater, and infectious disease could result if they are not removed or inactivated before an effluent reaches a receiving environment where humans can contact and ingest the water (e.g., drinking water, bathing beaches, shellfish beds)
    - Also, if excessive levels of nitrogen and phosphorus in wastewater are input to sensitive surface waters (e.g., pristine lakes, estuaries), this could result in undesirable ecosystem changes (e.g., increased productivity and eutrophication)

1.7

- Design and implementation to manage risks
  - Risk-based design and implementation of wastewater systems is desirable but can be quite difficult to explicitly accomplish
  - One could state the ultimate goal as being system design and implementation so that (1) there is no infectious disease attributable to a wastewater system, and (2) there is no measurable change in an ecosystem attributable to wastewater system inputs
  - Clearly, in a given setting, a wastewater system that provides no treatment at all may present the highest risk, while increasing levels of reliable treatment effectiveness could yield reduced levels of risk
  - However, since risk management requires consideration of nontechnical issues, such as acceptability and sustainability, the most advanced treatment system may not be the best overall risk management solution

1.8

- Federal and state requirements for design and implementation
  - Federal and state requirements may be based in part on risk-based considerations but requirements for selection, design, and implementation are typically not explicitly risk-based
  - Guidelines, criteria and standards can be used to define the level of wastewater treatment required and the quality of the effluent produced before its disposition or use
  - Treatment and water quality requirements are not always the same in multiple jurisdictions (e.g., requirements of one state vs. another in the United States or from one country to another)
    - The reason for differences is often not clear, but one explanation is that public health or environmental effects associated with pollutants and pathogens in water involves complex and sometimes uncertain concentration-risk relationships that require subjective interpretation

1.9

- Wastewater is increasingly being recognized for the resources it contains
  - Wastewater represents a resource by virtue of its content of:
    - Water—Water reclaimed from wastewater represents a valuable alternative water supply source
    - Organic matter—Organic matter recovered from wastewater can be used as a soil amendment or fertilizer
    - Nutrients—Nutrients (e.g., N, P, K) in wastewater represent a potentially valuable alternative to commercial chemical fertilizers
    - Energy content—Energy can be recovered from the organic matter in wastewater (e.g., biogas production)
  - The challenge is to select, design and implement approaches, technologies and systems that can recover resources of value in a given situation while also mitigating risks

1.10

1-3. Wastewater Infrastructure

- In the United States during the 20th century, major U.S. investments were made leading to:
  - Knowledge, laws, and regulations
  - Modernized fixtures and appliances and plumbing systems
  - Construction of water and wastewater infrastructure[a]
    - New and improved wastewater collection and treatment systems
    - Expansion of service areas and increased accessibility
- At the close of the 20th century, most of the U.S. population had acceptable and affordable access to:
  - Safe drinking water
  - Adequate wastewater management

[a] Infrastructure = The basic physical and organizational structures and facilities needed.

1.11

- Features of wastewater infrastructure in the United States
  - Infrastructure features are highlighted in Table 1.1 and Fig. 1.4

Table. 1.1 Features of wastewater infrastructure in the United States around the end of the 20th century

Type[a]	Representative features	Applications and scale of use
Centralized infrastructure ↑	Extensive collection system piping for long-distance transport of wastewaters for remote treatment at energy consuming, mechanical plants with discharge of treated effluents to surface waters. Engineered plants can have high operation and maintenance requirements but they can yield a high capacity per unit of land area which is often needed in densely populated areas	Commonly used for cities, suburban areas and larger development centers. About 16,000 municipal systems serving nearly 75 % of the nation's population (USEPA 2004)
↓ Decentralized infrastructure	Local treatment at, or very near, the building(s) where wastewater is generated. Treatment using lower energy, engineered or natural systems with discharge of treated effluents to the land (with recharge to groundwater) or surface waters. Treatment systems generally have low operation and maintenance requirements and provide a low capacity per unit of land area	Widely used for individual homes and businesses in rural areas and for smaller developments and towns. About 25,000,000 systems serving about 25 % of the nation's population (USEPA 1997)

[a]There is a continuum across developments served, scale and complexity of technology used, and water and resources used.

1.12

a. Cities and urban areas in high density settings with limitted land available

b. Small towns and clusters of houses and businesses in lower density settings, often with land area available

c. Individual houses and businesses in irural areas with land area available

Fig. 1.4 Illustration of classic centralized (**a**) and decentralized infrastructure (**b**, **c**). *Note*: Decentralized infrastructure includes onsite systems (**c**)

1.13

- What distinguishes "decentralized" and "onsite" infrastructure?
  - There are subtle differences
    - Onsite—Involves applications where wastewater treatment and discharge/reuse occurs on the same property where the source of the wastewater generation is located

      Example: a property with a single family home with a treatment and reuse system on that property
    - Decentralized—Involves applications where wastewater treatment and discharge/reuse occurs on a property close to the location(s) where the source(s) of wastewater generation is (are) located

      Example: multiple properties with a collection network and a treatment site located near the development
  - Decentralized terminology will be used throughout this book as it is commonly used today and it does encompass onsite

1.14

- Evolution of 20th century wastewater infrastructure
  - During the latter part of the 20th century, major U.S. Federal and State programs provided funding for construction of new and expanded centralized infrastructure (Table 1.2)
    - This was done to improve the quality of life in urbanized areas—often located near rivers, lakes, and coastal zones—where population densities were high and the risks associated with wastewater were also high
  - During this time there was little funding available for construction of decentralized systems serving homes and businesses
    - The wastewater-related risks were lower due to low population densities and locations in rural areas
    - In addition some viewed decentralized systems as temporary and only needed until they were replaced by a centralized system

1.15

Table 1.2 Government funding that promoted construction of centralized infrastructure for wastewater management in the United States (after Anderson and Otis 2000)

Government funding that promoted construction of centralized infrastructure
• The Clean Water Act was passed in 1972
• The Clean Water Act led to the Construction Grants Program (CGP) – This program provided over $62 billion and covered 75 % of the cost of construction of centralized sewers and wastewater treatment plants from 1972 to 1990 – From 1972 until the 1990s most of the larger secondary wastewater treatment plants were constructed
• The CGP was followed by the State Water Pollution Control Revolving Fund Program – This program provided billions more of low interest money for centralized wastewater management
• Meanwhile, little to no funding was provided for decentralized infrastructure

1.16

- During much of the 20th century, decentralized wastewater systems were used in rural areas and other areas with low population densities
  - Many of these systems were not designed or implemented to achieve treatment and reuse objectives over long-term use
  - Not surprisingly, such systems often suffered performance deficiencies ranging from hydraulic failures to localized contamination of groundwaters and surface waters
    - These were attributed to varied causes including poor system siting, improper design, faulty installation, and/or inadequate operation and maintenance
  - During the latter part of the 20th century, research and educational initiatives, along with changes in regulatory requirements and advancements in management and performance assurance, helped improve the standard-of-practice and mitigate performance deficiencies

1.17

- Growth in centralized infrastructure for wastewater management began to level off in the 1970s
  - Many urbanized areas of the United States had new and expanded centralized infrastructure for wastewater management
  - It was increasingly clear that larger centralized systems were not technically feasible or affordable to serve buildings and developments located in most rural and peri-urban areas and many small towns
- Then, during the 1990s, concerns grew about the sustainability of large centralized infrastructure (Table 1.3)
- At the same time, there was growing interest in decentralized infrastructure due to the potential benefits it might yield (Table 1.4)

1.18

Table 1.3 Example reasons for growing concerns expressed during the 1990s about the sustainability of large centralized infrastructure for wastewater management

Reason for concern	Explanation
Design lifespans were being reached	Large infrastructure systems were coming toward the end of their design lifespans and the costs and technical challenges of repairs and rehabilitation were viewed as enormous and in some cases technically impracticable
Safe drinking water losses and wasteful uses	During water distribution the leakage from pipelines and fixtures and appliances can be significant (e.g., up to 30 % of the clean water produced at a water treatment plant) plus toilet flushing for waste carriage can consume large quantities of drinking water (e.g., nearly 30 % of total indoor usage in homes and residences)
Public health and ecosystem impacts	Public health and ecosystem impacts were occurring through discharges of untreated wastewaters into rivers and coastal zones due to sewer overflows (e.g., combined sewer overflows) and treatment plant malfunctions (e.g., pump station failures)
High operation and maintenance costs	There was high consumption of energy resources and requirements for materials and chemicals to support conveyance networks and treatment plant operations
Barriers posed by co-mingling	Centralized infrastructure often results in co-mingling of domestic and industrial waste streams, which can present barriers to resource recovery and reuse

1.19

Table 1.4 Potential benefits to sustainability provided by decentralized infrastructure for wastewater treatment and water reclamation

Potential benefits to sustainability provided by decentralized infrastructure
Avoid large capital costs and reduce operation and maintenance costs
Reduce the use of drinking water to flush toilets and transport waste to remote wastewater treatment plants
Prevent pollutant discharges from large centralized systems by reducing or eliminating sanitary sewer overflows (SSOs), combined sewer overflows (CSOs), and leakage from conventional gravity sewers
Recharge water near the point of water extraction and avoid water export and depletion of local water resources (e.g., declining groundwater levels or stream flows)
Enable recovery and use of water, organic matter, and nutrients (N, P, K) in domestic wastewaters
Lower consumption of energy and chemicals, and reduce release of greenhouse gases through use of water efficient fixtures and appliances and natural treatment system technologies

1.20

- Advancing decentralized infrastructure in the 21st century
  - Activities and events around the turn of the Century helped promote decentralized infrastructure into the 21st century
  - 1997—The U.S. Environmental Protection Agency (USEPA) prepared a report to the U.S. Congress on the appropriate use of onsite and decentralized systems and concluded that:
    - "Adequately managed decentralized wastewater systems are a cost-effective and long-term option for meeting public health and water quality goals."—www.epa.gov/owm/mtb/decent/response/index.htm

    In their report (USEPA 1997), USEPA identified five major barriers to overcome:
    - Restricted access to funding for system construction and operation
    - Legislative and regulatory constraints on funding and implementation
    - Existing engineering practices favoring centralized infrastructure
    - Misinformation and limited knowledge about decentralized systems
    - Providing effective management of decentralized infrastructure

1.21

- 1997—U.S. Congress with USEPA, initiated the National Decentralized Water Resources Capacity Development Project (NDWRCDP) http://www.ndwrcdp.org/
  - An initial $8.2 M in funding supported projects to overcome the barriers identified in the 1997 USEPA report
  - Additional funding was provided in subsequent phases
- 2000s—U.S. Congress provides $15.6 M for projects in six areas to demonstrate decentralized technologies and management
- 2002 and 2003—USEPA published a new "Onsite Wastewater Treatment Systems Design Manual" and "Voluntary Guidelines for Management of Onsite and Clustered Wastewater Treatment Systems"
- 2003—NDWRCDP sponsored workshops focused on "Soft Path Integrated Water Resource Management"

1.22

- 2005—The U.N. Millennium project called out the need for clean water and sanitation worldwide
  - The basis for the worldwide need as assessed at the time included:
    * 2.5 billion people lacked appropriate sanitation
    * 1.2 billion people lacked clean water supply
    * 3.4 million people *died* yearly due to waterborne disease (Fig. 1.5)

Fig. 1.5 Illustration of water and wastewater in a low HDI setting

  - A U.N. Millennium Development Goal was to reduce by 50% the number of people without clean water and sanitation by 2015
  - Decentralized approaches, technologies and systems were viewed as necessary and appropriate

1.23

- 2007—"Baltimore Charter for Sustainable Water Systems" was prepared and signed by individuals from countries worldwide

"Water is at the heart of all life. In the past, we built water and wastewater infrastructure to protect ourselves from diseases, floods, and droughts. Now we see that fundamental life systems are in danger of collapsing from the disruptions and stresses caused by this infrastructure.

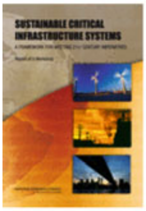
New and evolving water technologies and institutions that mimic and work with nature will restore our human and natural ecology across lots, neighborhoods, cities, and watersheds. We need to work together in our homes, our communities, our workplaces, and our governments to seize the opportunities to put these new designs in place. . . .

We commit to implementing more sustainable water systems by expanding uses and opening new markets for small-scale treatment processes, advancing research on micro-biological and macro-ecological scales, inventing new technologies based on nature's lessons, creating new management and financial institutions, reforming government policies and regulations, and elevating water literacy and appreciation in the public."

Source: http://www.ndwrcdp.org/documents/Balto_Charter.pdf

1.24

- 2008–2009—U.S. National Academy of Engineering panel reviews and special reports emphasized the need for sustainable water and wastewater infrastructure. . .
  . . .including decentralized approaches, technologies and systems (Fig. 1.6)



Sustainable Critical Infrastructure Systems: A Framework for Meeting 21st Century Imperatives

Toward Sustainable Critical Infrastructure Systems: Framing the Challenges Workshop Committee; National Research Council

ISBN: 0-309-13793-4, 82 pages, 7 x 10, (2009)

Free PDF can be downloaded from: http://www.nap.edu/catalog/12638.html

Technologies for Clean Water

Vol. 38, No. 3, Fall 2008. 72 pages.

Free PDF can be downloaded from: http://www.nae.edu/TheBridge

Fig. 1.6 Cover pages from two recent U.S. National Academy publications

- 2009—The Decentralized Water Resources Collaborative (DWRC) emerged
  - DWRC is a cooperative effort managed by the Water Environment Research Foundation (WERF) and funded by the USEPA (Fig. 1.7)
  - DWRC supports research and educational initiatives focused on decentralized wastewater and stormwater
  - DWRC research reports and other products were developed during 2009–2011

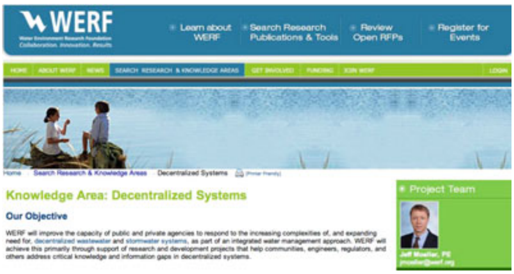


Fig. 1.7 Home page of the WERF website for decentralized systems. www.werf.org/AM/Template.cfm?Section=Decentralized_Systems

- DWRC research and products dissemination encompasses septic tanks and onsite systems, watershed scale solutions, urban applications and stormwater
- DWRC products include: technical reports, modeling tools, and decision aids available through a website, FAQ guide, product matrix guide, and videos (Fig. 1.8)

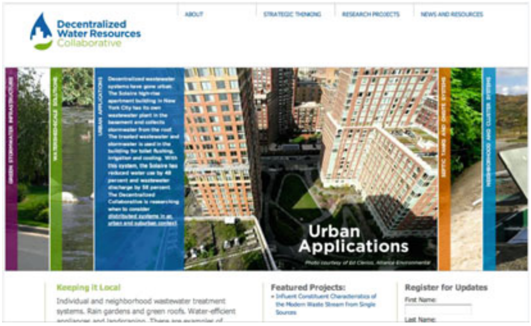


Fig. 1.8 Home page of the DWRC website for onsite and decentralized systems. www.decentralizedwater.org/

1.27

- 2011—U.S. National Science Foundation sponsors a major Engineering Research Center on water
  - The NSF ERC on "Reinventing the Nation's Urban Water Infrastructure (ReNUWIt)" was launched in August 2011 (Fig. 1.9)
  - ReNUWIt has a broad array of research and educational thrusts, and decentralized approaches and the associated technologies are one facet of the ERC

Fig. 1.9 Home page of the website for the NSF research center for reinventing urban water infrastructure. www.urbanwatererc.org/

1.28

- 2012—U.S. Decentralized Partnership to promote the use and improve the performance of decentralized wastewater treatment
  - The U.S. Decentralized Partnership is an agreement between the USEPA and 16 partner organizations to work collaboratively at the national level to improve decentralized system performance (Fig. 1.10)
  - Four papers described decentralized system uses and benefits:
    - Introduction to Decentralized Wastewater Treatment: A Sensible Solution
    - Decentralized Wastewater Treatment Can be Cost Effective and Economical
    - Decentralized Wastewater Treatment Can Be Green and Sustainable
    - Decentralized Wastewater Treatment Can Protect the Environment, Public Health and Water Quality

Fig. 1.10 The EPA decentralized memorandum of understanding partnership papers were released in 2012 (USEPA 2012). (http://water.epa.gov/infrastructure/septic/Decentralized-MOU-Partnership-Products.cfm)

1.29

- In the 21st century, water and wastewater paradigms are increasingly driven by sustainability concerns
  - View of water more holistically, differentiated by quality and intended uses rather than lumped as drinking water, stormwater, or wastewater
  - Approaches, technologies and systems are increasingly being judged based on their sustainability attributes, including
    - Human and environmental interactions and effects
    - Resilience to natural or human influenced upsets
    - Ability to deal with climate change

1.30

- Decentralized infrastructure in the 21st century
  - Decentralized approaches, technologies and systems can contribute to a 21st century sustainable water and wastewater infrastructure
    - To help promote and accomplish this, it is important to clearly understand decentralized infrastructure attributes and the potential uses and benefits in different applications
  - It is also critical to to clearly recognize and appreciate the differences between modern infrastructure and legacy systems
    - Modern 21st century decentralized infrastructure can be implemented in rural, peri-urban, suburban and urban areas for longer-term, high efficiency treatment and often water reuse and resource recovery
    - In contrast, older 20th century decentralized systems are often legacy systems that were installed in rural areas for shorter-term waste disposal

1.31

1-4. Decentralized Infrastructure

- Decentralized infrastructure is normally deployed to help achieve one or more of the following project goals
  - Effective treatment and disposal of wastewaters in areas where a decentralized system is the only option or where decentralized systems offer desired benefits
  - Treatment of wastewaters to provide a reclaimed water source in areas where decentralized systems can yield benefits by co-locating wastewater generation near a water reuse site
  - Minimized resource consumption and maximized resource recovery in areas where these are desired by the project owners for various reasons such as to support an environmental consciousness, realize cost incentives or savings, and earn points to achieve a desired sustainability rating

Note: Decentralized infrastructure can also be used for stormwater and other impaired waters.

1.32

- Decentralized infrastructure components include those listed below (Fig. 1.11)
  - Source modification options
    - Ultra efficient fixtures and appliances
    - Waste stream source separation
  - Collection and conveyance options
  - Treatment options
    - Bioreactors
    - Recirculating biofilters
    - Membrane bioreactors
    - Constructed wetlands
    - Subsurface soil infiltration
    - Landscape drip dispersal
    - Nutrient removal units
    - Disinfection units
  - Sensors and intelligent control options
  - Discharge and reuse/recovery options
    - Surface or subsurface discharge and groundwater recharge
    - Nonpotable water reuse
    - Recovery of wastewater organic matter, nutrients, energy

1.33

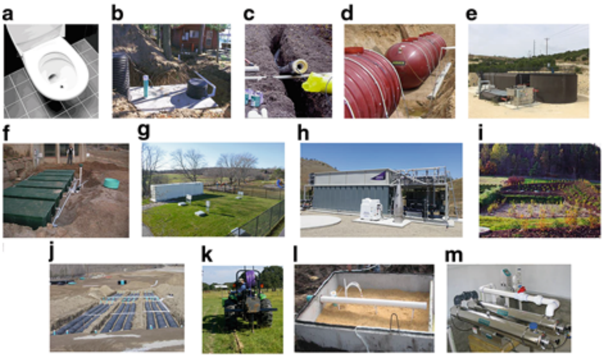
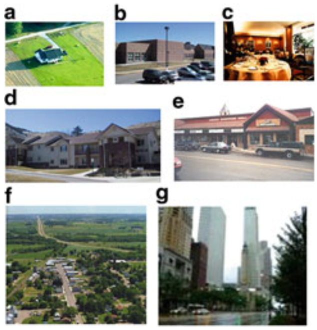


Fig. 1.11 Examples of system components: (**a**) urine diverting toilet, (**b**) septic tank and pump vault, (**c**) small diameter sewer, (**d**) primary settling and recirculation fiberglass tanks, (**e**) aerobic treatment unit, (**f**) recirculating textile media biofilters, (**g**) recirculating foam filter in a shipping container, (**h**) membrane bioreactor, (**i**) subsurface flow constructed wetland, (**j**) chamber-equipped subsurface soil treatment unit, (**k**) landscape drip dispersal unit, (**l**) denitrifying wood chip biofilter, (**m**) ultraviolet light disinfection unit

1.34

- Applications of decentralized infrastructure are varied
  - Development types include (Figs. 1.12 and 1.13):
    - Single homes, businesses, and institutions in rural and peri-urban areas
    - Neighborhood and commercial developments in small towns and sub-urban areas
    - Buildings and higher density developments in larger cities

Fig. 1.12 Illustration of developments where decentralized infrastructure has been deployed: (**a**) individual home, (**b**) school, (**c**) restaurant in rural areas, (**d**) apartment building or (**e**) strip mall in suburban areas, (**f**) homes and businesses in a small town or (**g**) high rise office and condominium buildings in a city



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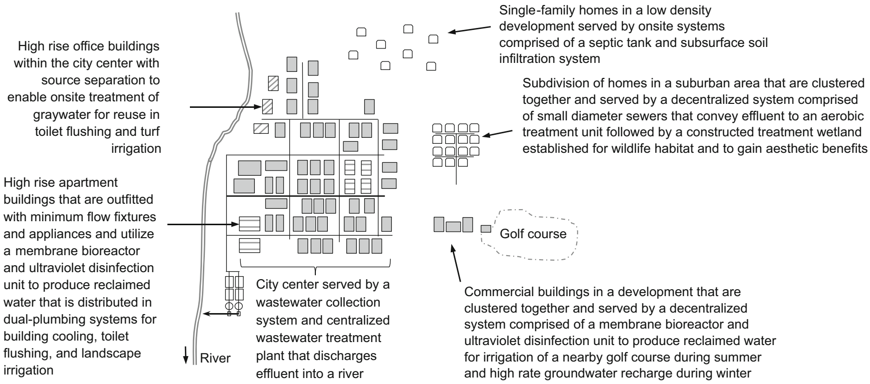


Fig. 1.13 Illustration of a city center area with nearby urbanizing areas where decentralized infrastructure can be deployed along with centralized infrastructure. (Note that management of decentralized infrastructure can be handled by a single central authority or by multiple authorities)

1.36

- Decentralized infrastructure can handle a wide range of design flows (Table 1.5)

Table 1.5 Decentralized systems can handle a wide range of design flows[a]

Size classification	Design flow (gal/day)	Example applications	Comments
Very small	<100	Single house or small clusters with minimum flows or source separated waste streams	Very small systems could handle the total flow if there are very low flows (e.g., due to ultra efficient fixtures and appliances) or just the diverted urine or graywater
Small	up to 2500	Individual houses, small businesses, recreational sites	This design flow often distinguishes a "small" versus "large" onsite system with respect to local versus state regulations, respectively, in the United States
Medium	2500–10,000	Schools, restaurants, country clubs	Medium size systems are often a transition size where more advanced technologies are increasingly deployed to produce higher quality effluents for discharge or water reuse
Large	10,000–100,000	Communities, residential developments, commercial complexes	Large and very large systems will often include clusters of sources with alternative conveyance to a local site where advanced technologies will be deployed for treatment and potentially water reuse and resource recovery
Very large	>100,000	Larger communities and developments	

[a]The size classification is based on the author's views with input from others and is for illustration purposes only

1.37

- Decentralized systems are configured from approaches and technologies
  - Compatible components are configured for a goal (Fig. 1.14)
  - A few example system configurations are illustrated in the following pages

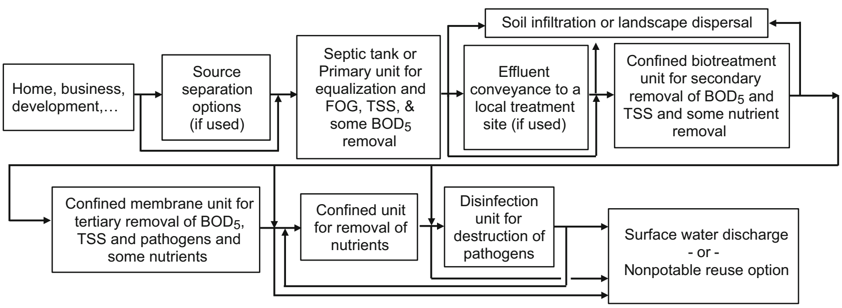
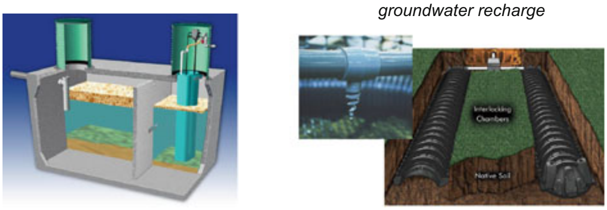


Fig. 1.14 Illustration of example approaches and technologies that can be used to configure a decentralized system for a particular discharge or reuse goal

1.38

- Illustration of a system for individual residential units and businesses in rural and peri-urban areas
  - Where land area is available and soil and site conditions are suitable for soil-based treatment (Fig. 1.15)

Raw wastewater ➔ *Advanced septic tank (pump unit, controls/sensors, effluent screen)* ➔ *Subsurface chambers w/ spray distribution and soil treatment before groundwater recharge*



Source: : Orenco Systems®, Inc.

Source: Infiltrator® water technologies

Fig. 1.15 Example of a system and illustration of key components for use at single houses and businesses where soil and site conditions are suitable for soil-based treatment

1.39

- Where land area is available and soil and site conditions are suitable for soil-based treatment and water reuse and nutrient recovery is desired (Fig. 1.16)

Raw wastewater ➔ *Advanced septic tank w/ integral textile biofilter and recirculation for denitrification* ➔ *Disk filter and landscape drip dispersal for effluent polishing and water and nutrient uptake by plants*

Source: : Orenco Systems®, Inc.

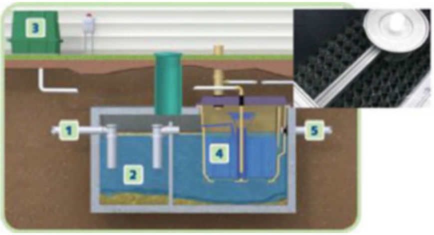
Source: Siegrist et al. 2014

Fig. 1.16 Example of a system and illustration of key components for use where soil and site conditions are suitable for soil-based treatment and water reuse and nutrient recovery is desired

1.40

- Where advanced treatment is needed for discharge to a local inland stream or lake (Fig. 1.17)

Raw wastewater ➔ *Aerobic treatment unit (submerged fixed film unit, sensors, effluent screen)* ➔ *Ultraviolet light disinfection* ➔ *Stream discharge*



Source: www.biomicrobics.com

Source: SCG Enterprises, Inc.

Fig. 1.17 Example of a system and illustration of key components for use where advanced treatment is needed to enable discharge to a local inland stream or lake

1.41

- Illustration of a system for multiple buildings in developments or small towns (Fig. 1.18)
  - Where land use planning and density characteristics require wastewater collection and conveyance to a local treatment and reuse site

Septic tank from each building ➔ *Small diameter gravity or pressure sewers* ➔ *Subsurface flow constructed wetland* ➔ *Drip dispersal effluent network for irrigation*

Source: Orenco Systems®, Inc.

Source: R.J. Otis

Fig. 1.18 Example of a system and illustration of key components used to serve a development or small town including wastewater collection and conveyance to a local site for treatment and reuse

1.42

- Illustration of a system for high rise buildings in highly urbanized settings and cities (Fig. 1.19)
  - Where water reuse and green building certification is desired

Untreated wastewater ➔ *Membrane Bioreactor* ➔ *Ultraviolet light / Ozone disinfection* ➔ *Reuse for toilet flushing, turf irrigation, cooling tower*

Fig. 1.19 Example of a membrane bioreactor system serving a high rise apartment building in a major urban area to produce reclaimed water for nonpotable reuse and help earn LEED certification. (Photographs are of the Solaire building in Battery Park City, NY which has 250 apartment units. http://www.werf.org/i/c/Decentralizedproject/When_to_Consider_Dis.aspx)

1.43

- System selection and design can be informed using a growing array of models and other decision aids
  - Analytical models for design and performance (Geza et al. 2014, Fig. 1.20)
  - Numerical models for simulating performance under complex conditions
  - Watershed models to simulate cumulative effects on water resources (Siegrist et al. 2005, Fig. 1.21)

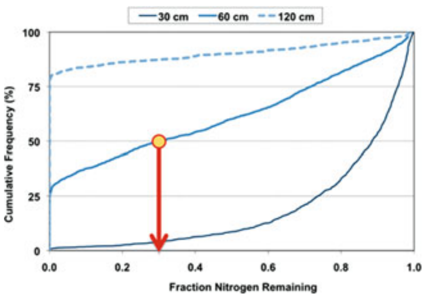


Fig. 1.20 Simulations with STUMOD revealing there is a 50 % probability of 70 % nitrogen removal at 60-cm depth in a subsurface soil treatment unit under the assumed conditions

1.44

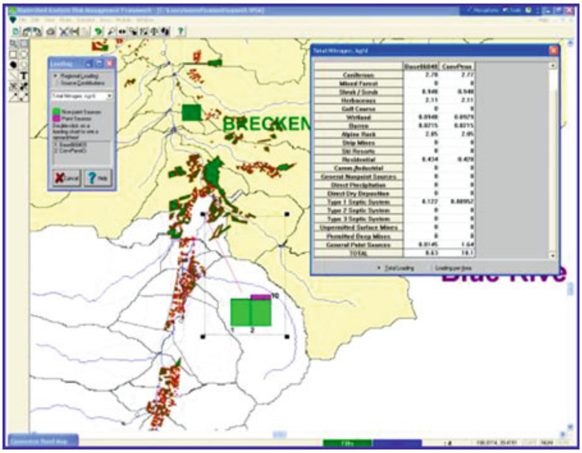


Fig. 1.21 Simulations with the watershed model, WARMF, revealing there would be an increase in nutrient loading to the Blue River if onsite systems are replaced by a centralized system

1.45

- Management of decentralized infrastructure is critical to achieving and sustaining a performance outcome
  - What is meant by “management”?
  - Management involves public and/or private entities and a set of activities, often organized within a jurisdiction, to assure:
    - Decentralized systems are properly considered during infrastructure and land use planning, and
    - If selected they are properly designed, constructed, and operated so the performance planned for are sustainably achieved
  - As decentralized systems have evolved to become a permanent part of the U.S. water and wastewater infrastructure, the need for, and critical role of, approaches for effective management have also evolved (refer to Chap. 2)

1.46

1-5. Summary

- Decentralized wastewater infrastructure has evolved and can now provide sustainable long-term solutions for:
  - Effective treatment and disposal of wastewaters
  - Treatment of wastewaters to provide a reclaimed water source
  - Minimized resource consumption and enhanced recovery
- Modern decentralized infrastructure encompasses approaches, technologies and systems that include:
  - Ultra high water use efficiency fixtures and appliances and in-building source separation
  - Small diameter wastewater collection and conveyance networks
  - Reactor-based and landscape-based treatment unit operations
  - System monitoring and performance assurance methods
  - Management systems to help assure long-term sustainability

1.47

Chapter 2

Selection, Design and Implementation of Decentralized Infrastructure

2-1. Scope

Decentralized infrastructure involves approaches, technologies and systems that are selected based on project goals and requirements under a specific set of circumstances. For a particular project, one or more decentralized systems can be configured from compatible approaches and technologies. System design and implementation needs to satisfy technical and deployment viability and sustainability requirements. Management is essential to successful deployment of decentralized systems. This chapter describes how decentralized systems can be selected, designed, and implemented to achieve project goals and satisfy requirements in a sustainable fashion.

2-2. Key Concepts

- Infrastructure can be defined as the basic physical and organizational structures and facilities needed for a given function within a society. In the context of wastewater management and water reclamation, decentralized infrastructure can encompass an array of approaches, technologies, and systems.
  - For example, decentralized infrastructure can span from:
    - Installation of ultra-efficient fixtures and appliances (e.g., waterless or urine diverting toilets) in buildings to reduce water use and wastewater generation, to
    - Installation of a complete treatment system (e.g., membrane bioreactor and ultraviolet light disinfection unit) to enable nonpotable water reuse for toilet flushing and irrigation.

R.L. Siegrist, *Decentralized Water Reclamation Engineering*,
DOI 10.1007/978-3-319-40472-1_2

- Decentralized infrastructure can be deployed to help achieve one or more project goals that commonly include:
  - Effective treatment and disposal of wastewaters in areas where a decentralized system is the only option or where decentralized systems offer desired benefits.
  - Treatment of wastewaters to provide a reclaimed water source in areas where decentralized approaches can yield benefits by co-locating wastewater generation near a water reuse site.
  - Minimize resource consumption and maximize resource recovery in areas where these are desired by the project owners for various reasons such as to support an environmental consciousness, realize cost incentives or savings, and earn points to achieve a desired green building rating.

- A variety of general considerations can influence if and how decentralized infrastructure might be deployed for a particular project including:
  - What are the key characteristics of the source(s) or development to be served by the approach, technology or system?
  - What are the optional receiving environments and boundary conditions that determine what water quality is required for a potential discharge or reuse option?
  - Is the project to provide upgraded or new service?
  - Are there any existing or planned systems near the existing or planned building or development?
  - What is the design life of the approach, technology or system?
  - How will the approach, technology or system be paid for and managed?
  - What regulatory program(s) governs design and implementation of an approach, technology or system?

- Selection, design and implementation of decentralized infrastructure must ensure viability and sustainability.
  - Approaches, technologies, and systems that are considered "technically viable" for a particular application are those that are inherently capable of achieving a required goal (e.g., a required treatment efficiency for an intended discharge or reuse plan and capable of satisfying high priority owner requirements).
  - Approaches, technologies, and systems that are considered "deployment viable" are those that are also compliant with applicable regulations and codes.
  - Approaches, technologies, and systems that are considered "sustainable" must provide a reliable performance in an affordable manner over the long-term and also yield an acceptable level of resource use and environmental impact.

- For most projects, decentralized infrastructure includes one or more systems for wastewater treatment and water reclamation, potentially including resource recovery.
  - Chapter 2 is focused on the selection, design and implementation of viable and sustainable decentralized systems for wastewater treatment and water reclamation.
  - As described in this chapter, a viable and sustainable decentralized system can configured from one or more approaches and technologies and can include minimum flow fixtures and appliances, waste stream source separation and treatment technologies and reuse and recovery options.

- Project requirements drive system selection, design and implementation and these can be categorized to include:
  - Requirements based on treatment efficiency and water quality—Treatment performance must protect public health and preserve or enhance environmental quality and also provide for an operation that is robust and reliable.
  - Requirements based on owner needs and desires—Owner(s) can have specific views about what infrastructure they want to implement, including: attributes related to aesthetics and land use planning, costs relative to an available budget, sustainability attributes, and contribution to achieving a certain green building or infrastructure rating.
  - Requirements based on regulations and codes—Codes and regulations can constrain what can be permitted and also specify how design and implementation is done.

- Project requirements can include specified treatment efficiency targets for constituents of concern (e.g., achieve $\geq$50 % removal of total inorganic nitrogen with an effluent concentration $<$10 mg-N/L). A system must have the inherent capability to achieve a target treatment efficiency, be properly designed and implemented for a particular project, be properly operated and maintained, and if needed be appropriately monitored to verify performance is achieved.

- Constituents of concern can be removed using approaches such as source separation and treatment processes such as biodegradation that are implemented in unit operations. Compatible approaches and unit operations can be selected to form a system that has a general capability to remove the constituents of concern. Systems commonly include a treatment train that consists of a sequence of compatible unit operations that connect the source to an intended discharge or reuse option. The unit operations in a treatment train can be categorized according to their function and the constituents of concern that are removed:

- Preliminary treatment—A term used to encompass processes and unit operations that are used to accomplish the initial processing of raw wastewaters generated in buildings, which often includes the removal of debris and fats, oils, and greases. Examples of preliminary treatment include grease interceptors, coarse screening units, grinders and comminutors.
- Primary treatment—A term used to encompass processes and unit operations that remove suspended solids (organic and inorganic) from wastewater by sedimentation or flotation processes. Advanced primary treatment includes some treatment of the separated solids (e.g., by anaerobic biodegradation of settled organic solids). Examples of primary treatment operations include settling basins, septic tanks, and upflow anaerobic sludge blanket reactors.
- Secondary treatment—A term used to encompass processes and unit operations that follow primary treatment and are designed to remove biodegradable dissolved and colloidal organic matter by aerobic biological processes. Advanced secondary treatment includes transformation and removal of nutrients (e.g., removal of ammonia nitrogen using a nitrifying extended aeration bioreactor). Examples of secondary treatment operations include: extended aeration bioreactors, porous media biofilters, and constructed wetlands.
- Tertiary treatment—A term used to encompass processes and unit operations that typically follow secondary treatment and are designed to remove specific constituents such as nutrients, trace organic compounds, heavy metals or dissolved salts. Examples of tertiary treatment operations include: denitrifying porous media biofilters, adsorptive media packed bed reactors, and ion exchange columns.
- Disinfection—Refers to the process of destroying pathogenic microorganisms in a media like water so that the risk of infectious disease transmission through human contact with that media is reduced. Examples of disinfection technologies include: chlorination, ozonation, ultraviolet light irradiation, and membrane filtration.

■ Unit operations and systems can have inherent treatment capabilities, which are established based on field experiences and testing and evaluation programs. The National Sanitation Foundation has had a program of testing and certification in place for more than 40 years. Today there are a number of standards that can be used to test and certify different types of fixtures and treatment systems relative to a set of criteria that must be met. Examples of current standards include: Standard 40 for residential treatment systems, Standard 245 for nitrogen reduction systems, and Standard 350 and 350-1 for water reuse treatment systems.

- The treatment that is actually realized when a technology or system is applied at a specific project can be better or worse than an inherent capability and the performance demonstrated in testing and evaluation programs. The fundamental reason for this is that specific projects can have design and implementation that can vary in the quality of execution and the conditions actually encountered during operation can depart from those envisioned during design and implementation.
  - Design reviews and approvals, construction supervision and inspections, education and training, and certification programs for those involved in key elements can help enable proper design and implementation.
  - Operation and maintenance can be critical to achieving an inherent system treatment capability over a system design life. Operation and maintenance requirements vary in complexity and frequency of occurrence. The operation and maintenance required to ensure that an inherent treatment capability is actually realized increases with system complexity and the stringency of the treatment efficiency targets.
  - The importance of monitoring depends on the risks associated with system performance deficiencies. Monitoring methods can be used to determine the operational status or treatment performance of a unit operation or system. Monitoring data can be used to assess and alter operations to help ensure achievement of the target treatment efficiency.

- System configurations can help satisfy environmental sustainability goals. These goals can include minimizing resource use directly or via recovery along with minimizing environmental impacts associated with resource use and conveyance and treatment operations.
  - Sustainability assessment can be used during strategic planning of infrastructure in areas or regions. Life Cycle Assessment has been used to assess decentralized versus centralized infrastructure options in several urban planning areas.

- Management is crucial to ensure the viability and sustainability of decentralized systems.
  - Modern management systems involve entities and activities, often organized within a jurisdiction, to ensure decentralized systems are properly considered during infrastructure and land use planning, and if selected they are properly designed, constructed, and operated so performance is satisfactory over a long-term planning period.

2-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 2 are presented in the Slides section.

2-4. Terminology

Terminology introduced and used in Chap. 2 is defined below.

Blackwater—Wastewaters from water-flush toilets and potentially including wastewaters from kitchen sink and dishwasher uses.

Configuring decentralized systems—The engineering process of selecting and combining compatible strategies and unit operations to form a system that is considered viable and sustainable for a particular project application.

Deployment viable systems—Decentralized systems that are technically viable for a particular application and are also compliant with applicable regulations and codes.

Disinfection—Refers to the process of destroying pathogenic microorganisms in a media like water so that the risk of infectious disease transmission through human contact with that media is reduced. Example processes include chlorination, ultraviolet light irradiation, and membrane filtration. See also Natural disinfection.

E. coli—*Escherichia coli* is a bacterium found in the gut that is used as an indicator of fecal contamination of water.

Graywater—Wastewaters produced by water use in basins, sinks and appliances in residential and nonresidential buildings. Mixed graywater includes food preparation related wastewaters (e.g., kitchen sink and dishwasher) while light graywater excludes food preparation wastewaters and possibly laundry wastewaters. All types of graywater exclude toilet wastewaters, which contain human excreta. Graywater can also be spelled as greywater.

Infrastructure—The basic physical and organizational structures and facilities needed for a given function such as water treatment and supply or wastewater treatment and discharge or water reuse.

Impaired water—Refers to water that has been used or impacted in a manner as to have quality characteristics that make it unsuited for one or more uses. Examples of impaired waters include: residential and commercial wastewater, municipal wastewater, graywater, stormwater, acid mine drainage, etc.

Management systems—Management systems involve entities and activities, often organized within a jurisdiction, to ensure decentralized systems are properly considered during infrastructure and land use planning, and if selected they are properly designed, constructed, and operated so performance is satisfactory over a long-term planning period.

Maximum Contaminant Level (MCL)—The highest level of a contaminant that is allowed in drinking water in the United States under the Safe Drinking Water Act.

Natural disinfection—Refers to the destruction of pathogenic microorganisms by die-off and predation mechanisms in unit operations that are not specifically designed as disinfection agent technologies. See also Disinfection.

Performance-based design—An explicit approach to achieving performance that allows designers to develop solutions to achieve a numerical performance requirement (e.g., 10 mg-N/L) that can provide for flexibility and innovation in design, but can require monitoring to verify performance.

Prescriptive design—An implicit approach to achieving performance where regulatory requirements dictate the steps and methods to be adhered to in system planning, design, and operation and satisfactory performance is presumed to be achieved if the prescribed code requirements are met.

Preliminary treatment—A term used to encompass processes and unit operations that are used to accomplish the initial processing of raw wastewaters generated in buildings, which often includes the removal of debris and fats, oils, and greases. Examples of preliminary treatment include: grease interceptors, coarse screening units, grinders and comminutors.

Primary treatment—A term used to encompass processes and unit operations that remove suspended solids (organic and inorganic) from wastewater by sedimentation or flotation processes. Advanced primary treatment includes some treatment of the separated solids (e.g., by anaerobic biodegradation of settled organic solids). Examples of primary treatment operations include settling basins, septic tanks, and upflow anaerobic sludge blanket reactors.

Reclaimed water—Reclaimed water is wastewater that has been treated to remove inorganic and organic substances and pathogenic microorganisms to a degree that the effluent can be considered reclaimed water with a quality that is fit for the purpose (i.e., appropriate for and of a necessary standard) of an intended discharge or water reuse plan.

SCADA—An acronym for supervisory control and data acquisition systems that are used to gather and analyze real-time data to monitor and control a unit operation or system.

Secondary treatment—A term used to encompass processes and unit operations that follow primary treatment and are designed to remove biodegradable dissolved and colloidal organic matter by aerobic biological processes. Examples of secondary treatment operations include extended aeration bioreactors, porous media biofilters, and constructed wetlands.

Sustainable systems—In the context of decentralized wastewater treatment and water reclamation, sustainable systems are systems that are selected, designed, and implemented for a particular application that are capable of achieving long-term, reliable performance, have affordable costs for construction and operation, and have acceptably low resource requirements and environmental impacts.

Technically viable systems—Decentralized systems for a particular application that are capable of achieving a required treatment efficiency for an intended discharge or reuse plan and are also capable of satisfying high priority owner requirements.

Tertiary treatment (Advanced treatment)—A term used to encompass processes and unit operations that typically follow secondary treatment and are designed to remove specific constituents such as nutrients, trace organic compounds, heavy metals or dissolved salts. Examples of tertiary treatment operations include: denitrifying porous media biofilters, adsorptive media packed bed reactors, and ion exchange columns.

Treatment technique—A required process (in the United States) intended to reduce the level of a contaminant in drinking water.

Treatment train—Within a decentralized system a treatment train consists of a sequence of compatible unit operations that connect the source to an intended discharge or reuse option.

Unit operation—A physical facility (e.g., basin, column, reactor, landscape) in which a physical, chemical, and/or biological process is made to occur for the purpose of removing or destroying constituents of potential concern in wastewater or other impaired waters.

Yellow water—Term that can be used to represent human urine.

2-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 2 are listed below.

BOD	Biochemical oxygen demand
BREEAM	Building research establishment environmental assessment method
cBOD	Carbonaceous BOD
CDPHE	Colorado Department of Public Health and Environment
Coli.	Coliform bacteria
CSM	Colorado School of Mines
E. coli	*Escherichia coli*
LCA	Life-cycle assessment
LEED	Leadership in energy and environmental design
MASSTC	Massachusetts Alternative Septic System Test Center
MBR	Membrane bioreactor
N	Nitrogen
ND	None detected

NPV	Net present value
NS	Not specified
NSF	National Sanitation Foundation (U.S.), National Science Foundation (U.S.)
NTU	Normal turbidity units
O&M	Operation and maintenance
O&P	Operational and performance
RME	Responsible management entity
SCADA	Supervisory control and data acquisition
TIN	Total inorganic nitrogen
TMDLs	Total maximum daily loads to a water body
TSS	Total suspended solids
U.S.	United States of America
USEPA	U.S. Environmental Protection Agency
UV	Ultraviolet light

2-6. Problems

2.1 A university is considering expansion of student housing on the campus and is interested in exploring decentralized infrastructure and options for water reclamation and reuse. As a design professional, you were invited to a meeting with a campus planning team to discuss this. List five relevant questions you might ask during the meeting or pursue after the meeting to help begin to understand whether there would be viable options for implementing a decentralized approach, technology or system.

2.2 Environmental regulations that are relevant for decentralized infrastructure are often conservative and can limit application of innovative technology. Briefly explain why this is the case.

2.3 There are perhaps eight different basic considerations that could be important during initial planning of a suburban housing development with a design flow of 10,000 gal/day that is located near the edge of a city that is already served by conventional centralized wastewater facilities. These considerations could strongly influence whether a decentralized system might be most appropriate compared to a extending a sewer to connect the development to the city's centralized treatment plant. Which of the following would likely apply: (1) distance from the subdivision development to the city and a sewer connection point, (2) development size and density, (3) architectural style of the houses, (4) topography and natural resources where the development is located, (5) amount of excess capacity in the city's wastewater facilities, (6) developer's desire or need to maintain control over beneficial reuse?

2.4 Treatment performance requirements can be achieved by prescriptive vs. performance-based design. Briefly explain the difference in prescriptive vs. performance-based design.

2.5 Decentralized systems can combine available unit operations in an optimum fashion to achieve required performance efficiencies, satisfy owner and user requirements, and comply with regulatory requirements. For a resort development of 100 condominiums, what might be the reason why a high priority owner requirement is to minimize resource requirements and enable recovery.

2.6 Which of the following are factors to consider when assessing technical viability (check all that apply): (1) previous experience with a particular type of application, (2) treatment requirements and process reliability, (3) compatibility with other unit operations or systems, (4) power, chemical, and other resource requirements, (5) type and management of treatment residuals?

2.7 For a new subdivision development in Arizona, you are tasked with configuring a decentralized system for each of the following two goals: (1) to effectively treat the wastewater generated for discharge of effluent into a nearby creek or (2) to produce reclaimed water for landscape irrigation in the subdivision. Assemble a technically viable decentralized system you would propose. Give a brief explanation concerning the basis for your selection and state any assumptions you need or choose to make.

2.8 There are several technical, environmental, and economic benefits that can be gained by water recycling and reuse. There are also a few concerns. Describe one important benefit and one important concern.

2.9 Rating systems for green buildings can motivate selection and use of decentralized water reclamation approaches, technologies, and systems. Give two examples of an approach or technology for which a rating system such as LEED can allocate points in the general category of water efficiency.

2.10 Regulations and guidelines can control water recycling and reuse practices. Fill in the blanks to complete the following phrases that represent common features of water recycling and reuse regulations and guidelines: (1) primary emphasis is on _____ protection, (2) requirements include _____ processes plus _____ limits for different recycling and reuse options, (3) _____ is almost universally required.

2.11 Answer the following questions concerning water recycling and reuse. (1) Compared to total domestic wastewater, household graywater should present a lower risk to human health and thus be more amenable to recycling and reuse—true or false? (2) Check which one of the following levels of treatment bests describes the generally accepted practice to produce water for landscape irrigation or toilet flushing:

secondary, secondary with disinfection, secondary plus filtration and disinfection. (3) For unrestricted urban reuse, regulations often require treatment that produces a reclaimed water with very low turbidity (e.g., $\leq$2 NTU) to improve disinfection reliability—true or false?

2.12 The primary risk factor controlling the level of treatment technically required to produce reclaimed water for landscape irrigation is the degree of _____.

2.13 For each of the following four situations (a–d) state which of the following management models might be most appropriate: (1) user awareness, (2) maintenance contract, (3) RME operation, (4) RME own and operate. Use all choices (1–4) but use each one only once. Situations: (a) A decentralized system managed by a sanitation district that includes individual septic tanks at 100 homes located along a lake shore with an alternative collection system and treatment using subsurface soil infiltration at a site located on an upslope area about 2 miles away from the lake. (b) Subsurface soil infiltration serving individual homes located on 5-acre lots in a rural county of eastern Colorado. (c) A decentralized system serving a private resort development in California with 20, 4-unit condominium buildings where water reuse for landscape irrigation is planned following treatment in a centrally located treatment facility including a membrane bioreactor and UV disinfection. (d) A county in Illinois with shallow groundwater where there is an increasing use of recirculating sand filters to produce a high quality effluent for landscape drip dispersal.

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Slides of Chapter 2

Decentralized Water Reclamation

Chapter 2: Selection, Design, and Implementation of Decentralized Infrastructure

Contents

2.1

2-1. Introduction

- Decentralized infrastructure can be defined as the basic physical and organizational structures and facilities needed for a given function within a society
  - In the context of wastewater management and water reclamation, decentralized infrastructure can be considered to encompass an array of approaches, technologies, and systems
  - For example, it can range from:
    - Installation of ultra-efficient fixtures and appliances (e.g., waterless or urine diverting toilets) . . . to . . .
    - Installation of a complete treatment and reuse system (e.g., membrane bioreactor and ultraviolet light disinfection unit for nonpotable water reuse for toilet flushing and irrigation)
  - For most projects, decentralized infrastructure includes one or more systems for wastewater treatment and water reclamation, potentially including resource recovery

2.2

- Decentralized infrastructure can be deployed to help achieve one or more project goals that commonly include:
  - Effective treatment and disposal of wastewaters in areas where a decentralized system is the only option or where decentralized systems offer desired benefits
  - Treatment of wastewaters to provide a reclaimed water source in areas where decentralized approaches can yield benefits by co-locating wastewater generation near a water reuse site
  - Minimize resource consumption and maximize resource recovery in areas where these are desired by the project owners (e.g., to realize cost incentives or savings and/or earn points to achieve a desired green building rating)
- Varied considerations can influence if, and how, decentralized infrastructure is deployed (Table 2.1)

2.3

Table 2.1 Considerations influencing if and how decentralized infrastructure may help achieve goals

Consideration	Comments
What are the key characteristics of the source(s) or development to be served?	Type of source, number of users, size of source, individual building or a development of a certain density, availability and cost of utility services, geographic location, climatic conditions
What are the optional receiving environments and what water quality is needed?	Receiving environments (typically land or water) can include reuse elements (e.g., irrigation, habitat enhancement). Recycling and reuse have different water quality requirements
Is the project to provide upgraded or new service?	Upgrading an existing building, development, facility can be much more complicated than implementing a new service
Are there existing or planned infrastructure systems near the project?	Connection to an existing or planned system might be a viable option
What is the design life of the infrastructure?	Design life, including operation and maintenance requirements, can influence viable options
How will the infrastructure be paid for and managed?	Private, corporate, vs. public ownership and management can dictate what infrastructure is feasible and affordable
Regulatory program(s) requirements?	Regulations often constrain and control what can be done

2.4

- Infrastructure selection, design, and implementation
  - Decentralized infrastructure needs to be selected, designed, and implemented to ensure viability and sustainability
  - Approaches, technologies, and systems that are considered "technically viable" for a particular application are those that are:
    - Capable of achieving a required treatment efficiency for an intended discharge or reuse plan
    - Capable of satisfying high priority owner requirements
  - Approaches, technologies, and systems that are considered "deployment viable" are those that are also compliant with applicable regulations and codes
  - Approaches, technologies, and systems that are considered "sustainable" must provide a reliable performance in an affordable manner over the long-term and also yield an acceptable level of resource use and environmental impact

2.5

- Depending on the project goals and requirements, decentralized infrastructure can take many forms
  - Decentralized approaches can be used within a centralized infrastructure setting, e.g.:
    - Installation of ultra-efficient fixtures and appliances in buildings to minimize water use and wastewater generation
  - Decentralized systems can be configured from technologies that are applicable to decentralized infrastructure settings, e.g.:
    - Compatible unit operations can be combined into one or more systems to provide wastewater treatment and water reclamation
- Focus of Chap. 2
  - Chapter 2 is focused on the selection, design, and implementation of viable and sustainable decentralized systems for wastewater treatment and water reclamation including systems which can enable resource recovery

2.6

2-2. Project Requirements

- Project requirements can be categorized to include:
  - Requirements based on treatment efficiency and water quality
    - System performance must yield a water quality suited to an intended discharge or reuse function
  - Requirements based on owner needs and desires
    - Owner(s) can have specific views about what system they want to implement, including:
      - Attributes related to aesthetics and land use planning
      - Costs relative to an available budget
      - Sustainability attributes
      - Contribution to achieving a green building rating
  - Requirements based on regulations and codes
    - System design and implementation must satisfy applicable code requirements
    - Codes can specify how design and implementation can occur

2.7

- Requirements based on treatment efficiency
  - Are designed to:
    - Protect public health and environmental quality
  - Can be based on:
    - Generic public health or environmental criteria or standards
    - Site-specific criteria and goals (e.g., by risk assessment)
  - Can be impacted by:
    - Other sources of pollutants within a given service area, e.g., area or watershed scale considerations (e.g., Total Maximum Daily Loads (TMDLs) in a watershed)
  - Can be stipulated by:
    - Regulations and codes

2.8

- Examples of criteria and standards for water quality that can impact treatment requirements are shown in Tables 2.2 and 2.3

Table 2.2 Example drinking water standards and criteria that could be used to set treatment efficiency requirements for a decentralized system discharging into the subsurface for groundwater recharge

Contaminant	MCL[a]	TT[b]	Reason
Nitrate (mg-N/L)	10	10	Infants below the age of 6 months who drink water containing nitrate in excess of the MCL could become seriously ill and, if untreated, may die. Symptoms include shortness of breath and blue-baby syndrome
Viruses (enteric)	Zero	TT	Gastrointestinal illness (e.g., diarrhea, vomiting, cramps)
Contaminant	**pH**	**Total dissolved solids**	**Iron**
Secondary guidelines[c]	6.5–8.5	500 mg/L	0.3 mg/L

Source: http://water.epa.gov/drink/contaminants/index.cfm#List.
[a]Maximum Contaminant Level (MCL)—The highest level of a contaminant that is allowed in drinking water.
[b]Treatment Technique—A required process intended to reduce the level of a contaminant in drinking water.
[c]Non-enforceable guidelines regulating contaminants that may cause cosmetic or aesthetic effects.

2.9

Table 2.3 Examples of state requirements for unrestricted urban water reuse that could be applied to effluent used for landscape irrigation (USEPA 2004)

Example state requirements for "unrestricted urban reuse"							
Reqt.	Arizona	California	Florida	Hawaii	Nevada	Texas	Washington
Treatment	Secondary, filtered, disinfected	Oxidized, coagulated, filtered, disinfected	Secondary, filtered, high-level disinfected	Oxidized, filtered, disinfected	Secondary, disinfected	NS[a]	Oxidized, coagulated, filtered, disinfected
BOD_5 (mg/L)	NS	NS	20 $cBOD_5$	NS	30	5	30
TSS (mg/L)	NS	NS	5	NS	NS	NS	30
Turbidity (NTU)	Avg. = 2 Max = 5	Avg. = 2 Max = 5	NS	Max = 2	NS	3	Avg. = 2 Max = 5
Total coli. ([b]/100 mL)	NS	Avg. = 2.2 Max. = 23	NS	NS	NS	NS	Avg. = 2.2 Max. = 23
Fecal coli. ([b]/100 mL)	Avg. = 0 Max. = 23	NS	75 % ND[b] Max. = 25	Avg. = 2.2 Max. = 23	Avg. = 2.2 Max. = 23	Avg. = 20 Max. = 75	NS

Note: For updated requirements and guidelines see USEPA 2012 Guidelines for Water Reuse.
[a]*NS* not specified in state regulations.
[b]None detected.

2.10

- Treatment efficiency requirements for a project can often lead to specifications
  - Example specifications for a system involving treated effluent dispersal into subsurface soil and local recharge of groundwater is presented in Table 2.4

Table 2.4 Example specifications for a decentralized system that disperses treated effluent into the subsurface with recharge of local groundwater

Specification parameter	Specification value
Critical parameters of concern	Nitrate nitrogen
Maximum allowable concentration	10 mg-N/L
Media and point at which requirements apply	Groundwater at the property boundary
Frequency and intensity of observation	Quarterly monitoring of three down gradient groundwater wells
Method of comparing data to requirements	Annual average < allowable concentration

2.11

- Treatment efficiency requirements can be met via prescriptive- or performance-based design
  - Prescriptive design—common; can be an implicit approach to achieving performance
    - Regulatory requirements *dictate* the steps and methods to be adhered to in system planning, design, and operation
    - Satisfactory performance is *presumed* to be achieved if the prescribed code requirements are met
  - Performance-based design—less common; is an explicit approach to achieving performance
    - Allows designers to develop solutions to achieve a numerical performance requirement (e.g., 10 mg-N/L)
    - Can provide for flexibility and innovation in design, but can require monitoring to verify performance

2.12

- Owner requirements also need to be met, including:
  - Requirements regarding system type and land use planning
  - Requirements based on affordability and cash flow
  - Requirements based on sustainability attributes
- Owner requirements regarding system type and land use
  - Owners can have specific desires related to the type of system and land use planning
    - Preferences for one system type over another
      - e.g., landscape-based system versus a reactor-based system (e.g., preference for a constructed wetland over an aerobic unit)
    - Preferences based on type and density of development
      - e.g., preference for larger individual lots vs. smaller lots with more open space in a clustered development

2.13

- Owner requirements based on affordability and cash flow
  - Owners can have a certain budget and financing arrangement that they have to work within
  - As a result, financial costs need to be estimated
  - Costs include projected capital costs plus operation and maintenance (O&M) costs
    - Capital costs = one-time costs to build the infrastructure
    - O&M costs = recurring annual costs
  - The estimated life cycle costs are computed from the amortized capital costs combined with annual O&M costs
  - Based on cash flow considerations, owners can have a preference for systems with low capital costs and higher O&M costs or vice versa

2.14

- Owner requirements based on sustainability attributes
  - Preferences for a system that is environmentally friendly—for example, owners may prefer a more passive natural system versus a more active energy consuming mechanical plant
  - Preferences for a system that helps achieve a certain sustainability rating
    - Green building rating systems, e.g.:
      - Building Research Establishment Environmental Assessment Method (BREEAM) Rating System (www.breeam.org)
      - Leadership in Energy & Environmental Design (LEED) (www.usgbc.org/certification)
    - Other rating systems include a focus beyond a single building to include varied infrastructure components, e.g.:
      - Envision® Sustainable Infrastructure Rating System (www.sustainableinfrastructure.org/rating/)

2.15

- In terms of water management, BREEAM, LEED, and other building rating systems are similar
  - Rating systems have many common elements:
    - Management of the construction process and sedimentation
    - Stormwater management for quantity control
    - Stormwater management for quality/pollution control
    - Landscape/irrigation water use reduction
    - Wastewater treatment, either onsite or by reducing offsite flow
    - Internal fixture water use reduction
    - Commissioning
    - Metering of water systems
  - The total points available for all credit categories related to water management range from about 12 to 18 %

2.16

- Requirements based on regulations and codes
    - Regulations and codes can impose varied requirements on system design and implementation
    - Regulations and codes include, but may not be limited to:
        - Building codes (plumbing, electrical,. . .)
        - Drinking water supply regulations
        - Wastewater treatment and discharge regulations
        - Reclaimed water use regulations
        - Stormwater regulations
        - Wetland regulations
        - Water rights regulations
    - Jurisdictions involved in administration of regulations and codes:
        - City and county
        - State and regional
        - Federal

2.17

- For U.S. regulations and codes specific to decentralized wastewater systems, system size and location are often important
    - Smaller individual sources are commonly regulated at the local level (typically at the county level)
        - Common size cut-off used to define “small” is 2000–5000 gal/day, but this varies from state to state
        - Common to have a state-wide code that sets minimum standards for small systems that are implemented at the local level
    - Larger individual sources, clusters, and small communities are often regulated at the state level
    - Potential options and design requirements can vary widely from state to state and from county to county within a state

2.18

- It is important to keep in mind that regulations and codes can be very conservative and constraining
  - Approaches and systems that appear technically viable for a particular application may, in fact, not be permitted under a particular regulation or code
  - This is particularly true for infrastructure to serve single-family homes and small businesses in rural areas
  - This also can be true for “nonconventional” options such as:
    - Source modification (e.g., urine diversion and recovery)
    - Innovative technologies (e.g., membrane units)
    - Reuse options (e.g., nonpotable reuse for toilet flushing)

2.19

- Conservatism often reflects current practical attributes of decentralized systems such as:
  - Highly varied and potentially changing wastewater flows and composition
  - Potential limitations on assuring that all needed operation and maintenance will be provided
  - Difficulties and costs to monitor performance of some components
    - Notably, natural treatment unit operations and systems
  - Difficulties in achieving corrective action if performance deficiencies do occur
- Fortunately, regulations and codes can evolve and become more contemporary and science-based

2.20

2-3. Treatment Performance

- Project requirements often include specified treatment efficiency targets such as:
  - Producing an effluent quality with BOD_5 and TSS < 30 mg/L
  - Achieving $\geq 50\,\%$ reduction in TIN with an effluent <10 mg-N/L
- Combinations of approaches and technologies can be configured into systems that can offer capability to achieve treatment efficiency targets
  - Compatible approaches and technologies can be selected based on their general capability to remove constituents of concern as described in Table 2.5 and illustrated in Fig. 2.1
  - Within a decentralized system, a treatment train consists of a sequence of compatible unit operations that connect the source of the wastewater to an intended discharge or reuse option as illustrated in Fig. 2.2

2.21

Table 2.5 Treatment levels based on unit operations and the constituents of concern removed

Level[a]	Description
Preliminary treatment	Processes and unit operations that are used to accomplish the initial processing of raw wastewaters generated in buildings, which often includes the removal of debris and fats, oils, and greases. Examples of preliminary treatment include grease interceptors, coarse screening units, and grinders
Primary treatment	Processes and unit operations that remove suspended solids (organic and inorganic) from wastewater by sedimentation or flotation processes. Advanced primary treatment includes some treatment of the separated solids (e.g., by anaerobic biodegradation of settled organic solids). Examples of primary treatment operations include settling basins, septic tanks, and upflow anaerobic sludge blanket reactors
Secondary treatment	Processes and unit operations that follow primary treatment and are designed to remove biodegradable dissolved and colloidal organic matter by aerobic biological processes. Advanced secondary treatment includes transformation and removal of nutrients (e.g., a nitrifying extended aeration bioreactor). Examples of secondary treatment operations include: extended aeration bioreactors, porous media biofilters, and constructed wetlands
Tertiary treatment	Processes and unit operations that typically follow secondary treatment and are designed to remove specific constituents such as nutrients, trace organic compounds, heavy metals or dissolved salts. Examples of tertiary treatment operations include: denitrifying porous media biofilters, adsorptive media packed bed reactors, and ion exchange columns
Disinfection	Processes and unit operations that are designed to destroy pathogenic microorganisms. Examples of disinfection technologies include: chlorination, ozonation, ultraviolet light irradiation, and membrane filtration. Natural disinfection can occur by separation, die-off, predation, and other processes in primary, secondary or tertiary treatment unit operations

[a]The definition of treatment levels can also include specific concentrations of constituents of concern (e.g., secondary treatment achieves 30 mg/L of BOD_5 and TSS) and treatment levels can also be defined in regulations in specific jurisdictions.

2.22

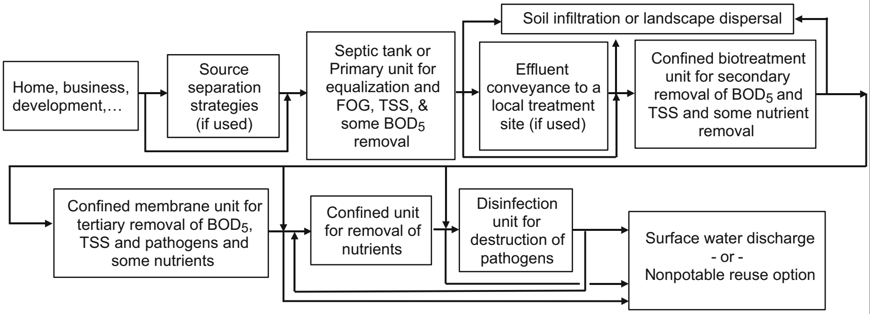

Fig. 2.1 Generalized sequence of strategies and unit operations that could be used to configure one or more viable decentralized systems

2.23

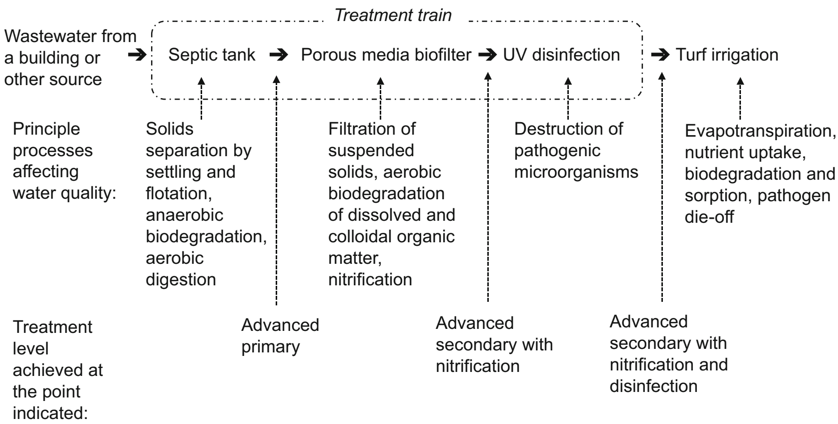

Fig. 2.2 Illustration of a treatment train within a decentralized system

2.24

- General and project-specific treatment efficiency capabilities of a particular unit operation or system
  - Different unit operations and systems have general treatment capabilities that a designer can be confident about, for example:
    - Capability of a septic tank to produce primary quality effluent
    - Capability of a recirculating sand filter to produce secondary effluent
    - Capability of a membrane bioreactor to produce tertiary quality effluent
  - The capability to meet specific expectations for a particular project can be less certain, for example:
    - Capability of a particular system to produce an effluent with BOD_5, TSS, and TIN ≤ 5 mg-N/L consistently (≥ 90 % of the time) when applied to the Mountain Pines Resort

2.25

- Establishing general treatment capabilities
  - General capability can be established through several means
    - For commonly used conventional unit operations and systems
      - Documentation of treatment capability often occurs through full-scale system field experiences and case histories
    - For innovative and alternative unit operations and systems
      - Documentation of treatment capability can occur through demonstration and testing, e.g.:
        - National Sanitation Foundation (NSF) testing (Table 2.6)
        - Testing centers (e.g., MASSTC at Buzzard's Bay)
        - USEPA Environmental Technology Verification projects
        - Research programs (e.g., CSM Small Flows)
        - State experimental system programs
  - Tabulated values of achievable treatment efficiencies (such as shown in this book) can thus be developed for different technologies and systems

2.26

Table 2.6 Examples of NSF testing and certification standards relevant to decentralized systems

NSF Standard number and description[a]
NSF/ANSI 40 is a standard for residential wastewater treatment systems with rated capacities between 400 and 1500 gal/day NSF can evaluate any kind of treatment system in test facilities in the U.S., Canada and Europe. To achieve certification, systems must produce an acceptable quality of effluent during a 6-month (26-week) test. Class I systems must achieve a 30-day average of 25 mg/L $CBOD_5$ and 30 mg/L TSS or less, and pH 6.0-9.0. System service and maintenance are prohibited during the test period
NSF Standard 41 certifies composting toilets and similar treatment systems that do not use a liquid saturated media as a primary means of storing or treating human excreta or human excreta mixed with other organic household materials. The standard requires a minimum of 6 months of performance testing, which includes design loading and stress testing appropriate to the product class: residential, cottage or day-use park. NSF evaluates a minimum of one system in a controlled laboratory setting, and a minimum of three systems in a mature field setting
NSF/ANSI Standard 245 defines total nitrogen reduction requirements to meet the growing demand for nutrient reduction in coastal areas and sensitive environments. NSF/ANSI 245 covers residential wastewater treatment systems with rated capacities between 400 and 1500 gal (1514 and 5678 L) per day We can evaluate any kind of system, regardless of treatment technology, in test facilities in the U.S., Canada and Europe
To achieve certification, treatment systems must produce an acceptable quality of effluent during a 6-month (26-week) test. System service and maintenance are prohibited during the test period
NSF/ANSI Standard 350 and 350-1 establish material, design, construction and performance requirements for onsite residential and commercial water reuse treatment systems. They also set water quality requirements for the reduction of chemical and microbiological contaminants for non-potable water use. Treated wastewater (i.e. treated effluent) can be used for restricted indoor water use, such as toilet and urinal flushing, and outdoor unrestricted water use, such as lawn irrigation

[a]The listing provided here is for illustrative purposes only and is not comprehensive. Comprehensive and current informatio can be obtained at the NSF website: http://www.nsf.org/services/by-industry/water-wastewater/onsite-wastewater/.

2.27

- Treatment efficiency actually achieved vs. general capabilities
  - Treatment efficiency actually achieved for a particular project can be better or worse than expected—why?
    - A system may be improperly designed and implemented
      - Incorrect estimates of design flows and pollutant loadings
      - Inaccurate design computations
      - Poor siting and construction
      - Inadequate startup and early operation
    - A system can be properly designed and implemented, but. . .
      - Conditions encountered during operation may be different than design assumptions, for example:
        - Differences in occupancy and daily flows
        - Differences in business functions and wastewater characteristics
      - Operation and maintenance may be inadequate, e.g.:
        - Failure to repair a malfunctioning pump or aerator

2.28

- Treatment achieved by each system or all systems?
  - Focusing on each and every system in a given area
    - Specifying that the treatment efficiency of each system must meet requirements (e.g., individual system average TIN $<$10 mg-N/L)
    - Could be the best approach where local risk is high, e.g.:
      - Contamination of groundwater used for drinking water
      - Aquatic toxicity from discharges to sensitive surface waters
  - Focusing on a population of systems in a given area
    - Specifying that the treatment efficiency of a population of systems must meet requirements (e.g., population wide average TIN $<$10 mg-N/L)
    - Could be the best approach where local risk is not high but there is a collective risk, e.g.:
      - In a watershed where there are concerns over cumulative effects on sensitive surface waters

2.29

2-4. Sustainability Attributes

- Sustainability of a system is often important to a project and this can be assessed with respect to:
  - Long-term, reliable treatment performance
  - Affordable costs for construction and operation
  - Acceptably low resource requirements and impacts
- Long-term, reliable treatment performance
  - A sustainable system should have predictable reliability during the design life of the project
  - System reliability is determined by proper system selection and design and implementation, including:
    - Use of redundancy and parallel treatment trains for larger flows
    - Provision of all requisite operation and maintenance
    - Use of on-line, real time monitoring and process control

2.30

- Affordable costs for construction and operation
  - A sustainable system must be affordable
  - Affordability includes one-time capital and recurring annual costs
    - Capital costs result from system design, construction and startup
    - Annual costs result from routine operation and planned, or otherwise required, maintenance activities
  - Economic analysis can be used to assess the net present value (NPV) of a particular system
    - NPV = capital costs plus amortized annual costs based on an assumed interest rate
  - In some situations, project owners can have a preference for a system with low capital costs and higher recurring annual costs even though the NPV could be higher for this latter system

2.31

- Acceptable resource requirements and impacts
  - A sustainable system must have acceptable resource attributes
  - Different systems typically have different resource attributes
    - Water—interior use by fixtures and appliances and exterior use for washing and irrigation functions
    - Nutrients—N, P, K are present at high levels in wastewaters
    - Energy—used to produce and deliver drinking water and collect and treat wastewater and used for heating water
    - Chemicals—used in consumer products for washing and cleaning, and for treatment of water and wastewaters
    - Materials—plastics, metals, and other materials used to construct water and sanitation systems
  - Resource attributes, including requirements, can be dependent on project goals and treatment efficiency requirements as illustrated in Fig. 2.3

2.32

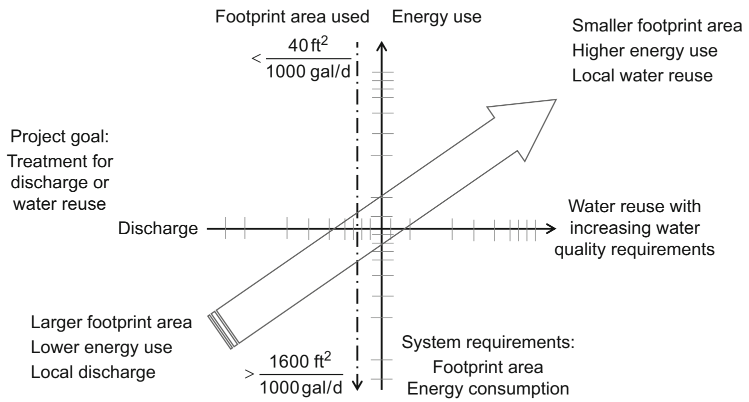

Fig. 2.3 Example of how footprint area and energy use can differ between systems designed for discharge versus water reuse

2.33

- More sustainable systems with respect to environmental impacts tend to minimize resource use directly or via recovery
  - More sustainable systems minimize environmental impacts due to resource use during wastewater collection and treatment
  - Systems can be configured that meet treatment efficiency needs and have relatively lower resource requirements
    - Different unit operations are listed in Table 2.7 along with their relative resource requirements for:
      - Land and site
      - Water
      - Power
      - Chemicals
      - O&M labor and materials
      - Capital and operating costs
  - Systems can also be configured to benefit from the resource attributes of water use efficiency and source separation approaches (Table 2.8)

2.34

Table 2.7 Example treatment unit operations and their relative resource requirements

Treatment unit operation	Resources and example relative requirements[a]					
	Land and site	Water	Power	Chemicals	O&M	Costs
Septic tank units	1	1	0	0	0	1
Aerobic treatment units	1	1	2	0	1	1, 2
Intermittent sand filters	2, 3	1	1	0	1	1, 2
Recirculating biofilters	1, 2	1	1	0	1	2
Membrane bioreactors	1	1	3	0	2	3+
Constructed wetlands	3	1	0, 1	0	0	2
Infiltration soil treatment units	3	1	0, 1	0	0	1
Landscape drip dispersal	3	1	1	0	0, 1	1, 2
Nutrient removal biofilter units	2	1	0, 1	0, 1	0, 1	1, 2
Disinfection units	1	1	1	0, 1	1	1, 2

[a]Numbers are given as examples only to indicate relative req.: 0 = none; 1 = low; 2 = moderate 3 = high.

2.35

Table 2.8 Resource attributes of water use efficiency and source separation approaches

Approach	Example resource use avoided directly or by recovery[a]				
	Water	Energy	Nutrients	Chemicals	Materials
Water efficient fixtures and appliances for flow reduction	3	1, 2	0	0, 1	0, 1
Source separation for urine diversion	0, 1	1	3	0, 1	0, 1
Source separation for graywater treatment, recycling or reuse	1, 2	0, 1	0, 1	0, 1	0, 1
In-source water recycling of reclaimed water	2, 3	0, 1	0, 1	2, 3	0, 1
Local water reuse of reclaimed water	2, 3	1, 2	2, 3	1, 2	1, 2

[a]Numbers are given as examples only and indicate relative degree of resource use avoided 0 = none; 1 = low; 2 = moderate; 3 = high.

2.36

2-5. Configuring Viable Systems

- Configuring viable systems requires thoughtful consideration of project goals and requirements
  - Many, if not most, projects involving decentralized infrastructure involve configuring a viable treatment train from available unit operations
    - Table 2.9 presents example treatment trains and systems that might be configured for a particular project goal and set of requirements
    - A generalized decision diagram is given in Fig. 2.4 and then used in Figs. 2.5, 2.6, 2.7, 2.8, and 2.9 to illustrate example treatment trains for different project goals and requirements
  - Source separation options within buildings can also enable system configurations that satisfy goals and requirements (Fig. 2.10)

2.37

Table 2.9 Examples of typical system configurations for common goals and requirements for projects serving homes or businesses

Project goals/requirements	Example treatment train and flow scheme
Treatment and safe subsurface discharge with a passive or low input system	Septic tank ➔ Subsurface soil infiltration ➔ Groundwater recharge
	Septic tank ➔ Packed bed biofilter or Aerobic treatment unit ➔ Landscape drip dispersal ➔ Evapotranspiration
	Septic tank ➔ Aerobic treatment ➔ Mounded soil infiltration ➔ Groundwater recharge
Treatment and safe discharge to the surface with a passive or low input system	Septic tank ➔ Constructed wetland ➔ Discharge to a bog or stream channel
Treatment and safe discharge to a sensitive surface water	Septic tank ➔ Packed bed biofilter or Aerobic treatment unit ➔ Chlorine or UV disinfection ➔ Discharge to a stream or river
	Septic tank ➔ Packed bed biofilter ➔ Denitrifying biofilter ➔ Chlorine or UV disinfection ➔ Discharge to a nutrient limited lake or estuary
Treatment and nonpotable reuse	Primary settling ➔ Membrane bioreactor ➔ UV disinfection ➔ Toilet flushing and turf irrigation

Note: The project goals/requirements and treatment trains shown are examples only.

2.38

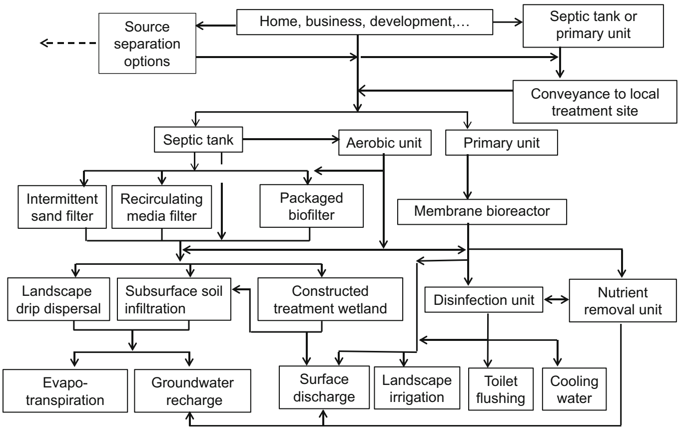

Fig. 2.4 Generalized decision support diagram to aid configuring viable decentralized systems

2.39

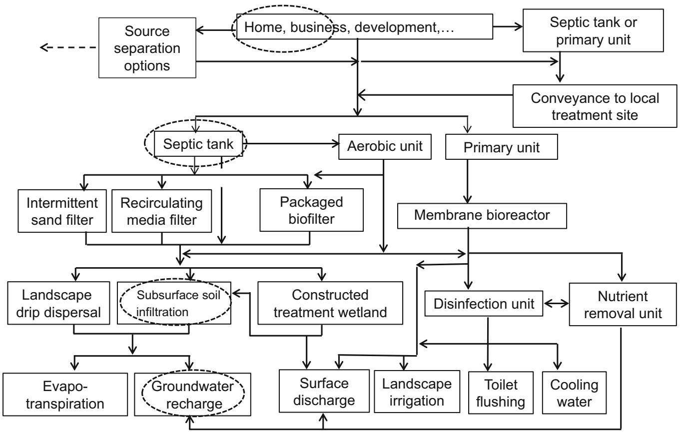

Fig. 2.5 Example system for a single source to provide passive treatment and discharge with limited cost and O&M needs

2.40

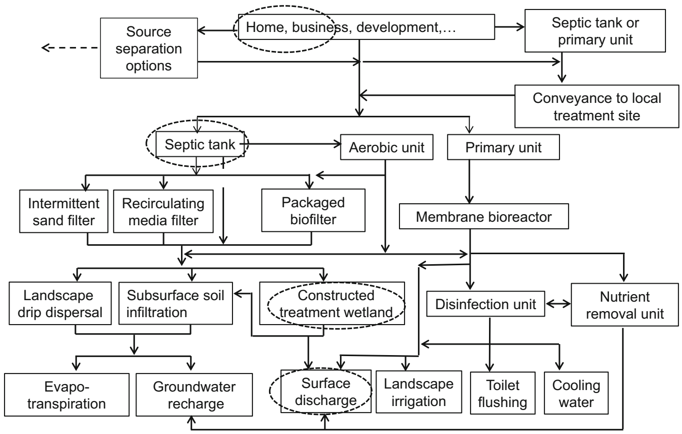

Fig. 2.6 Example system for a single source and passive treatment plus aesthetic benefits

2.41

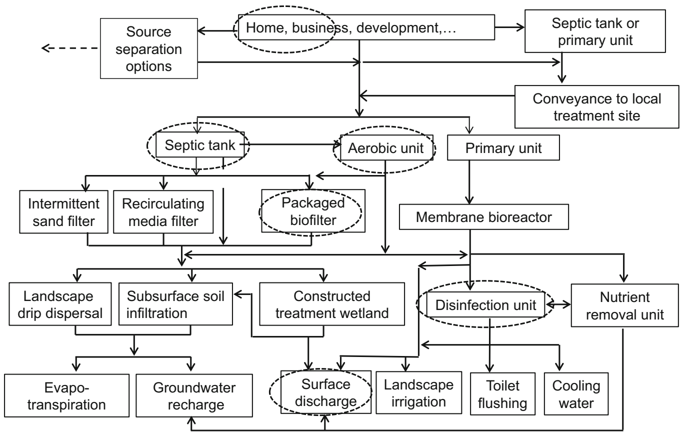

Fig. 2.7 Example of two optional systems for a single source where site conditions require secondary treatment (aerobic unit or packaged biofilter) with discharge to a stream

2.42

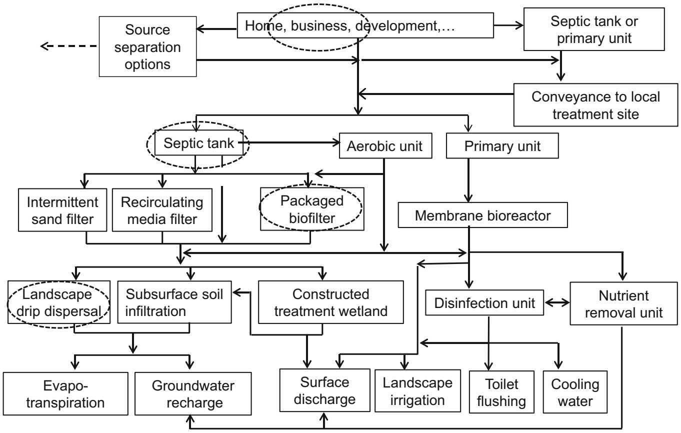

Fig. 2.8 Example system for a single source where treatment is needed to enable water and nutrient recovery by turf irrigation

2.43

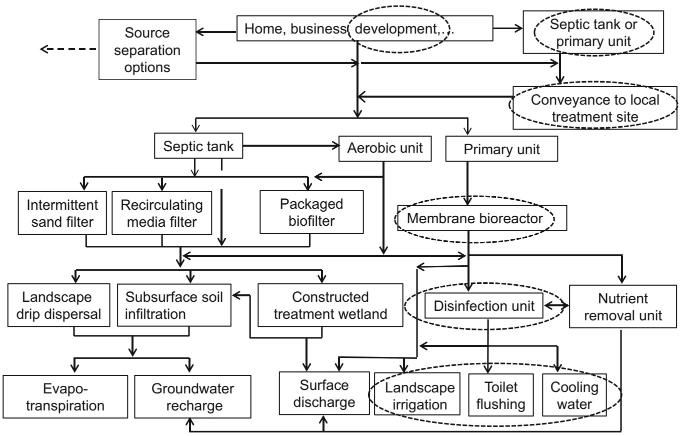

Fig. 2.9 Example system for a development where nonpotable reuse is desired with a high capacity treatment system that is not constrained by natural site conditions

2.44

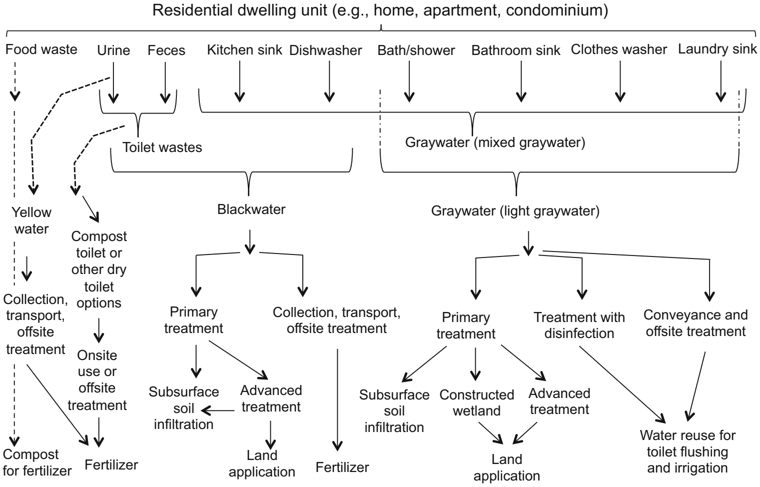

Fig. 2.10 Illustrative schematic of source separation options and associated treatment and reclamation alternatives for projects serving homes or businesses

2.45

2-6. Ensuring System Performance

- Unit operations and systems can have expected performance attributes
  - Designers configure systems which can be designed and implemented for a particular project
  - Proper design and quality construction and startup can be ensured through various means including:
    - Training and certification of designers and contractors
    - Design reviews and approvals
    - Construction supervision and inspections
  - For systems that are properly designed, constructed and started up, ensuring that the performance actually realized over the design life meets design expectations requires that there be:
    - Appropriate routine and reliable operation and maintenance
    - Appropriate monitoring and process control

2.46

- Operation and maintenance
  - O&M requirements vary in complexity and frequency, e.g.:
    - Inspections of system operations (e.g., flow readings, pumps and aerators functioning, etc.)—e.g., every 0.5–1 year
    - Pumping of residuals (e.g., from septic tanks)—e.g., every 5 years
    - Cleaning or replacement of media (e.g., in porous media biofilters)—e.g., every 10 years or more
  - O&M required versus complexity is illustrated in Fig. 2.11

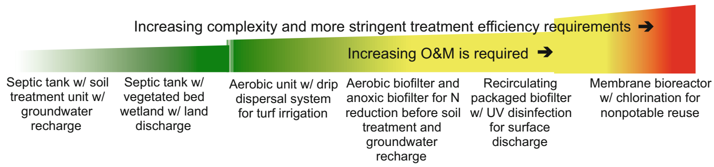

Fig. 2.11 Illustration of O&M requirements as a function of system complexity and treatment efficiency requirements of a particular project

2.47

- Monitoring
  - The need for, and importance of, monitoring depends on system complexity and the impacts of performance deficiencies should they occur (Fig. 2.12)
  - Monitoring methods can be used to determine the following:
    - Operational status of a unit operation or system, e.g.:
      * Is the circuit providing power to a dosing pump energized?
      * Is the lamp intensity in a UV disinfection system okay?
    - Treatment performance of a unit operation or system, e.g.:
      * Is the concentration of *E. coli* $\leq$ 10 org per 100 mL?
      * Is the total N in the groundwater $\leq$ 10 mg-N/L?
  - Examples of monitoring data to assess system operational status and performance are summarized in Tables 2.10 and 2.11

2.48

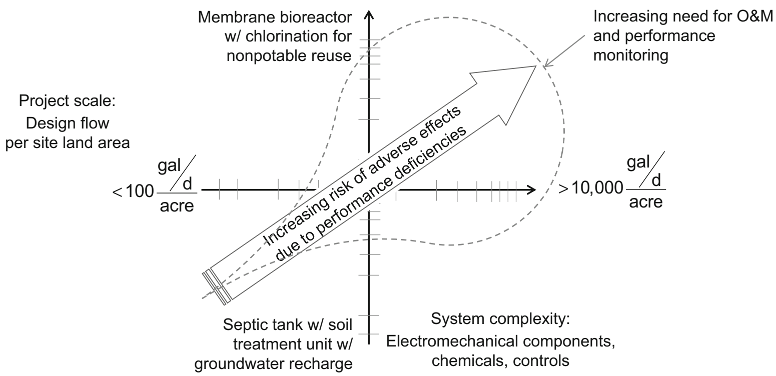

Fig. 2.12 Illustration of the interaction of project scale and system complexity in determining the need for O&M and monitoring to help assure inherent performance capability is realized

2.49

Table 2.10 Examples of data types that provide information on system operation

Data type	Data uses	Methods	Relative difficulty to obtain data
Power supply	Verify powered equipment is energized	Power meter	Low
Pressure	Provide insight into blockages in pipe-lines and piping networks or to liquid levels in tanks	Gages; Transducers	Low
Temperature	Provide insight into freezing conditions or rates of reactions	Thermocouples; thermometers	Low
Flow rate (daily)	Compare actual flow to design flows to determine if unit operation or system is under- or over-loaded hydraulically	Flow meters for continuous flows; Cycle counters on batch flow events	Low
Event cycles	Verify operation of batch events (e.g., siphon discharge, timed dosing events)	Mechanical or electrical counters	Low

2.50

Table 2.11 Examples of data types that provide information on system treatment performance

Data type		Data uses	Methods	Data type and difficulty and cost
Effluent water quality	A. Routine parameters (e.g., pH, alkalinity, BOD_5, TSS, N, P, Fecal coliforms)	Verify quality is consistent w/design values and/or permit reqt.	Grab sampling and analysis	A. Low B. Moderate–High
	B. Advanced analyses (e.g., virus, trace organics)		Composite sampling and analysis; in-line chemical sensors	A. Moderate–High B. High
Soil pore water and ground-water	A. Routine parameters (e.g., pH, alkalinity, DOC, BOD_5, TSS, N, P, *E. coli.*)	Verify land-based treatment units are performing per design and/or permit reqt.	Soil suction lysimeters or soil coring for sam-pling and analysis	A. High B. Very High
	B. Advanced analyses (e.g., virus, trace organics)		Groundwater probes or wells; sampling and analysis; in situ sensors	A. Moderate–High B. High–Very High
Surface water	A. Routine parameters (e.g., pH, alkalinity, DOC, BOD_5, TSS, N, P, *E. coli.*)	Verify absence of effects on surface water quality	Grab sampling and analysis	A. Low B. Moderate–High
	B. Advanced analyses (e.g., virus, trace organics)		Composite sampling and analysis; in situ chemical sensors	A. Moderate–High B. High

2.51

- Automated and real-time monitoring
  - This can be very important for decentralized infrastructure
  - Devices and technologies for data acquisition include:
    - In-line and in situ flow and chemical sensors
    - Alarms and control devices
    - Programmable logic controllers
    - Data loggers
    - Auto dialers
    - Telemetry systems
  - Supervisory control and data acquisition (SCADA) systems
    - SCADA systems can be used to gather and analyze real-time data to monitor and even control a unit operation or system

2.52

- Reasonable, appropriate and affordable monitoring
  - Operational and performance (O&P) data should provide information that is useful, if not, critical for decision making
  - O&P data should be reasonable and appropriate based on:
    * System configuration and complexity
    * Performance effects of operational departures from design
    * Risks associated with inadequate operation or performance
  - Adequate resources need to be allocated to generate sound O&P data for the intended information and decision-making purposes

2.53

2-7. Management Systems

- Management is essential to proper selection, design, and implementation of decentralized infrastructure
- What is meant by "management systems"?
  - Management systems involve entities and activities, often organized within a jurisdiction, to assure:
    - Decentralized systems are properly considered during infrastructure and land use planning, and
    - If selected they are properly designed, constructed, and operated so performance is satisfactory over a long-term planning period
  - As decentralized systems have evolved to become a permanent part of the wastewater infrastructure in the U.S., the need for, and critical role of, management has also evolved
    - According to USEPA (2002), "Management is the key to ensuring that the requisite level of environmental and public health protection for any given community is achieved."

2.54

- Characteristics of successful management programs
  - Successful programs often have key elements:
    - Clear and specific program goals
    - Public education and outreach
    - Technical guidelines for site evaluation, design, construction, O&M for conventional and alternative options
    - Regular system inspections, maintenance and monitoring
    - Licensing or certification of all service providers
    - Adequate legal authority, effective enforcement mechanisms, and compliance incentives
    - Adequate record management
    - Periodic program evaluations and revisions
    - Available funding mechanisms

2.55

- A successful management program may have multiple agencies or entities involved
  - Federal, state, and tribal agencies
  - Local governments: county, township, village, or city
  - Special-purpose districts and public utilities
  - Privately owned and operated management entities
- Regardless of the entities involved, a successful program will have elements that are:
  - Publicly accepted
  - Politically feasible
  - Fiscally viable
  - Measurable
  - Enforceable

2.56

- Management models for decentralized systems and different situations
  - USEPA (2003, 2005) developed five models that address different levels of risk determined by site- and system-specific considerations
    - The five management models include:
      * Owner awareness
      * Maintenance contracts
      * Operating permits
      * Responsible management entity (RME) O&M
      * RME ownership, O&M and management
    - "Risk factors" (Table 2.12) influence the type of management needed as illustrated in Fig. 2.13

2.57

Table 2.12 Risk categories and factors that influence the type of management system needed for a given project or jurisdiction (USEPA 2003, 2005)

Risk category	Example factors that can lead to higher risk conditions[a]
• Environmental sensitivity	• Impermeable soils such as heavy clay • Shallow depths to groundwater • Rock layers near the surface • Hilly terrain with thin soils and steep slopes • High densities of system installations • Sensitive waterbodies nearby
• Public health	• Drinking water wells nearby • Recreational waters nearby • Effluent surfacing or plumbing backups • Potential for rapid groundwater movement • Systems more than 25 years old not maintained • Illegal system discharges
• Wastewater characteristics	• Heavy sewage loads (high-strength wastewaters)
• Treatment complexity	• Industrial and certain commercial wastewaters • High fat, oil, and grease content in wastewater • Electrical and mechanical system components

[a]These risk categories and factors are focused on decentralized systems that utilize land-based treatmen operations including legacy leachfields, drainfields and soil absorption systems.

2.58

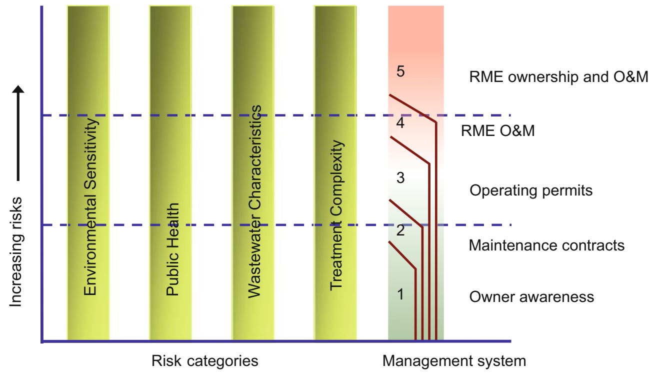

Fig. 2.13 Recommended management system based on increasing risks (USEPA 2003, 2005)

2.59

2-8. Summary

- Decentralized infrastructure is selected, designed and implemented for a particular project to satisfy project goals and requirements
- Achieving the performance capabilities of a unit operation or system requires proper design, construction and startup along with appropriate operation and monitoring
- Monitoring can help assess operation status and verify performance achievement, but it needs to be reasonable, appropriate, and affordable based on conditions and risks
- Management is essential to ensure proper selection, design, and implementation of decentralized systems that can and will achieve a desired performance in a sustainable manner

2.60

Chapter 3

Contemporary Water Use and Wastewater Generation

3-1. Scope

Knowledge of contemporary water use and wastewater generation is needed as a basis for engineering design of decentralized infrastructure. It is also needed to understand the benefits of water use efficiency and source separation as well as identify opportunities for recycling and reuse of reclaimed water. This chapter presents the characteristics of contemporary water use and wastewater generation in buildings within the United States and similar countries including residential (e.g., houses and other dwelling units) and nonresidential sources (e.g., commercial, institutional and recreational buildings and developments). Approaches for predicting contemporary water use and wastewater generation data are described.

3-2. Key Concepts

- Contemporary water use and wastewater generation is defined as the water use and wastewater generation that results from the typical activities and events associated with the use of a set of installed plumbing fixtures and appliances.
  - For example, for a building constructed in 1990 with traditional fixtures and appliances typical of that period, the contemporary water use and wastewater generation of that building in 2015 would be determined by the activities and events of 2015 that result in water use via the 1990-era fixtures and appliances.
  - Characterization data for residential and nonresidential buildings and developments (also known as sources) have been developed through monitoring studies completed over the past 40 years. Data

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in these sources can be used with one or more approaches to make predictions of water use and wastewater generation, which might be considered representative of a certain set of conditions. Chapter 3 covers this subject.
- As described in Chap. 4, water efficient fixtures and appliances and waste stream source separation approaches have evolved as sustainability concerns have grown and these approaches can dramatically modify contemporary water use and wastewater.

■ In the United States, comprehensive studies of residential water use were completed during the past 20 years. One of the national studies completed during the late 1990s revealed an indoor water use of about 69.3 gal/day per capita and 178 gal/day per dwelling unit (DU) (Mayer et al. 1999). A study to update the earlier data was being completed in 2015 and revealed daily indoor water use had dropped to about 138 gal/day per DU (DeOreo 2014). The decline was primarily due to the increasing use of more water-efficient toilets and clothes washers.

■ In a typical DU in the United States (e.g., house, apartment), 60–70 % of the indoor water use is normally due to toilet flushing, showers, and clothes washing. Hot water use amounts to 33 % of total indoor water use and about 70 % of the hot water use is attributed to faucet use and showers.

■ Indoor water use in a particular DU can vary with time (e.g., hour to hour, day to day, month to month) due to water using activities and events, which normally occur intermittently and variably over time. Variations in water use (and wastewater flow rates) that can occur at individual DUs are attenuated toward the average as the number of contributing DUs increases (e.g., cluster of 20 DUs vs. 1 DU).

■ Indoor water use in nonresidential sources varies widely within and among different categories of sources (e.g., hotels, restaurants, office buildings) due to business-specific operations.

■ Wastewater is generated by indoor water using activities and events and the flow and composition is determined by the water used and materials added during those activities and events. Wastewater composition data are important for treatment system selection and design to help ensure effective removal of constituents of concern and also help enable resource recovery and reuse.

■ In a typical residential dwelling, different activities and events contribute different amounts of pollutants and pathogenic microorganisms, e.g.:
- Toilet flushing contributes the most total suspended solids, nitrogen, phosphorus, and pathogenic microorganisms.

- Clothes washing, bathing, and sink use together contribute the most organic matter and BOD_5.
- Concentrations in wastewaters generated from a particular source can increase if substantial reductions in water use and wastewater flow rates occur due to installation of water efficient fixtures and appliances, but the mass discharged (e.g., lb/day) can remain unchanged.

■ Predicting wastewater flow and composition from houses and other residential sources can be done using existing or obtainable data (e.g., indoor water use per capita or per DU, census data for occupancy, number of dwelling units in a development) to achieve an estimate with a reasonable degree of accuracy.

■ For nonresidential sources, average wastewater flow rate and composition characteristics differ among different sources even within the same category (e.g., different restaurants). As a result, predictions of flow rate and composition are difficult to make with any certainty. Collection of relevant monitoring data from an existing or similar source can be very important for proper characterization and generation of necessary data.

■ Peaking factors are used to account for known higher-than-average flow rate conditions. For example, at an individual DU the peak day wastewater flow rate is typically about 2.5 times the average day flow rate.

■ In addition to wastewater generation due to indoor water use, infiltration and inflow of clean water (e.g., stormwater, groundwater) can increase daily wastewater flow rates from a source (e.g., residential subdivision or commercial development) by 10–20 % or more. Decentralized infrastructure can help reduce or even eliminate infiltration and inflow.

■ Factors of safety can be applied implicitly or explicitly to predictions of water use and wastewater flow rates and composition to account for uncertainty in estimates. Explicit factors of safety are preferred since they more clearly convey the magnitude of the factor of safety used.

3-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 3 are presented in the Slides section.

3-4. Terminology

Terminology introduced and used in Chap. 3 is defined below.

Appliance—A water-using piece of equipment that requires power to function properly (e.g., a dishwasher or clothes washer).

Blackwater—Wastewaters from water-flush toilets and potentially including wastewaters from kitchen sink and dishwasher uses.

Building—A structure (such as a house, school, restaurant, etc.) with a roof and walls that is used for a given purpose (e.g., living, working, storage, etc.).

Composition—The character and concentrations of dissolved, suspended and colloidal substances in water, the nature and degree of which determine the level of impairment of the water and its quality.

Constituent of concern (COC)—Constituents of concern include dissolved and suspended inorganic and organic substances and biological organisms that can cause undesirable human health effects or degraded environmental conditions under a given water reclamation plan for discharge or reuse.

Contemporary—A term used to describe the water use and wastewater generation characteristics of a dwelling unit or other building for a particular period of time (e.g., 1990s) based on the fixtures and appliances present and the water use behaviors typical of that period of time.

Dwelling unit (DU)—A single unit of residential occupancy for one person or one family such as a house, apartment unit, condominium unit, etc. A building can contain one dwelling unit (e.g., a house) or many (e.g., multiple apartments in a single building).

Excreta—Human urine and feces.

Factor of safety (FOS)—Factors of safety can be used to account for uncertain or unknown attributes, such as usage at a commercial establishment, while peaking factors account for known variability.

Fixture—A water-using piece of equipment that does not require power to function properly (e.g., a sink faucet or toilet).

Flow—(1) Water or wastewater liquid movement in a pipeline, basin or unit operation. (2) Water use or wastewater flow associated with use of an appliance or fixture in a building.

Graywater—Wastewaters produced by water use in basins, sinks and appliances in residential and nonresidential buildings. Mixed graywater includes food preparation related wastewaters (e.g., kitchen sink and dishwasher) while light graywater excludes food preparation wastewaters and possibly laundry wastewaters. All types of graywater exclude toilet wastewaters, which contain human excreta. Graywater can also be spelled as greywater.

Indoor water use—Water use that occurs through use of fixtures and appliances within a building. Indoor water use generates wastewaters. Indoor water use is also referred to as interior water use.

Infiltration and inflow (I&I)—Infiltration is due to groundwater seepage into conveyance piping and tankage through holes, cracks, joint failures, and faulty connections. Inflow is due to stormwater flow directly into conveyance piping or tankage via roof drain downspouts, foundation drains, storm drain cross-connections, and through holes in covers.

Maximum day—Compared to an average day at a particular building or development, the maximum day can be defined as one that recurs periodically (e.g., recurring maximum day that occurs 1 day every month) or very rarely (e.g., extreme maximum day that occurs only 1 day every year).

Nonresidential—Buildings that are used for purposes other than providing residency for day-to-day living by individuals or families. Buildings can be used for commercial, institutional, recreational, or other purposes. Examples of nonresidential buildings include: hotels, motels, restaurants, laundromats, schools, veterinary clinics, gasoline service stations, highway rest stops, and recreational park facilities.

Peaking factor—A multiplier used to estimate a peak flow rate compared to an average flow rate (e.g., maximum day flow compared to average day flow).

Quality—Quality is a qualitative term used to describe the degree of "impairment" of a water due to use and changes in composition (e.g., low vs. high quality).

Residential—Buildings that are used for individuals or families to live in over extended periods. Examples of residential buildings include single houses, apartments buildings, and condominium buildings.

Source—Source is defined as the origin of the wastewaters that are generated and will be treated for discharge or reuse. A source can include an individual dwelling unit, an apartment building, a cluster of dwelling units, a commercial or institutional building, a development of residential and/or commercial buildings, a portion of a city-wide service area, etc.

Source separation—In decentralized systems, refers to the separation and separate management of individual wastes and waste streams. For example using dual plumbing systems, blackwater comprised of toilet wastes and kitchen sink wastewaters can be separated from graywater produced by basins, other sinks, and appliances. Another example is the diversion of urine from fecal wastes using a urine-diverting toilet to enable urine processing and use as a fertilizer.

Toilet wastewater—Toilet wastewater consists of urine and feces plus toilet tissue.

Trace organic compounds—Refers to a group of organic compounds that can occur in wastewater and other impaired waters that are derived from biogenic substances, pharmaceuticals, consumer product chemicals, pesticides, and flame retardants. These compounds can be present at very low levels but still be constituents of concern. Trace organic compounds are sometimes referred to as organic micropollutants.

Wastewater—Wastewater consists of water plus materials added during water use. The types and concentrations of materials depend on the characteristics of the source (e.g., house, restaurant, school, veterinary clinic). Materials can include human excreta, foodstuffs, consumer products, pharmaceuticals and personal care products, heavy metals, silt, etc.

Water use efficiency—Water use efficiency can encompass water use conservation measures with traditional fixtures and appliances (e.g., showering less frequently and for a shorter duration) or water efficient fixtures and appliances (e.g., a toilet with a lower flush volume per use).

3-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 3 are listed below.

Ag	Silver
Avg	Average
AWWA	American Water Works Association
BOD	Biochemical oxygen demand
C&I	Commercial and institutional
Ca	Calcium
cap	Capita (or persons)
$cBOD_5$	Carbonaceous biochemical oxygen demand measured over 5 days
Cd	Cadmium
CFU	Colony forming units
CI	Confidence interval
COC	Constituent of concern
COD	Chemical oxygen demand
CSM	Colorado School of Mines
DO	Dissolved oxygen
DOC	Dissolved organic carbon
DU	Dwelling unit (e.g., homes, apartments, condominiums)
E. Coli	*Escherichia coli*
Fecal coli.	Fecal coliform bacteria
FOG	Fats, oils, and greases
FOS	Factor of safety
g	Gram
gal	Gallon
I&I	Infiltration and inflow
kg	Kilogram
L	Liter
min	Minute
MPN	Most probable number

N	Nitrogen
Na	Sodium
NH_4^+	Ammonium N
P	Phosphorus
P	Person (or capita)
Pb	Lead
PF	Peaking factor
PO_4^{-3}	Phosphate
REUWS1	Residential end uses of water study 1
REUWS2	Residential end uses of water study 2
SD	Standard deviation
SO_4^{-2}	Sulfate
SSWMP	Small Scale Waste Management Project
STE	Septic tank effluent
TKN	Total Kjeldahl nitrogen
TOC	Total organic carbon
TS	Total solids
TSS	Total suspended solids
TVS	Total volatile solids
TVSS	Total volatile suspended solids
U.S.	United States of America
UPC	Uniform Plumbing Code
USEPA	U.S. Environmental Protection Agency
WERF	Water Environment Research Foundation
WI	Wisconsin

C	Concentration of a particular constituent in the average daily flow
C_{RAW}	Average concentration in raw wastewater
C_{STE}	Average concentration in septic tank effluent
i	Different contributions (e.g., meals, guest toilet use, employee uses) within a particular source (e.g., motel)
j	Different sources contributing to the development flow being estimated, such as a motel, gas station, cafeteria, etc.
N_{BR}	Number of bedrooms (bedrooms in a dwelling unit)
N_{DU}	Number of dwelling units (e.g., homes, apartments, condominiums)
N_P	Household size (number of persons)
N_S	Number of a given unit of expression in a source (e.g., number of motel rooms,. . .)
N_U	Number of units (e.g., persons) causing a water-using event or activity during a given period (e.g., guests per motel room)
P_{BR}	Persons per bedroom
Q_A	

	Average indoor water use per household per day, average daily flow
Q_A/DU	Average DU flow when total flow is normalized to DUs contributing
Q_A/P	Per capita average daily flow rate (gal/day per capita)
Q_P	Peak flow rate
Q_U	Lumped flow rate per unit of expression (e.g., gal/day per guest), flow rate during an activity (e.g., 2.5 gal/min during showering)
R_A	Ratio of average concentration in raw wastewater to septic tank effluent
S1, S2, . . .	Source contributing to the development wastewater generation
T_U	Time used during an activity (e.g., 4 min per shower)
U_U	Uses per N_U per time period (e.g., one toilet flush per person per day)
V_U	Volume used per event (e.g., 3 gal per toilet flush)

3-6. Problems

3.1. For a development of 10 houses with an average occupancy of 3 persons per house, which of the following best represents the estimated average daily wastewater flow from the development: (1) about 70 gal/day, about 700 gal/day, or about 2000 gal/day?

3.2. If the average daily wastewater flow for a new residential development is estimated to be 10,000 gal/day, which one of the following best represents the likely maximum recurring daily flow: 10,000 gal/day, 25,000 gal/day, or 50,000 gal/day?

3.3. Estimate the maximum daily flow (Q_P in gallons/day) that you would anticipate from a four-bedroom house that will be built in a rural area just east of Denver. The house will be equipped with modern fixtures and appliances (but not minimum flow) and have a new onsite water supply well and decentralized wastewater treatment system.

3.4. The Mines Park student housing complex is located on the Colorado School of Mines campus in Golden, Colorado. The complex has 26 apartment buildings with the characteristics shown in the table below. Assuming all apartments have traditional water using fixtures and appliance typical of those used in the late 1990s (except there are no automatic clothes washers), estimate the average daily wastewater flow and the recurring maximum daily wastewater flow (Q_A and Q_P in gal/day for the combined 26 buildings) that you would anticipate to occur during the academic year (September to May).

Characteristics of the apartment buildings at the Mines Park student housing complex			
Building numbers	Apartment units per building	Bedrooms per apartment	Occupancy per apartment (persons)
1–6	8	1	1
7–12	8	2	3
13–20	8	2	2
21–26	8	3	4

3.5. For Problems 3.3 and 3.4, estimate the design daily wastewater flows based on requirements in Colorado Regulation 43—Onsite Wastewater Treatment System Regulation (see URL below). Briefly explain the possible reasons for any discrepancies in your estimates versus those made per Regulation 43 requirements. (https://www.colorado.gov/pacific/sites/default/files/Regulation-43.pdf)

3.6. A subdivision of 50 single-family homes is located near Denver, Colorado. The homes are all three bedroom dwelling units that were built in the 1980s with plumbing fixtures and appliances. What is the range in the individual house average indoor water use (gal/day per house) that you would estimate could occur in the group of 50 houses? What is the subdivision-wide average indoor water use that you would estimate on a dwelling unit basis (i.e., total flow for the subdivision divided by 50 houses)?

3.7. Alpine Meadows is a condominium complex that was built in the foothills near Golden, Colorado in the early 1980s. Alpine Meadows has a total of 32 dwelling units (DU) (4 DUs in each of 8 buildings) and each DU has two bedrooms and there are conventional plumbing fixtures and appliances. Estimate the average daily indoor water use and wastewater flow (Q_A in gal/day) and the peak daily flow (Q_P in gal/day) you would use during consideration of options for a new decentralized system to replace a failing legacy system that was installed when Alpine Meadows was constructed.

3.8. A retired couple with a year-round home in Morrison, Colorado is planning to build a rustic cabin in the mountains about 1 h away. The couple and occasional guests plan to use the cabin during the summer from about Memorial Day to Labor Day. The couple wants to keep the cabin very rustic. It will not have normal water using fixtures and appliances. Instead, the cabin will have a composting dry toilet. It will only have a spigot outside the back door, which delivers water to the cabin from an aboveground water holding cistern. Potable water will be delivered to the cistern by truck to provide safe water for drinking plus cooking, hand washing and tooth brushing. Bathing and laundry will be done as needed at their year-round home. How big a cistern (in gallons) would the cabin owners have to install so potable water delivered at the beginning of each summer would last all summer?

3.9. When compared to residential dwellings, it is much more difficult to predict design daily flow rates with any certainty for new commercial buildings and developments. Briefly explain why this is the case.

3.10. An existing highway rest area is located along Interstate Highway I-25 near the Colorado—Wyoming border. The rest area was built in 1989 to serve both north- and south-bound lanes and currently has the original water using fixtures. Estimate the average daily wastewater flow rate (Q_A) (in gal/day) that you would predict is generated from the rest area.

3.11. Estimate the average daily wastewater flow (Q_A in gal/day) that you would expect from a small commercial development located near Idaho Springs, Colorado that was built in 1993. The development has a 50-room motel, a small sit-down restaurant (30 seats) and a gasoline service station with restrooms.

3.12. You have been tasked with estimating the indoor water use characteristics and wastewater generation for a planned office building to be located just outside Denver. The four story building is to be occupied by an insurance company and will house 200 workers. There will be restrooms and drinking water fountains on every floor, a sink fixture in a lunchroom located on the second floor, and a men's and women's locker room with showers on the ground floor. Assuming water-efficient fixtures are used (e.g., 1.6 gal/flush toilets, 1 gal/use urinals, 2.5 gal/min showers, 2.5 gal/min sink faucets), provide an estimate of the total water use and wastewater flow that you would project for an average year (in gal per year).

3.13. Factors are used to account for known higher-than-average flow rate conditions. What is a reasonable peaking factor used to estimate maximum recurring daily flow at a complex of 24 condominium homes?

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Slides of Chapter 3

Decentralized Water Reclamation

Chapter 3: Contemporary Water Use and Wastewater Generation

Contents

3.1

3-1. Introduction

- Water use and wastewater generation components
  - Indoor (interior) water use occurs through activities and events within a building
  - Indoor water use generates wastewater
    - Wastewater = water + materials added during water use (Fig. 3.1)

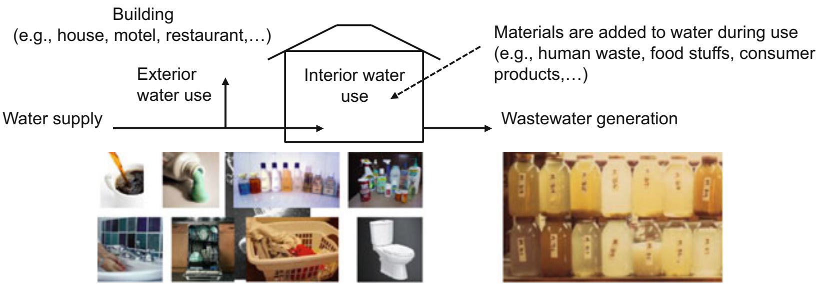


Fig. 3.1 Illustration of indoor water use activities and events leading to wastewater generation

3.2

- Buildings of interest for decentralized infrastructure
  - Residential buildings (Fig. 3.2)
    - Includes individual dwelling units or clusters of dwelling units, e.g.:
      - * A single-family house or single apartment
      - * A subdivision of houses or a building with multiple apartments or condominiums


Single-family house in an urbanized area


Single-family houses in a rural small town


Apartment building in a university housing development


High-rise condominium building in the center of a larger city

Fig. 3.2 Illustration of different types of residential buildings and situations

3.3

- Nonresidential buildings (Fig. 3.3)
  - Includes single buildings or developments of multiple buildings
    - * Buildings include commercial, institutional, and recreational, e.g.:
      - – Restaurants, motels, schools, churches, medical clinics, veterinary clinics, highway rest areas, picnic areas, etc.


Restaurant


Retail shops


Roadside convenience store


Church


School


Veterinary clinic

Fig. 3.3 Illustration of different types of nonresidential buildings and situations

3.4

- Water use and wastewater generation data are important for various reasons
  - Water use data can be needed to:
    - Assess water supply sources and their adequacy in quantitative terms
      - For example, assess rainwater capture and its viability to provide adequate water for specific fixture and appliance uses
    - Assess potential of, and benefits from, using water-efficient fixtures and appliances
      - For example, assess savings in daily water use by installing composting dry toilets in place of conventional water flush toilets
    - Assess and size systems involving water recycling and reuse
      - For example, to consider viability of using treated graywater for toilet flushing and how much equalization or backup fresh water might be required

3.5

- Wastewater flow and composition data can be needed to:
  - Assess options for waste minimization and resource recovery
  - Enable treatment and reuse system selection and design
    - Establish constituents of concern and treatment efficiency needs
    - Size individual unit operations (e.g., bioreactor, wetland) and select compatible unit operations for a complete system
      - Flow rates can determine:
        Hydraulic retention time in flow-through reactors
        Areal loading rates in filters and similar units
      - Composition can determine:
        Need for, type of, and extent of treatment required
        Pollutant and pathogen loads, which can impact system sizing
  - Assess pollutant and pathogen loads to water resources

3.6

- Wastewater composition and quality
  - Wastewater composition is quantified by a variety of measurable parameters as illustrated in Table 3.1
  - Quality is a term for the degree of "impairment" of a water due to use and changes in composition (e.g., low vs. high quality)

Table 3.1 Wastewater composition categories and example parameters

Category	Example parameters[a]
General water quality	Temperature, color, turbidity, pH, alkalinity, conductance
Common physical and chemical characteristics	Carbonaceous 5-day biochemical oxygen demand ($cBOD_5$), chemical oxygen demand (COD), total organic carbon (TOC), total solids (TS), total suspended solids (TSS), total Kjeldahl nitrogen (TKN), ammonium N (NH_4^+), nitrate N (NO_3^-), phosphate P (PO_4^{-3})
Commonly occurring microorganisms	Total and fecal coliform bacteria, *E. coli* bacteria
Other pollutants and pathogens occurring less frequently and/or at low concentrations	Virus (e.g., hepatitis A), heavy metals (e.g., Ag, Cd, Pb), trace organics (e.g., consumer product chemicals, pharmaceuticals)

[a]Some of the pollutants are also listed in the U.S. Clean Water Act required 303 and 305 state wate quality reports.

3.7

- Constituents of concern (COC) in a wastewater
  - COCs can be based on public health or environmental quality impacts
    - * Suspended solids—aesthetic concerns, anaerobic conditions
    - * Biodegradable organics—depletion of DO, aquatic life kills
    - * Pathogens—infectious waterborne disease transmission
    - * Nutrients—public health threat due to methemoglobinemia, eutrophication, DO depletion in receiving waters
    - * Heavy metals—can limit reuse potential
    - * Dissolved inorganics—Ca, Na, SO_4^{-2}, . . . can limit reuse potential
    - * Priority pollutants—can be carcinogens, mutagens, toxics
  - COCs can be based on adverse effects on treatment efficiency
    - * COCs can interfere with normal operations
    - * Certain COCs can limit treatability of other COCs

3.8

- Composition data—raw or treatment unit effluent
  - Composition data can be considered to help determine what unit processes might be exploited to remove COCs
  - Table 3.2 lists example COC groups and unit processes with potential for COC removal

Table 3.2 Example constituents of concern and unit processes for their removal

Example COC group	Example unit process
Suspended solids	Sedimentation
Fats, oils and greases	Flotation
Biodegradable organics	Biological transformation
Dissolved compounds	Sorption, precipitation, or ion exchange
Volatile organic compounds	Volatilization, or biological transformation
Heavy metals	Sorption, or precipitation and separation
Pathogenic microorganisms	Disinfection

3.9

3-2. Characterization Data

- Contemporary water use and wastewater generation
  - Contemporary water use and wastewater generation is defined as the water use and wastewater generation that results from the contemporary activity and events associated with the use of a set of installed plumbing fixtures and appliances
  - For example, for an existing building constructed in 1990 with fixtures and appliances typical of that period, the contemporary water use and wastewater generation of 2015 would be determined by the water-use behaviors typical of 2015 and the activities and events that occur using the 1990-era fixtures and appliances
  - Characterization data are used to describe water use and wastewater generation in residential and nonresidential buildings

3.10

- What is “normal” water use and wastewater generation?
  - Monitoring studies have yielded characterization data for what occurs under “normal” conditions at a particular type of building and what source- and time-dependent variations might reasonably be expected
  - But, most buildings were constructed during the 20th Century when there were not concerns over sustainability such as there are today
  - As a result, characterization data describing “normal” water use and wastewater generation often do not reflect what is, and can be, achieved under sustainability-constrained conditions
  - Chapter 3 presents characterization data and prediction methods for what might be considered “normal” under contemporary conditions while Chap. 4 describes what can be done to achieve more sustainable water use and wastewater generation

3.11

- Characterization data are available in literature publications (Table 3.3)

Table 3.3 Examples of water use and wastewater characterization data that can be obtained from published sources

Building	Data type	Examples of data available	Example references
Residential buildings—individual and multiple	Daily water use—total and/or indoor	gal/day per capita gal/day per dwelling unit	Siegrist et al. (1976), Mayer et al. (1999), Lowe et al. (2009), DeOreo (2014)
	Fixture and appliance usage frequencies and flow rates and volumes	Toilet flushes per day per capita Toilet use gal per day per capita Showering minutes per day per capita Showering gal per day per capita	Siegrist et al. (1976), Mayer et al. (1999), Coomes et al. (2010)), Rockaway et al. (2011)
	Daily wastewater flow and composition	Flow in gal/day per capita or per dwelling BOD_5, TSS, N, P,. . . in mg/L Bacteria, virus, parasites in organisms/L Consumer product chemicals in μg/L	Conn et al. (2006), Lowe et al. (2007), Stephens (2007), Lowe et al. (2009), Conn (2008), Dobbs et al. (2010)
Non-residential buildings	Daily water use—total and/or indoor	gal/day per patron, per seat, per room, etc.	Crews and Miller (1983), Dziegielewski et al. (2010)), UPC (2015)
	Daily wastewater flow and composition	Similar to residential data, but not as comprehensive or detailed	SSWMP (1978), Siegrist et al. (1985), Lowe et al. (2007), Conn (2008)

3.12

3-3. Water Use and Wastewater Flow

- Average daily water use for an individual dwelling unit (DU)
  - Characterization studies were completed during the past 40 years, including several focused on decentralized infrastructure
  - University of Wisconsin study
    - Statistical analysis of literature data and monitoring at 11 houses in WI
    - Major findings (Siegrist et al. 1976, SSWMP 1978)
      - * Daily indoor water use expressed on a per capita basis
        - – Average = 47.5 gal/day/cap; 95 % CI = 40.6–54.3 gal/day/cap
      - * Daily indoor water use expressed on a DU basis (Eq. 3.1)

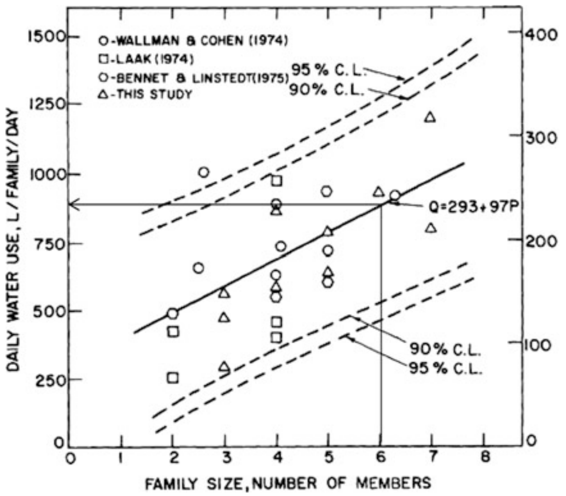
$$Q_A = 77.4 + 25.6N_P \tag{3.1}$$

Where:
Q_A = average indoor water use per household per day (gal/day)
N_P = household size (persons)

3.13

* A statistical analysis of literature data including monitoring at 11 houses in WI in the 1970s is shown in Fig. 3.4

Fig. 3.4 Relationship of household indoor water use to family size as measured in the 1970s (Siegrist et al. 1976, SSWMP 1978)



3.14

- American Water Works Association study
  - Residential End Uses of Water Study 1 (REUWS1) was completed at 1188 houses in 12 areas across the United States
    - Water use data were obtained by continuous data-logging during 2 weeks of summer and 2 weeks of winter at all houses during the period of May 1996 to March 1998
  - Major findings (Mayer et al. 1999)
    - Total water use and indoor water use on a DU basis (Table 3.4)
    - Indoor water use data for the 12 locations are shown in Table 3.5

Table 3.4 Water use data obtained through monitoring at 1188 houses in 12 areas across the United States in the 1990s (after Mayer et al. 1999)

Water use	No. of houses	Mean persons/house	Water use (gal/day per house)		
			Average	Median	SD
Total water use	1188	2.8	409	311	486
	95 % used <1000 gal/day and 75 % used <500 gal/day				
Interior water use	1188	2.8	173	157	94
	90 % used <300 gal/day and the maximum used was 769 gal/day				

3.15

Table 3.5 Indoor water use as a function of house location across the United States (Mayer et al. 1999, Table 5.1. Also reprinted as Table 3-2 in USEPA 2002)

Indoor water use by geographic location (ranked from lowest to highest)	No. of houses	Average persons/ house	Indoor water use (gal/day per person)		
			Average	Median	SD
Seattle, WA	99	2.8	57.1	54.0	28.6
San Diego, CA	100	2.7	58.3	54.1	23.4
Boulder, CO	100	2.4	64.7	60.3	25.8
Lompoc, CA	100	2.8	65.8	56.1	33.4
Tampa, FL	99	2.4	65.8	59.0	33.5
Walnut Valley Water District, CA	99	3.3	67.8	63.3	30.8
Denver, CO	99	2.7	69.3	64.9	35.0
Las Virgenes MW District, CA	100	3.1	69.6	61.0	38.6
Waterloo & Cambridge, ON	95	3.1	70.6	59.5	44.6
Phoenix, AZ	100	2.9	77.6	66.9	44.8
Tempe & Scottsdale, AZ	99	2.3	81.4	63.4	67.6
Eugene, OR	98	2.5	83.5	63.8	68.9
All dwelling units	1188	2.8	69.3	60.5	39.6

3.16

* Distribution of average indoor water use for each house expressed on a per capita basis is shown in Fig. 3.5

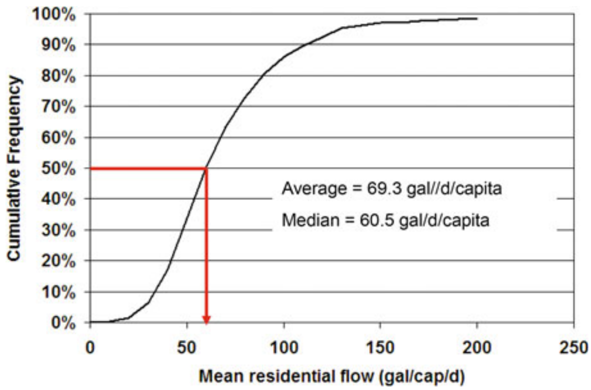


Fig. 3.5 Distribution of average indoor water use expressed on a per capita basis (prepared from the data of Mayer et al. (1999) collected at 1188 households at 12 different locations across the United States in the 1990s)

3.17

* Fixture and appliance utilization rates were determined as shown in Table 3.6.

Table 3.6 Average fixture and appliance utilization rates determined from monitoring 1188 houses in 12 study areas across the U.S. (Mayer et al. 1999)

Activity or event	Usage per person		
	Units	Average (range in averages)[a]	SD
Toilet flushing	No. per day	5.05 (4.49–5.62)	2.69
Showers and baths	No. per day	0.75 (0.63–0.90)	0.51
Clothes washing events	No. per day	0.37 (0.30–0.42)	0.24
Dishwashing events	No. per day	0.10 (0.06–0.13)	0.09
Faucet usage	Min. per day	8.1 (6.7–9.4)	5.3

[a]Range in the average values determined for each of the 12 study areas

3.18

* Estimating average household water use within a DU
  - An equation for representing indoor water use was developed by Mayer et al. (1999) using the data collected
  - Equation 3.2 includes a fixed amount of indoor water plus a contribution for each occupant in the house
  - Equation 3.2 is similar in form to that determined about 25 years earlier in the UW study (see Eq. 3.1)

$$Q_A = 69.2 + 37.2N_P \tag{3.2}$$

Where:
Q_A = indoor water use per household per day (gal/day)
N_P = household size (persons)
Note: Coeff. of determination (R^2) = 0.9944

3.19

- Water Research Foundation study
  - Residential End Uses of Water Study 2 (REUWS2) was initiated to update the results of the REUWS1 (DeOreo et al. 2016)
    * Data logging occurred at U.S. houses in nine locations and water use data and survey records were obtained at 16 others
  - Major findings (DeOreo 2014)
    * Key findings of REUWS2, including a comparison with REUWS1, are highlighted here
    * Total indoor water use per DU revealed that:
      - Average indoor water use was about 138 gal/day per DU
      - Indoor water use was ≤300 gal/day in 96 % of the DUs studied
      - Indoor water use at a DU as a function of DU residents follows a nonlinear relationship
    * Table 3.7 summarizes indoor water use by activity
      - Compared to REUWS1, indoor use was reduced by 22.7 %, mainly due to the use of high efficiency toilets and clothes washers

3.20

Table 3.7 Indoor water use contributions of different activities and events as measured in REUWS2 compared to the earlier REUWS1

Indoor water use by type	Contribution to average indoor water use (gal/day per DU)		
	REUWS1 (Mayer et al. 1999)	REUWS2 (DeOreo 2014)	Change (%)
Toilet use	45.2	33.1	−26.8
Clothes washer use	39.3	22.7	−42.2
Shower use	30.8	28.1	−8.8
Faucet use	26.7	26.3	−1.5
Bathtub use	3.2	3.6	+12.5
Dishwasher use	2.4	1.6	−33.3
Other uses	7.4	5.3	−28.4
Leaks	21.9	17.0	−22.4
Total	176.9	137.7	−22.2

3.21

* Of the total water indoor water use at a DU, REUWS2 data show the following use characteristics
  – 61 % (83.9 gal/day) of indoor water use is due three activities
    - Toilet = 33.1 gal/day per DU
    - Clothes washer = 22.7 gal/day per DU
    - Shower = 28.1 gal/day per DU
  – Hot water use amounts to 45.5 gal/day (33 % of total indoor water use) with the distribution as shown below:
    - Shower = 17.8 gal/day per DU
    - Faucets = 15.4 gal/day per DU
    - Clothes washer = 4.4 gal/day per DU
    - Bath = 2.6 gal/day per DU
    - Dishwasher = 2.2 gal/day per DU
    - Other = 0.9 gal/day per DU
    - Leaks = 2.1 gal/day per DU

3.22

- Colorado School of Mines study
  - Comprehensive literature review of data for single- and multiple-family residential units and some nonresidential sources (Lowe et al. 2007)
  - This was followed by monitoring of indoor water use at each of 16 individual DUs in Colorado, Florida and Minnesota (Lowe et al. 2009)
  - Major findings
    * Total indoor water use on a DU basis is shown in Table 3.8

Table 3.8 Water use data for houses in the United States based on a literature review and field monitoring (Lowe et al. 2007, 2009)

Project	Indoor water use (gal/day per house)		
	Average	Median	Range
Literature review of 30 studies focused on single-family and multi-family residential units (Lowe et al. 2007)	278	244	95–587
Monitoring at 16 homes in Colorado, Florida, and Minnesota (Lowe et al. 2009)	172	–	58.7–301 (1.18–2.30)[a] (1.47–10.2)[b]

[a]Range in the ratio of maximum day to average day at a particular house.
[b]Range in the ratio of the maximum day to the minimum day at a particular house.

3.23

* Distribution of average indoor water use for each house
  – The CSM analysis of the literature compared to a CSM monitoring study of 16 houses in three locations across the United States is shown in Fig. 3.6

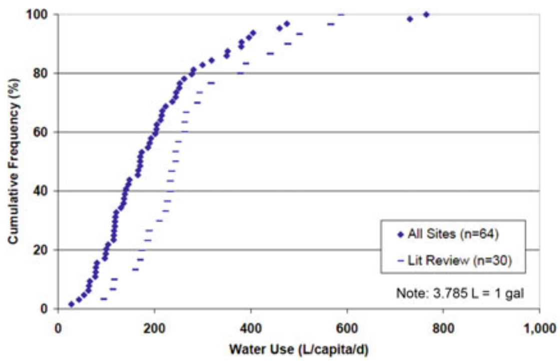


Fig. 3.6 Distribution of average indoor water use expressed on a per capita basis (Lowe et al. 2007, 2009)

3.24

- Water use contributions of fixtures and appliances
  - Indoor water use occurs via fixture and appliance use and leaks
    - Figure 3.7 shows batch vs. continuous flow activities and events

Fig. 3.7 Water use can occur as a batch event (e.g., toilet flush, clotheswasher or dishwasher use) or due to a flow rate over a period of usage (e.g., sink or shower use)

3.25

- Water use contributions of the fixtures and appliances commonly found in dwelling units have been determined in several studies
  - University of Wisconsin study (Siegrist et al. 1976)
    - Toilet use, bath/shower use, and automatic clothes washer use contributed 65–86 % of the total indoor water use
  - REUWS1 study (Mayer et al. 1999)
    - Toilet use, bath/shower use, and automatic clothes washer use contributed 67 % of the total indoor water use (Fig. 3.8)
  - WERF study (Rockaway et al. 2011)
    - Toilet use, automatic clothes washer use and shower use contributed about 60–65 % of total indoor water use (Table 3.9)
  - REUWS2 study (DeOreo 2014)
    - Toilet use, bath/shower use, and automatic clothes washer use contributed 61 % of the total indoor water use
    - Showers and faucets contribute 73 % of the total indoor hot water use

3.26

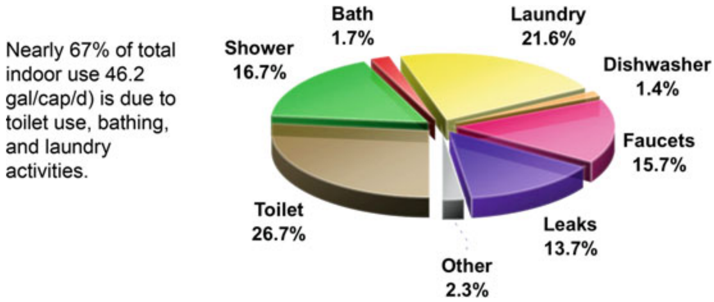


Fig. 3.8 Relative contribution to total indoor water use due to different activities based on an indoor water use study of 1188 houses located in nine locations across the United States (after Mayer et al. 1999)

3.27

Table 3.9 Average indoor water associated with usage of different fixture and appliances in a U.S. house (after Rockaway et al. 2011)

Source	Indoor water use (gal/day per DU and (% of total))								
	Total	Toilet use	Bath use	Shower use	Clothes washer use	Dishwasher use	Faucets	Leaks	Other
Mayer et al. (1999)	177 (100)	45.2 (25.5)	3.2 (1.8)	30.8 (17.4)	39.3 (22.2)	2.4 (1.4)	26.7 (15.1)	21.9 (12.4)	7.4 (4.2)
Denver water (2006)	155.6 (100)	38.6 (24.8)	2.9 (1.9)	28.9 (18.6)	31.5 (20.2)	2 (1.3)	21.5 (13.8)	24.5 (15.7)	5.7 (3.7)
Louisville water (2007)	151.6 (100)	37.5 (24.7)	3.1 (2.0)	18.4 (12.1)	32.4 (21.4)	1.9 (1.3)	20.5 (13.5)	33.8 (22.3)	4.0 (2.6)
Average of above studies	161.4 (100)	40.4 (25.0)	3.1 (1.9)	26.0 (16.1)	34.4 (21.3)	2.1 (1.3)	22.9 (14.2)	26.7 (16.5)	5.7 (3.5)

3.28

- Factors affecting indoor water use at a specific DU
  - Indoor water use at a specific DU depends on:
    - Type and features of plumbing fixtures and appliances present
      - Houses can have different types and numbers
      - Fixtures and appliances can have different water use attributes
    - Water use behavior of occupants
      - Individuals and families can have very different frequencies and durations of water using activities and events
    - Water use studies have identified the key factors that can affect the fixtures and appliances present and water use behaviors
      - Family size and demographics
      - Occupation(s) and residency attributes
      - Socioeconomic status
      - Local weather and climate

3.29

- Results from two studies are highlighted below
  - Lowe et al. (2009) examined the potential effects of geographic location, season, and age on daily water use
    - Geographic location and season did not have a significant effect on indoor water use
    - But, the age of residents in the DU did have a significant effect on indoor water use
    - DUs with residents older than 65 years of age had a significantly higher per capita daily water use (average = 297 and SD = 177 gal/day/capita) compared to those with residents under 65 years of age (average = 148 and SD = 78 gal/day/capita)

3.30

- In the REUWS2 study (DeOreo 2014) a number of factors were found to impact indoor water use at a DU:
  - Number of persons residing at the house
  - Number of teenagers and number of children
  - Parcel size (proxy for income level)
  - Adults employed outside of the house
  - Number of persons home during the day
  - Sewer rate
  - Presence of high efficiency toilets and clothes washers
  - Presence of hot water recirculation systems

3.31

- Changes in DU indoor water use have occurred over time
  - Table 3.10 illustrates how indoor water use in DUs has declined during the past 20 years, largely due to increased use of water efficient fixtures and appliances

Table 3.10 Recent studies that have revealed declines in indoor water use

Project	Average Indoor water use (gal/day per house)		
	1990	2007	Change
WERF study (Coomes et al. 2010, Rockaway et al. 2011). Change in indoor water use at 65 households in the Louisville, Kentucky area, after adjusting for weather, demographics, and housing variables	208	187	−19 (−9.1 %)
	(2.52)[a]	(2.38)[a]	−0.14 (−5.6 %)[a]
	The majority of the decline was attributed to increased use of low-flow fixtures and appliances		
REUWS2 (DeOreo 2014) compared to REUWS1 (Mayer et al. 1999). Comparison of residential fixture and appliance utilization and water use flow rate data during the 2010s versus the 1990s (Table 3.7)	**1990s**	**2010s**	**Change**
	177.9	137.7	−40.2 (−22.6 %)
	The majority of the decline (29.7 gal/day) was due to increased use of higher efficiency toilets and clothes washers. (*Note:* average occupancy did not decline significantly from REUWS1 to REUWS2)		

[a]Average occupancy level at each house.

3.32

- DU water use rates are a function of the period of interest
  - Examples of water use flow rates are listed below and the ratio of the respective maximum to average rate is illustrated in Fig. 3.9
    - Instantaneous use at a particular time—0–10 gal/min
    - Hourly use throughout the day—0–100 gal/h per DU
    - Daily use during the week—0–500 gal/day per DU
    - Weekly use over a year—0–1800 gal/week per DU
    - Annual use over a decade—45,000–75,000 gal/year per DU

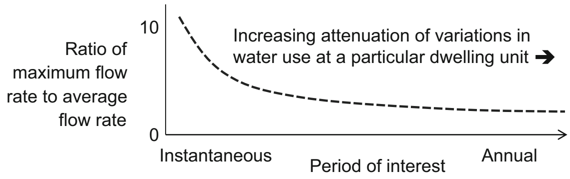


Fig. 3.9 Ratio of maximum rates to average rates as a function of flow rate of interest

3.33

- Wastewater flow rates from an individual DU
  - Wastewater flows are often approximated by indoor water use
  - But, wastewater flow rates may not equal indoor water use rates
    - Wastewater flow rates can be higher, for example:
      - Average daily wastewater flow rates can be higher if rain-water capture and use generates wastewaters
      - Discharge rates from batch events can be higher than water use rates that occur during filling the fixture or appliance (e.g., filling a clothes washer or refilling a toilet tank)
    - Wastewater flow rates can be lower, for example:
      - Average daily wastewater flow rates can be lower if some interior water use does not result in wastewater (e.g., using laundry water for watering plants and grass)
      - Instantaneous rates can be lower since the piping network in a building plumbing system can attenuate discharges from individual fixtures and appliances

3.34

- Average daily wastewater flow rates from individual DUs
  - Flow rates can be inferred from indoor water use or directly measured in building sewers (Table 3.11)
    - Note that just like indoor water use, wastewater flow rates vary between DUs and over time at a given DU

Table 3.11 Average daily wastewater flow rates from individual dwelling units

Study	Study features	Study average daily flow (gal/day per capita)		
		Average	Median	SD
U.S. EPA (1980)	Based on multiple studies of water use or wastewater flows conducted in the 1970s	45.6	40	No data
Mayer et al. (1999)	Indoor water usage measured at each of 1188 houses in 12 areas in CO, CA, OR, WA, AZ, FL, Ontario in 1996–1998	69.3	60.5	39.6
Lowe et al. (2009)	Building sewer flows measured at each of 17 houses in CO, MN, FL in 2007–2008	54.7	45.2	37.8

3.35

- Water use and wastewater flow from a cluster of DUs
  - As the number of DUs in a cluster increases, daily water use and wastewater flow rates can be attenuated (Fig. 3.10)

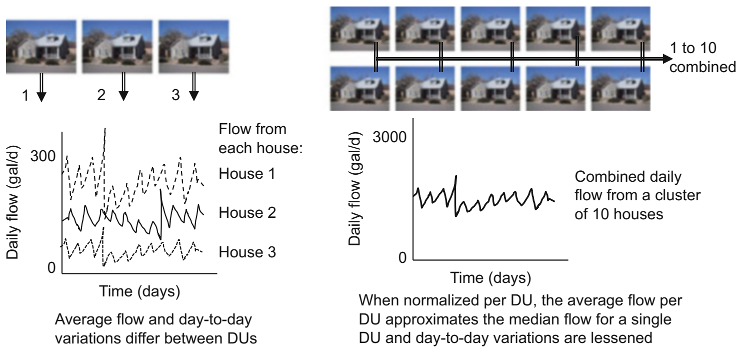


Fig. 3.10 Illustration of the daily wastewater flow rates from three individual houses (*left*) versus the combined flow from a cluster of ten houses (*right*)

3.36

- Probability of occurrence of a daily wastewater flow rate from a cluster of houses depends on the number of houses (Fig. 3.11)

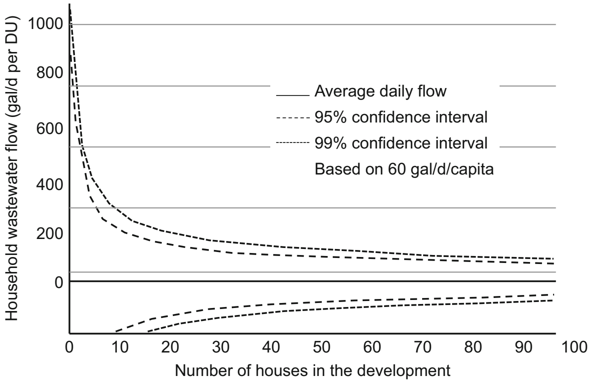


Fig. 3.11 Probability of occurrence of an average daily flow from a development of houses declines as the number of houses contributing increases (Stephens 2007)

3.37

■ Water use in nonresidential buildings

- Water using activities and events
  - ○ Water using activities and events can be similar to those in dwelling units or can be very different due to business-specific operations
  - ○ Events and activities can vary widely among similar as well as between different types of nonresidential buildings, for example:
    - * A coin laundry has clothes washing and some toilet and sink use
    - * A highway rest area has mostly toilet use and some sink use
    - * All restaurants have food preparation, dishwashing, and toilet use but little or no bathing or laundry, but. . .
      - – A busy restaurant will have more daily water use than one that has less business, and
      - – A fast food sandwich restaurant may have different water uses compared to a fish house sit-down restaurant

3.38

- Characterizing normal water use within nonresidential buildings and sources is difficult
  - Developing characterization data for normal water use in nonresidential buildings and other sources is much more complicated and uncertain compared to doing so for residential buildings like DUs
  - Business-specific operations can vary widely within a single category of nonresidential buildings (e.g., hotels) and there can be variable user and customer attributes (e.g., employees or patrons per day)
  - Units of expression have attempted to capture this variability within nonresidential buildings and sources by expressing water use in different business-specific units, for example:
    * gal/day per patron, per seat, per meal served, per ft^2 of floor area, etc.
  - Despite the challenges, studies have been completed to characterize water use in prevalent types of nonresidential buildings and sources

3.39

- Two major studies of commercial and institutional (C&I) sources have been completed in the United States
  - Corps of Engineers study (Crews and Miller 1983)
    * Developed a library of water use coefficients for different commercial, institutional and industrial sources (Table 3.12)
  - AWWA Research Foundation study (Dziegielewski et al. 2010)
    * Water use data were obtained by two methods
      – Auditing billing records for C&I customers in urban areas across the United States
      – Continuous data-logging in 25 buildings in five urban areas of the United States (Table 3.13)

3.40

Table 3.12 Average daily water use rates from commercial and institutional developments (Crews and Miller 1983)

Type	Unit	gal/day per unit	Type	Unit	gal/day per unit
Barber shops	Chairs	54.6	Drive-in movies	Car stall	5.33
Beauty shops	Station	269	Nursing homes	Bed	133
Bus/rail depots	ft^2	3.33	New office buildings	ft^2	0.19
Car washes	Inside ft^2	4.78	Old office buildings	ft^2	0.14
Churches	Member	0.14	Jails and prisons	Person	133
Golf/swim clubs	Member	22.20	Restaurants	Seat	24.2
Bowling alleys	Alley	133	Drive-in restaurants	Car stall	109
Residential colleges	Student	106	Night clubs	Person served	1.33
Hospitals	Bed	346	Retail space	Sale ft^2	0.11
Hotels	ft^2	0.26	Elementary schools	Student	3.83
Laundromats	ft^2	2.17	High schools	Student	8.02
Laundry	ft^2	0.25	YMCA/YWCA	Person	33.3
Medical offices	ft^2	0.62	Service stations	Inside ft^2	0.25
Motels	ft^2	0.22	Theaters	Seat	3.33

3.41

Table 3.13 Average daily water use characteristics for five common commercial and institutional categories (Dziegielewski et al. 2010)

CI category	Unit of measure	Audit data percentiles[a]				Field monitoring data[b]
		No.	10 %	50 %	90 %	
Supermarket	gal/year per ft^2	33	17.3	33.3	63.6	40.3, 33.7, 25.3, 20.0, 16.4
Office	gal/year per ft^2	74	3.9	14.2	45.5	9.7, 22.9, 13.5, 40, 4.1
Restaurant	gal/year per ft^2 gal/day per employee gal per meal served gal/day per seat	87	110 73.6 5.8 19.5	306 145.6 11.2 32.5	768 532 35.5 73.8	 2.7, 10.5, 5.4, 16.2, 3.4 9.7, 51.2, 31.3, 17.3, 30.8
Hotel	gal/year per ft^2 gal/day per room gal/day per occupied room	100	17.4 55 81.3	72.6 116.8 163.4	206.5 187.9 271.0	103, 227, 110, 139, 114
School	gal/year per ft^2 gal/student per calendar d gal/student per school d gal/year per student or staff	139	9.1 3.3 5.9	24.4 6.4 11.5	57.0 13.7 24.3	6.7, 19.3, 8.1, –, 10.3 566, 1273, 1203, –, 740

[a]Water use billing records were analyzed for: 33 supermarkets and grocery stores from 18 cities in two states (California Arizona); 74 office buildings in 27 cities across four states (Arizona, California, Colorado, Florida); 87 restaurants fron 38 cities in three states (California, Florida, Colorado); 100 hotels and motels from 39 cities in four states (Arizona California, Colorado, Florida); and 139 schools from 35 cities in four states (Arizona, California, Colorado, Florida).
[b]Water use monitoring was completed at one building of each type in each of five urban areas: Irvine, Los Angeles, Phoenix San Diego, and Santa Monica.

3.42

- Wastewater flow rates from nonresidential buildings
  - Wastewater flow rates are often roughly estimated based on indoor water use data
  - However, as noted for residential buildings, wastewater flow rates may not be the same as indoor water use rates
  - In addition, in some nonresidential buildings or sources, water using activities and events can result in wastewater flows that can be higher than the water use data would suggest
    - For example, wastewater flow rates from restroom usage at sports arenas or highway rest areas may exceed water use rates
      - Wastewater flow rates can exceed water use rates due to the addition of human wastes (e.g., urine) to the water use associated with urinal and toilet use

3.43

3-4. Wastewater Composition

- Residential buildings and wastewater composition
  - Contributions of water-using activities and events (Table 3.14)

Table 3.14 Average per capita pollutant contributions (grams/cap/day) by individual activities and events in DUs in the United States (Siegrist et al. 1976)

Parameter	Toilet flush		Garbage disposal use	Kitchen sink use	Dishwasher use	Clothes washer use	Bath/ shower use
	Feces	Urine					
BOD_5	4.34	6.38	10.9	8.34	12.6	14.8	3.09
$BOD_{5\ filtered}$	2.34	3.98	2.57	4.58	7.84	9.81	1.87
TOC	3.53	4.25	7.32	5.00	7.28	10.3	1.75
$TOC_{filtered}$	1.58	3.17	3.91	4.11	4.69	7.29	1.13
TS	10.7	17.8	25.8	13.8	18.2	48.4	4.59
TVS	7.76	12.0	24.0	9.73	10.5	19.5	3.60
TSS	6.24	6.28	15.8	4.11	5.27	11.0	2.26
TVSS	5.09	5.12	13.5	3.84	4.46	6.51	1.58
Total N	1.50	2.64	0.63	0.42	0.49	0.73	0.31
NH_3-N	0.59	0.52	.01	0.03	0.05	0.03	0.04
NO_3-N	0.01	0.02	0.00	0.00	0.00	0.03	0.01
Total P	0.27	0.28	0.13	0.42	0.82	2.15	0.04
Ortho P	0.12	0.19	0.09	0.18	0.38	0.52	0.02

3.44

- Wastewater composition after flows from fixtures and appliances are combined in a drainage plumbing system (Tables 3.15, 3.16, and 3.17)

Table 3.15 Characteristics of wastewaters generated in DUs in the United States (Lowe et al. 2009)

Constituent	Units	Median	Range	Constituent (units)[a]	Mean (range)
Alkalinity	mg-$CaCO_3$/L	260	65–575	Oil and grease	18
TS	mg/L	1028	252–3320	(mg/L)	10–109
TSS	mg/L	232	22–1690	Fecal Coliforms	4.93×10^{5}[b]
$cBOD_5$	mg/L	420	112–1101	(MPN per 100 mL)	1.0×10^4–1.73×10^8
COD	mg/L	849	139–4584	*E. coli*.[a]	8.09×10^4
TOC	mg/L	184	35–738	(MPN per 100 mL)	1.0×10^4–8.16×10^7
DOC	mg/L	110	29–679	Consumer product chemicals (e.g., caffeine, nonylphenol, triclosan) (μg/L)	Frequently detected but at low (μg/L) levels
Total N	mg-N/L	60	9–240		
Kjeldahl N	mg-N/L	57	16–248		
Ammonium N	mg-N/L	14	2–94	Pharmaceuticals, pesticides, flame retardants (ng/L)	A few detected at very low levels (e.g., ibuprofen, naproxen, salicylic acid)
Nitrate N	mg-N/L	1.9	BDL-9		
Total P	mg-P/L	10.4	0.2–32		

[a]Value given is the geometric mean.

3.45

Table 3.16 Comparison of properties commonly of interest in wastewaters generated in DUs in the United States (Lowe et al. 2009)

Constituent	Units	Lowe et al. (2009)		USEPA (2002)	C&T (1998)
		Median	Range	Range	Range
Alkalinity	mg-$CaCO_3$/L	260	65–575	Not rept.	Not rept.
TS	mg/L	1028	252–3320	500–880	350–1200
TSS	mg/L	232	22–1690	155–330	100–350
$cBOD_5$	mg/L	420	112–1101	155–286	110–400
COD	mg/L	849	139–4584	500–660	250–1000
TOC	mg/L	184	35–738	Not rept.	80–290
DOC	mg/L	110	29–679	Not rept.	Not rept.
Total N	mg-N/L	60	9–240	26–75	20–85
Kjeldahl N	mg-N/L	57	16–248	Not rept.	Not rept.
Ammonium N	mg-N/L	14	2–94	4–13	12–50
Nitrate N	mg-N/L	1.9	BDL-9	<1	0
Total P	mg-P/L	10.4	0.2–32	6–12	4–15

USEPA = U.S. Environmental Prot. Agency, *C&T* = Crites and Tchobanoglous.

3.46

Table 3.17 Microorganisms including pathogens that are commonly present in wastewaters generated in DUs in the United States (from Lowe et al. 2007)

Type	Organism	Range (MPN per 100 mL)
Bacteria	Total coliform	10^7–10^{10}
	Fecal coliform	10^6–10^8
	Clostridium perfringens	10^3–10^5
	Enterococci	10^4–10^5
	Fecal streptococci	10^4–10^6
	Pseudomonas aeruginosa	10^3–10^4
	Shigella	10^0–10^3
	Salmonella	10^2–10^4
Virus	Enteric virus	10^3–10^4
	Coliphage	10^3–10^4
Protozoa	*Cryptosporidium parvum* oocysts	10^1–10^4
	Entamoeba histolytica cysts	10^{-1}–10^3
	Giardia lamblia cysts	10^3–10^4

3.47

- Nonresidential buildings and wastewater composition
  - If the water-using activities and events are similar, the water use and wastewater characteristics can be similar to those in residential applications (e.g., toilet flushing, sink use, laundry, etc.)
  - But more often, the composition of wastewaters from nonresidential buildings is very different from that of residential buildings
    - It can be more or less concentrated in pollutants, e.g.:
      - Restaurant wastewater BOD_5 can exceed 1000 mg/L
      - Nonresidential buildings typically have higher N and P levels; e.g., school wastewater total N can exceed 125 mg-N/L
    - It can contain different types of pollutants, e.g.:
      - Commercial and institutional wastewaters can have higher levels of consumer product chemicals and pharmaceuticals
  - Composition data for wastewaters generated from nonresidential buildings are highlighted in Table 3.18 and Figs. 3.12 and 3.13

3.48

Table 3.18 Wastewater composition determined through monitoring at 14 commercial and institutional sites in Colorado (Conn 2008)

Constituent	Units	Average	Median	Minimum	Maximum	Number
Alkalinity	mg-$CaCO_3$/L	390	410	20	75	40
pH	–	6.80	6.78	4.92	8.69	40
$cBOD_5$	mg/L	430	320	80	1200	27
TOC	mg/L	100	89	33	340	25
DOC	mg/L	87	77	21	230	25
Total N	mg-N/L	100	92	6	190	25
Ammonium N	mg-N/L	99	87	4	210	26
Nitrate N	mg-N/L	1.9	1.4	<0.5	9.5	24
Total P	mg-P/L	17	16	1.7	37	26
Fecal coliforms	CFU per 100 mL	4.19×10^6	6.75×10^5	1.50×10^5	3.34×10^7	12

Notes: Nonresidential source types include commercial (two restaurants, one bakery, two convenience stores, three retail) and institutional (two schools, one church, three veterinary hospitals). Grab samples were taken at the inlet to the septic tanks at each site. Each site was sampled three times.

3.49

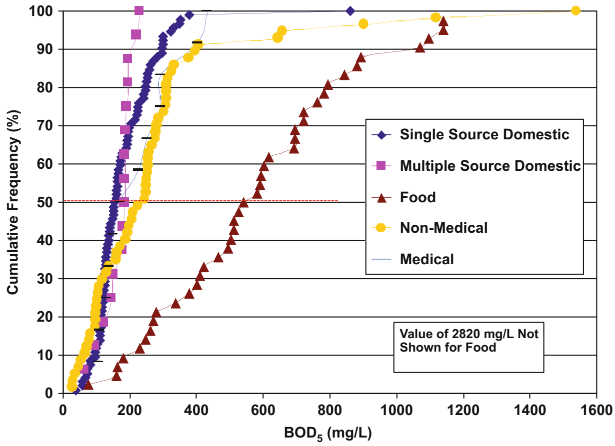


Fig. 3.12 Illustration of how wastewater concentrations are on average higher (e.g., compare medians) and can vary more widely (compare slopes) for nonresidential buildings compared to residential sources (Lowe et al. 2007)

3.50

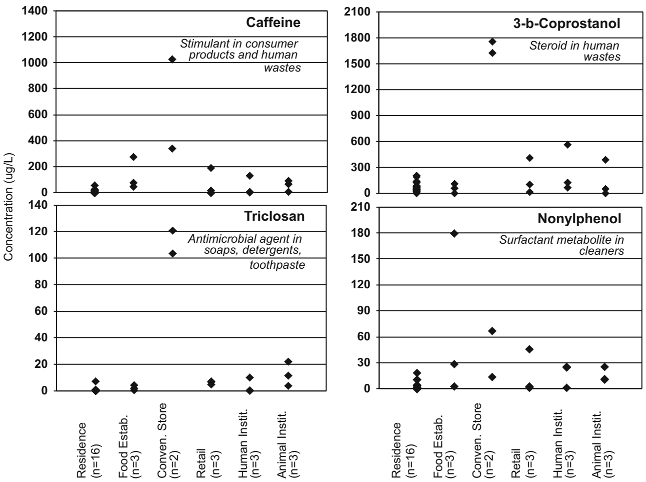


Fig. 3.13 Consumer product chemicals in wastewaters from nonresidential compared to residential sources (Conn et al. 2006) (n = no. of sources)

3.51

3-5. Predicting Flow and Composition

- Approaches to predicting flow and composition
  - Predicting flow rate and composition data involves consideration of the answers to several questions (Table 3.19)
    - What is the purpose and intended use of the predictions made?
    - What is the type and status of the building or development for which predictions are to be made?
    - What are the sources of data available for use in making predictions?
    - What are the limitations of extrapolating flow rate and composition data from one source to another?
    - What regulations might stipulate how predictions are to be made?
  - Figure 3.14 illustrates a generalized decision diagram and Table 3.20 presents a summary of the approaches available for predicting flow and composition

3.52

Table 3.19 Key questions that need to be addressed when developing an approach to use for predicting flow and composition data

Questions	Description
What is the purpose and intended use of the estimates?	• This can affect the type of data being estimated and help guide the selection of appropriate input data for calculations, for example • For sizing a treatment unit, estimates can be needed for the maximum daily flow (gal/day) and influent concentrations of BOD_5 and TSS (mg/L) • For sizing a pump, estimates of instantaneous peak flow (gal/min) are often needed.
What is the type and status of the building or development?	• Types can span a single house, a cluster of houses, an apartment building, a commercial building, shopping center, etcetera • Status concerns whether the building or development already exists or if it is one that is planned but not yet constructed
What are the sources of data for use in estimation?	• For existing buildings and developments, monitoring data can be acquired: • Interior water use records, or newly obtained data, can often be used to estimate flow rates • Wastewater flow and composition data for an existing treatment system might be available already, or could be readily obtainable • Composition data can sometimes be estimated based relevant published characterization data • For new buildings and developments, estimates require assumptions and external data sources

(continued)

3.53

Table 3.19 (continued)

Questions	Description
Extrapolation of flow and composition data from one source to another?	• Extrapolation of flow and composition data from one source to another can be uncertain, the magnitude of which depends on the circumstances. For single-family homes and multifamily residential buildings, estimates can be fairly accurate. For clusters of homes and small communities estimates can be accurate also. But depending on system characteristics and conditions, infiltration and inflow into sewer systems may need to be accounted for and this can be uncertain. For commercial and institutional buildings and facilities, estimates of water use and wastewater flow can be highly uncertain. Estimates of wastewater composition, particularly for unusual constituents like consumer product chemicals, can be very uncertain
What regulations might stipulate how estimates are to be made?	• Conservative procedures are often used to make estimates which can result in very conservative data (e.g., the estimated daily flow $\gg$ than the likely daily flow)

3.54

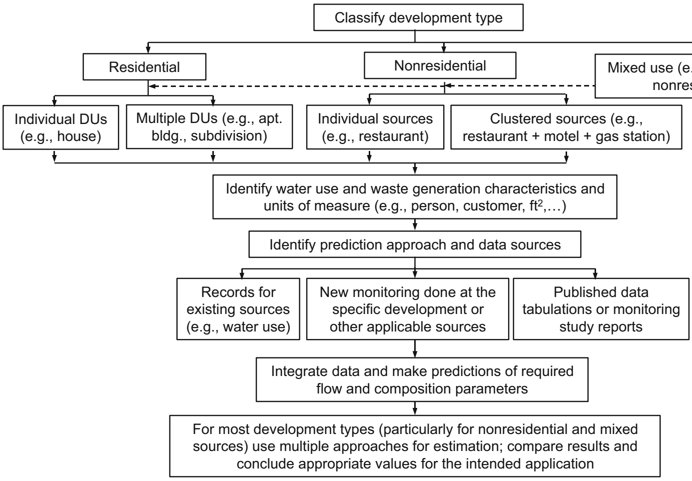


Fig. 3.14 Generalized decision diagram for predicting flow and composition characteristics

3.55

Table 3.20 Summary of approaches available for predicting flow and composition

Approach	Description	Additional data needed
1. Monitoring	Monitoring data collected from the project site or a similar residential or nonresidential source near the project site	Depends on monitoring data available
2. Use of published data for a specific type of source	Published data can include: (1) tabulations in reference texts or manuals providing per capita or per unit flow rates and concentration data or (2) results of characterization studies completed at specific sites	Estimates of population contributing for DUs (e.g., three persons per home) or unit of measures for nonresidential sources (e.g., customers/day)
3. Extrapolation from residential DUs to nonresidential buildings	Apply flow rates and COC loads from comparable water-using activities and events that occur in residential DUs to nonresidential applications (e.g., toilet flushing, sink use, laundry, etc.)	Estimates of water-using activity and event usage
4. Extrapolation of composition data generated in DUs or specific nonresidential buildings to mixed use developments	Identify and apply relevant concentration data from DUs or nonresidential sources so a flow-weighted mass balance can be used to estimate concentrations in the total development flow	Estimates of the water use or wastewater flows from each source in the development

3.56

Predicting average daily flow rates—Single DU

- For a single DU, the average daily indoor water use or wastewater flow can be estimated using Eqs. 3.2–3.4 and information in Table 3.21

$$Q_A = 69.2 + 37.2N_P \tag{3.2}$$

$$Q_A = \left(\frac{Q_A}{P}\right)(N_P) \tag{3.3}$$

$$Q_A = \left(\frac{Q_A}{P}\right)(P_{BR})(N_{BR}) \tag{3.4}$$

Where:
Q_A = indoor water use per DU per day (gal/day per DU)
N_P = household size (persons in a dwelling unit)
Q_A/P = per capita average daily flow rate (gal/day per capita)
P = person (or capita)
P_{BR} = persons per bedroom
N_{BR} = number of bedrooms (bedrooms in a dwelling unit)

3.57

Table 3.21 Summary of sources of information for parameters included in Eqs. 3.2–3.4

Parameter	Example sources of information	Example data
$\frac{Q_A}{P}$	• Data from published studies (e.g., Mayer et al. 1999, Lowe et al. 2009, DeOreo 2014) • Monitoring of local DUs	The average daily per capita flow for an average house in a large population of houses = 69.3 gal/cap/day. The average daily flow for a single house with high daily flow could be much higher (e.g., 221 gal/cap/day)
N_P	• Local knowledge of the DU • Census data (e.g., Census Bureau 2013)	For a single-family DU, U.S. Census Bureau data reveal a National avg. $N_P = 2.51$, while 60 % of DU have ≤2 and 98.5 % have ≤6
P_{BR}	• Local knowledge of the DU • Census data for area • Assumptions	For a single-family DU, U.S. Census Bureau data reveal 74.4 % of DU have $P_{BR} \leq 1.0$
N_{BR}	• Local knowledge of the DU • Census data for area	For a single-family DU, U.S. Census Bureau data reveal 48 % of DU have $N_{BR} \leq 2$ BR

Note: monitoring of individual DUs in the vicinity of the project site is an option but one that is unlikely to be used for a single DU.

3.58

- Occupancy data is available in census data (Fig. 3.15)
- A comparison of average daily flow rates estimated for an individual DU using different approaches and input parameter values is shown in Table 3.22

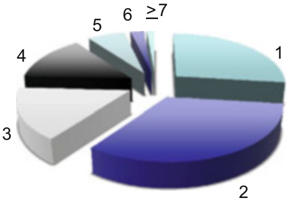


Fig. 3.15 Number of persons occupying individual dwelling units in the United States based on census data (data from Table 2-9 in U.S. Census Bureau, American Housing Survey 2009)

3.59

Table 3.22 Comparison of DU average daily flow rate predictions made using different approaches

Approach	Basis for estimate: $\frac{Q_A}{P}$ (gal/day/cap)	N_P	P_{BR}	N_{BR}	Q_A (gal/day per DU)
Average Q_A for a population of DU based on Eq. 3.2	n/a	2.51 (U.S. Natl. avg.)	n/a	n/a	163
Average Q_A for a population of DU based on literature data	Q_A for a DU as measured by Mayer et al. (1999) or Lowe et al. (2009)				173, 172
Average Q_A for a population of DU based on Eq. 3.4 with census data and literature data	69.3	n/a	1.0	2	139
Extreme Q_A for a single DU based on Eq. 3.2	69.3	6 (98.4 % of U.S. DU)	n/a	n/a	292
Extreme Q_A for a single DU based on literature data	Q_A for a DU + 2 SD as measured by Mayer et al. (1999) or Lowe et al. (2009), or the 96 %-tile of DUs as measured by DeOreo (2014)				361, 362 300

3.60

- Predicting average daily flow rates—Multiple DUs
  - For a residential building or development with multiple DUs, the average daily water use or wastewater flow can be estimated using Eq. 3.5
    - The values for the input parameters in Eq. 3.5 need to be carefully selected (or estimated) based on the nature and size of the development (Table 3.23)
    - It is very important that the attenuating effects of clustering are accounted for when estimating average daily flow rates

$$Q_A = \left(\frac{Q_A}{DU}\right)(N_{DU}) \tag{3.5}$$

Where:
Q_A = average daily flow for a development (units of gal/day or similar)
Q_A/DU = average DU flow when total flow is normalized to DUs contributing
DU = dwelling unit (e.g., homes, apartments, condominiums)
N_{DU} = number of dwelling units (e.g., homes, apartments, condominiums)

3.61

Table 3.23 Summary of sources of information for parameters included in Eq. 3.5

Parameter	Example sources of information	Example data
$\frac{Q_A}{DU}$	Published data from indoor water use and wastewater flow studies that provide data on the average DU flow rate based on the total daily flow from a development of multiple DUs normalized by the number of DUs contributing (e.g., Stephens 2007)	Based on development monitoring in Michigan (Stephens 2007), the grand average of the average DU flow rates observed in each of 14 developments was 158 gal/day per DU (SD = 58 gal/day per DU). Based on DU water use reported by DeOreo (2014), the average water use was about 138 gal/day and 96 % of DUs had $Q_A \leq 300$ gal/day
	Probabilistic modeling of the likelihood of concurrent flow rates from multiple DUs (e.g., Dobbs et al. 2010)	Based on modeling by Dobbs et al. (2010) to limit risk of exceedance of a design flow to 1 %, Q_A/DU = 250 gal/day per DU for systems with >15 DU, 225 gal/day per DU for >30 DUs, 200 gal/day per DU for very large developments
	Calculations of the average Q_A/DU for a population of DUs	Average values for Q_A/DU range from 139 to 173 gal/day per DU (Table 3.22).
N_{DU}	Local knowledge of a development	Depends on development attributes

3.62

- Predicting average daily flow rates—Nonresidential
  - For nonresidential buildings or developments, the total average daily flow can be estimated based on two approaches
    - Approach 1—Based on contributions per lumped unit of expression

$$Q_A = \sum_{j=1}^{n} \left\{ \sum_{i=1}^{n} [(N_S)(N_U)(Q_U)]_i \right\}_j \tag{3.6}$$

Where:
Q_A = average daily flow for a development (in gal/day or similar)
N_S = number of a given unit of expression in a source (e.g., number of motel rooms,. . .)
N_U = number of units (e.g., persons) causing a water-using event or activity during a given period (e.g., guests per motel room)
Q_U = lumped flow rate per unit of expression (e.g., gal/day per guest)
i = different contributions (e.g., meals, guest toilet use, employee uses) within a particular source (e.g., motel)
j = different sources contributing to the development flow being estimated, such as a motel, gas station, cafeteria, etc.

3.63

- Approach 2—Based on water using events and activities

$$Q_A = \sum_{j=1}^{n} \left\{ \sum_{i=1}^{n} [(N_U)(U_U)(V_U) + (N_U)(U_U)(Q_U)(T_U)]_i \right\}_j \tag{3.7}$$

Where:
Q_A = average daily flow for a development (e.g., gal/day or similar)
N_U = number of persons or fixtures and appliances causing an event or activity (e.g., 100 persons using a bathroom)
U_U = uses per N_U per time period (e.g., 1 toilet flush per person per day)
V_U = volume used per event (e.g., 3 gal per toilet flush)
Q_U = flow rate during an activity (e.g., 2.5 gal/min during showering)
T_U = time used during an activity (e.g., 4 min per shower)
i = different contributions (e.g., meals, guest toilet use, employee uses) within a particular source (e.g., motel)
j = different sources contributing to the development flow being estimated, such as a motel, gas station, cafeteria, etc.

3.64

- Sources of information for input into Eqs. 3.6 and 3.7
  - For some nonresidential sources, values for fixture and appliance water use can be extrapolated from published sources (e.g., Mayer et al. 1999, Dziegielewski et al. 2010)
  - For common nonresidential sources, tabulations of water use and waste flows have been published (e.g., Table 3.24)

Table 3.24 Examples of water use and wastewater flow estimates for different nonresidential buildings and sources (UPC 2015)

Type	Unit	gal/day per unit
1. Airports	Employee; passenger	15 5
2. Auto washers	Station	
3. Bowling alleys	Lane	75[a]
4. Camps: Campgrounds—with flush toilets, no showers Day camps—no meals	 Person Person	 25 15
5. Churches: Sanctuary only With kitchen	 Seat Seat	 5 7
6. Dance halls	Person	5
7. Factories No showers With showers With cafeteria, add	 Employee Employee Employee	 25 35 5
8. Hospitals Kitchen waste only Laundry waste only	Bed Bed Bed	250 25 40
9. Hotels (no kitchen waste)	Bed	60

3.65

Type	Unit	gal/day per unit
10. Institutions (resident) Nursing home Rest home	Person Person Person	75 125 125
11. Laundries (self-serv.)	Wash cycle	50
12. Motel With kitchen	Bed space Bed space	50 60
13. Offices	Employee	20
14. Parks, mobile homes Picnic parks (toilets only) Recreational vehicles: Without water hookup With water and sewer	Space Space Space Space	250 20 75 100
15. Restaurants—cafeterias Toilet Kitchen waste Add for garbage disp. Add for cocktail lounge Kitchen waste—dis. serv.	Employee Customer Meal Meal Customer Meal	20 7 6 1 2 2
16. Schools—staff and office Elementary students Intermediate and high With gym & showers, add With cafeteria, add Boarding, total waste	Person Student Student Student Student Person	20 15 20 5 3 100
17. Service station, toilets First bay Addl. bays	 Bay Bay	 1000 500
18. Stores Public restrooms, add	Employee 10 ft^2 floor space	20 1
19. Swimming pools, public	Person	10
20. Theaters, auditoriums Drive-ins	Seat Space	5 10

[a]Check with manufacturer.

3.66

- Predicting peak flow rates—residential or nonresidential
  - For some purposes, peak flow rate data are needed
    - Examples of peak flow rate data used in design include:
      - * Recurring maximum daily flow (gal/day)
        - – This could be the highest daily flow rate that occurs somewhat routinely (e.g., once every 30 days)
      - * Extreme maximum daily flow (gal/day)
        - – This could be estimated such that it would be highly unlikely for it to be exceeded
        - – For example, this could be the highest daily flow rate that occurs very infrequently (e.g., only once in 300 days or more)
      - * Instantaneous peak flow (gal/min)
        - – This could be the highest discharge rate possible from a source (e.g., house, school, restaurant)

3.67

- Peaking factors for predicting peak flow rates
  - Peaking factors have commonly been used for estimating peak flow rates such as maximum daily or hourly flow rates
  - Equation 3.8 presents the simplified form for applying a peaking factor

$$Q_P = (PF)(Q_A) \tag{3.8}$$

Where:
Q_P = peak flow rate (e.g., maximum gal/day or maximum gal/h)
P_F = peaking factor (unitless)
Q_A = average flow rate (e.g., average daily flow (gal/day) or average hourly flow (gal/h))

  - Peaking factors depend on the type of building or development as illustrated in Table 3.25

3.68

Table 3.25 Example peaking factors for maximum daily flow rates for decentralized applications (typical values and range)

Peak flow rate of interest	Residential		Nonresidential		Mixed use[c]	Small community[c]
	Single or a few DUs[a]	Multiple DUs[b] (e.g., >15)	Commercial[c]	Institutional[c]		
Maximum day: recurring monthly	2.25 1.25–3.0	2.0 1.5–2.5	3.0 2–6	3.0 2–6	2.5 2–4	2.5 2–4
Maximum day: extreme value during a year[d]	4.5	4	6	6	5	5

[a]Values for maximum day/average day are from Lowe et al. (2009). Typical value represents 95 % of DUs monitored while range represents the ratios for the lowest and highest DU.
[b]Values for maximum day/average day are from Stephens (2007) and Mayer et al. (1999). Typical value represents 95 % of DU groups monitored.
[c]Values are from Crites and Tchobanoglous (1998) (Table 4-20). Peaking factors for Institutional are assumed equal to commercial and mixed use is assumed equal to small communities.
[d]Extreme value for maximum day/average day peaking factors were estimated as 2× the typical value.

3.69

- Predicting wastewater composition—Residential
  - Published data such as shown in Table 3.26 can be used to estimate the concentrations of different constituents in the waste-water from residential sources

Table 3.26 Representative characterization data for untreated wastewater and septic tank effluent generated from residential sources (after Lowe et al. 2007)

Source		Statistic	BOD_5 (mg/L)	TSS (mg/L)	Total N (mg-N/L)	Total P (mg-P/L)	Fecal coliforms (CFU per 100 mL)
Single domestic sources	Raw	Ave. (SD) Range	359 (220) 30–1147	405 (454) 18–2233	87 (45.2) 44.1–189	19.1 (4.15) 13.0–25.8	4.4×10^5 3.0×10^4–7.4×10^6
	STE	Ave. (SD) Range	180 (104) 38–861	79 (58.6) 22–276	57.7 (17.1) 26–124	12.2 (7.86) 3–39.5	2.2×10^5 1.9×10^3–1.3×10^8
Multiple domestic sources	Raw	Ave. (SD) Range	273 (104) 144–580	285 (91.7) 180–477	–	–	–
	STE	Ave. (SD) Range	169 (44) 63–229	66.4 (20.3) 27–99	49.3 (21.7) 29.8–75.3	7.03 (1.9) 5–10	7.0×10^5 1.4×10^5–2.7×10^6

3.70

- If septic tank effluent (STE) data are available but raw wastewater data are not, a rough estimate can be obtained using Eq. 3.9
  - Concentrations estimated this way can be insightful, but it is important to recognize they can be very imprecise

$$C_{RAW} = (R_A)(C_{STE}) \tag{3.9}$$

Where:
C_{RAW} = average concentration in raw wastewater (in mg/L or similar)
R_A = ratio of average concentration in raw wastewater to STE (–) (e.g., Table 3.27)
C_{STE} = average concentration in septic tank effluent (in mg/L or similar)

Table 3.27 Summary of common constituents of concern and the ratios of the average concentrations in raw wastewater versus that in STE

Source	Statistic	BOD_5 (mg/L)	TSS (mg/L)	Total N (mg-N/L)	Total P (mg-P/L)	Fecal Coliforms (CFU per 100 mL)
Single domestic sources	R_A (% removal)[a]	1.99 (50 %)	5.12 (80 %)	1.51 (33 %)	1.56 (36 %)	2 (50 %)
Multiple domestic sources	R_A (% removal)[a]	1.62 (38 %)	4.3 (77 %)	–	–	–

Based on the data shown in Table 3.26.
[a]The removal efficiency implied by the ratio is given in ( ).

3.71

- Predicting wastewater composition—Nonresidential
  - Published data can be used to obtain rough estimates of the composition of the wastewater from some nonresidential sources
  - However, compared to residential data, there is relatively less information available for nonresidential sources
    - Data that is available is often lumped into broad categories such as food service establishments, convenience stores, etc.
    - Data can include concentrations in untreated wastewater or septic tank effluents (e.g., Lowe et al. 2007, 2009)
    - There can be high variability among nonresidential sources, even those grouped into common categories, due to differences in business-specific operations
  - Depending on the intended application of the prediction data, appropriate monitoring is often recommended, if not essential, to enable reasonably accurate predictions

3.72

- For some nonresidential sources, rough predictions can be made for wastewater concentrations using simple mass balances (Eq. 3.10)

$$C = \left\{ \frac{[(Q_A)(C)]_{S1} + [(Q_A)(C)]_{S2} + \ldots}{(Q_A)_{S1} + (Q_A)_{S2} + \ldots} \right\} \tag{3.10}$$

Where:
C = concentration of a particular constituent in the average daily flow (in mg/L or similar)
Q_A = average daily flow from a contributing source (in gal/day or similar)
C = average concentration of a constituent in a source (mg/L or similar)
S1, S2, ... = source contributing to the development wastewater generation (e.g., motel, gas station, cafeteria,...)

3.73

3-6. Current and Emerging Issues

- Infiltration and inflow (I&I) contributions
  - Infiltration and inflow can contribute to flow volumes
    - Infiltration = Groundwater seepage into conveyance piping and tankage through holes, cracks, joint failures, and faulty connections
    - Inflow = Stormwater flow directly into conveyance piping or tankage via roof drain downspouts, foundation drains, storm drain cross-connections, and through holes in covers
  - Some system types and locations are susceptible to potential I&I flows, e.g.:
    - Systems that are old or poorly designed
    - Sites with high precipitation or high groundwater levels
  - I&I must be prevented to minimize extraneous clean water entering a decentralized treatment system
    - I&I can increase Q_A by 20 % or more and cause peak flow events

3.74

- Factors of safety
  - To account for uncertainty in estimates of water use or wastewater generation, factors of safety (FOS) can be used
    - FOS account for uncertain or unknown attributes, such as usage at a commercial establishment, while peaking factors account for known variability
    - FOS can be explicit (i.e., a value in a calculation) or implicit (i.e., buried within assumed parameter values)
      - Explicit FOS are preferable so it is clear how conservative an estimate or calculation might be
  - The magnitude of a FOS depends on the uncertainty in the estimate (e.g. of Q_A or Q_P) and the intended use of the data (e.g., sizing a robust treatment unit vs. sizing one that is very sensitive to flow or composition)
    - Reasonable values could be in the range of 1.0–2.0

3.75

- Occurrence of trace organic compounds
  - During the past decade, there has been growing interest in the occurrence and fate of micropollutants such as trace organics
  - Trace organic compounds originate from human activities and are categorized to include:
    - Biogenic compounds (e.g., 17-β-estradiol)
    - Consumer product chemicals (e.g., bisphenol-A)
    - Pharmaceuticals (e.g., ibuprofen)
    - Flame retardants (e.g., perflourooctane sulfonate)
  - These compounds can be present at trace levels (ng/L to μg/L) but still potential for serious harmful effects
  - A framework for predicting the occurrence and fate of trace organics in decentralized systems is shown in Fig. 3.16

3.76

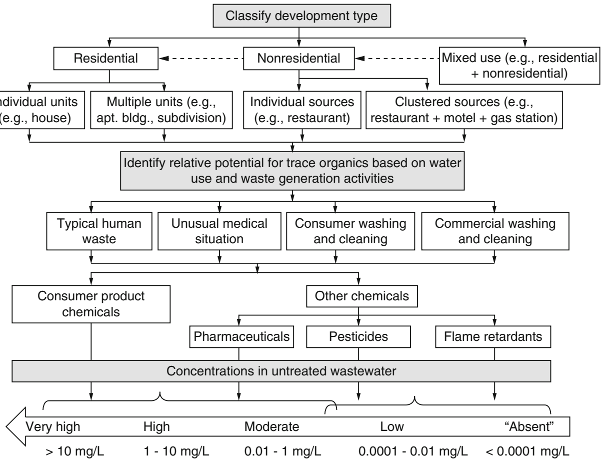


Fig. 3.16 Generalized decision diagram for predicting the occurrence of trace organic compounds in decentralized systems (Siegrist and Conn 2014)

3.77

3-7. Summary

- Contemporary water use and wastewater generation data are needed for decentralized system selection, design and implementation
  - Data are available for residential and nonresidential sources, including the flow and composition of individual water-using activities and events and for the combined flow out of a building
  - In a U.S. dwelling unit nearly 2/3 of the water used and waste generated is from toilet, shower, and clothes washer use
  - Water use and waste generation characteristics for nonresidential sources are typically very different than residential sources
- Predicting water use and wastewater generation
  - Predictions can be made by using available or obtainable monitoring data for a specific project or published literature data
  - Predictions for residential sources are more certain than those for commercial and other nonresidential sources

3.78

3-8. Example Problems

- 3EP-1. Predicting indoor water use at a single-family house
  - Given information
    - A single family house (three bedrooms) is located in a subdivision outside Denver, Colorado
  - Determine
    - Estimate the average daily indoor water use for this particular DU (gal/day)
    - Estimate the extreme daily indoor water use such that it will not likely be exceeded (gal/day)
      - This estimate can be considered the extreme average daily flow

3.79

- Solution
  - Using Eq. 3.2 estimate the average daily indoor water use

$$Q_A = 69.2 + 37.2N_P \quad (3.2)$$
$$Q_A = 69.2 + 37.2(4) = 218\,\text{gal/day}$$

  - Estimate the extreme Q_A by using conservative values for equation parameters
    - For example, $N_P = 4$ (represents 90 % of DUs in the U.S.) and add 2 SD for the average DU flow rates (1 SD = 94 gal/day per DU)

$$\text{Extreme}\,Q_A = 218 + 2(94) = 406\,\text{gal/day}$$

3.80

- 3EP-2. Predicting indoor water use at a condominium complex
  - Given information
    - Alpine Meadows is a condominium complex that was built in the foothills near Golden, Colorado in 1990
    - Alpine Meadows has a total of 32 dwelling units (DU) (4 DUs in each of 8 buildings)
    - Each DU has two bedrooms and there are conventional plumbing fixtures and appliances that were installed in 1990
  - Determine
    - Estimate the likely average daily indoor water use (gal/day)
    - Estimate the maximum recurring indoor water use (gal/day)
    - Estimate the daily indoor water use (gal/day) that will likely never be exceeded (e.g., <1 % chance)

3.81

- Solution
  - For the likely average daily indoor water use (Q_A) use Eqs. 3.5 and 3.2.
    - Due to the size of development (32 DUs), the parameter values used to calculate the development Q_A can approximate the average values for per capita flow rates and occupancy
    - Given the construction period was 1990, select indoor water use from relevant literature data (e.g., 69.3 gal/cap/day from Mayer et al. 1999) and select occupancy based on contemporary census data (e.g., 2.5 persons per DU)

$$Q_A = \left(\frac{Q_A}{DU}\right)(N_{DU}) \tag{3.5}$$

$$Q_A = \left(\frac{Q_A}{P}\right)(N_P) \tag{3.2}$$

$$Q_A = (69.3 \times 2.5)(8 \times 4) = 5544\,\text{gal/day}$$

3.82

- For the maximum recurring daily indoor water use Eq. 3.8 and assume a peaking factor of 2 (Table 3.25)

$$Q_P = (PF)(Q_A) \tag{3.8}$$
$$Q_P = (2)(5544 \text{ gal/day}) = 11,088 \text{ gal/day}$$

- Average daily indoor water use that will never be exceeded
  - For the average daily indoor water use that will likely never be exceeded, use $Q_A = 225$ gal/day for each of the 30 DUs per Dobbs et al. (2010)

$$Q_A = \left(\frac{Q_A}{DU}\right)(N_{DU}) \tag{3.5}$$
$$Q_A = (225)(8 \times 4) = 7200\,\text{gal/day}$$

3.83

- 3EP-3. Predicting indoor water use at a office building
  - Given information
    - A planned office building will be located in Colorado Springs
    - The four-story building will be occupied by an insurance company and will house about 100 workers
    - There will be restrooms and drinking water fountains on every floor, a sink fixture in a lunch room located on the second floor, and men's and women's locker room with showers on the ground floor
    - Water-efficient fixtures will be installed and have the following water use: toilets = 1.6 gal per flush, urinals = 1 gal per flush, showerheads = 2.5 gal/min, sink faucets = 2.5 gal/min
  - Determine
    - Provide an estimate of the total indoor water use that you would predict would occur during an average year (in gal per year)

3.84

- Solution
  - Using data given along with reasonable assumptions, you can estimate flows by two approaches and compare the results
  - Approach 1—Estimate flows using lumped values for contributors to flow using Eq. 3.6
    - * Assumptions made concerning usage:
      - – There are 100 employees and assume 20 gal/day per employee as typical for an office building (Table 3.23)
      - – Assume 5 workdays each week for a total of 240 days per year accounting for holidays

$$Q_A = \sum_{j=1}^{n}\left\{\sum_{i=1}^{n}[(N_S)(N_U)(Q_U)]_i\right\}_j \qquad (3.6)$$

$$Q_A = [(N_S)(N_U)(Q_U)]$$

$$Q_A = [(100)(1)(20\,\text{gal}/\text{day})] = 2000\,\text{gal}/\text{day}$$

$$\text{Annual } Q_A = (2000\,\text{gal}/\text{day})(240\text{ day}/\text{year}) = 480{,}000\,\text{gal}/\text{year}$$

3.85

- Approach 2—Estimate flows from water-using activities and events using Eq. 3.7
  - * Assumptions made concerning usage:
    - – Workers are 50 % men and 50 % women; men use toilet once and urinal twice each work day; women use the toilet three times each day
    - – Lunch room sink is used 20 min. each day; bathroom sinks are used 1 min. per visit
    - – Assume showers are used by 20 % of the workers each day and showers take 5 min. (assume toilet and urinal use is included above under 1, but add 2 min. of sink use)
    - – Assume water usage for routine office building cleaning amounts to 100 gal/day
    - – Assume five workdays each week for a total of 240 days per year accounting for holidays

3.86

$$Q_A = \sum_{j=1}^{n} \left\{ \sum_{i=1}^{n} [(N_U)(U_U)(V_U) + (N_U)(U_U)(Q_U)(T_U)] \right\}_{i\,j} \tag{3.7}$$

$$\text{Toilets\&Urinals } Q_A = \sum \left\{ \begin{aligned} & 50\,\text{males}\left[\left(\frac{1\,\text{use}}{\text{day}}\right)\left(\frac{1.6\,\text{gal}}{\text{toilet use}}\right) + \left(\frac{2\,\text{use}}{\text{day}}\right)\left(\frac{1.0\,\text{gal}}{\text{urinal use}}\right)\right] \\ & +50\,\text{females}\left[\left(\frac{3\,\text{use}}{\text{day}}\right)\left(\frac{1.6\,\text{gal}}{\text{toilet use}}\right)\right] \end{aligned} \right\} = 420\,\text{gal/day}$$

$$\text{Sink } Q_A = \sum \left[(100\text{ persons})\left(\frac{2.5\,\text{gal}}{\text{min}}\right)\left(\frac{1\,\text{min sin k}}{\text{visit}}\right)\left(\frac{3\,\text{visits}}{\text{day}}\right) + (1\text{ lunch sin k})\left(\frac{2.5\,\text{gal}}{\text{min}}\right)\left(\frac{20\,\text{min sin k}}{\text{day}}\right)\right]$$
$$= 800\,\text{gal/day}$$

$$\text{Shower } Q_A = \sum \left\{(0.2 \times 100\text{ persons})\left[\left(\frac{2.5\,\text{gal}}{\text{min}}\right)\left(\frac{5\,\text{min shower}}{\text{day}}\right) + \left(\frac{2.5\,\text{gal}}{\text{min}}\right)\left(\frac{2\,\text{min sin k}}{\text{day}}\right)\right]\right\}$$
$$= 350\,\text{gal/day}$$

$$\text{Cleaning } Q_A = 100\,\text{gal/day}$$

$$Q_A = 420 + 800 + 350 + 100 = 1670\,\text{gal/day}$$

$$\text{Annual } Q_A = (1670\,\text{gal/day})(240\,\text{day/year}) = 400,800\,\text{gal/year}$$

- Note the two approaches yield results that are 91 or 109 % of the average of 440,400 gal/year and this agreement is quite good

3.87

- 3EP-4. Predicting wastewater flow and composition from an apartment building
  - Given information
    - An apartment building is located in Golden, Colorado
    - The apartment building was constructed in 1999 and has 8 dwelling units, each of which has two bedrooms
    - Each dwelling unit has water using fixtures and appliances typical of the construction period except there are no automatic clothes washers
  - Determine
    - Estimate the maximum daily wastewater flow rate (gal/day)
    - Estimate the average concentrations of BOD_5 and TSS in the untreated wastewater (in mg/L)

3.88

- Solution
  - Maximum daily flow rate
    * Due to the small development size (8 DUs), the parameter values should be conservatively selected to account for the possibility there could be a higher than usual average daily flow
    * Compute wastewater flow based on Eq. 3.2
      – Use of Eq. 3.2 is conservative since it includes automatic clothes washers and there are none in this development
      – Select a value of 4 persons to be conservative (90 % of U.S. rental DUs have < 4 persons)
      – Use a PF of 2.0 for maximum daily flow

$$Q_A = 69.2 + 37.2N_P = 69.2 + 37.2(4) = 218\,\text{gal/day} \tag{3.2}$$

$$Q_A = \left(\frac{Q_A}{DU}\right)(N_{DU}) = (218\,\text{gal/day})(8\,\text{DU}) = 1744\,\text{gal/day} \tag{3.5}$$

$$Q_P = PF(Q_A) = 2(1744\,\text{gal/day}) = 3488\,\text{gal/day} \tag{3.8}$$

3.89

  - Average concentrations of BOD_5 and TSS in raw wastewater
    * Since the wastewater is from a residential source with typical fixtures and appliances (except there are no automatic clothes washers), one could assume the concentrations of BOD_5 and TSS should approximate literature data for residential sources
    * Representative data for multiple domestic sources as presented in Lowe et al. 2007 (see Table 3.26)
      – Average $BOD_5 = 273$ mg/L
      – Average TSS = 285 mg/L

3.90

- 3EP-5. Predicting wastewater flow rate and composition from a commercial development
  - Given information
    - A small commercial development is located along Highway 34 near Loveland, Colorado
    - It consists of a 30-unit motel and a gas station with a small convenience store and 50-seat cafeteria
  - Determine
    - Estimate the average daily wastewater flow rate from each of the three businesses in the development (gal/day)
    - Estimate the average daily wastewater flow rate from the development as a whole (gal/day)
    - Estimate the average concentrations of BOD_5 and TSS in the raw wastewater from the development as a whole (in mg/L)

3.91

- Solution
  - Assumptions made about usage:
    - Patrons and employees (potential input from the owner)
      - Motel—2 guests per day per room plus 2 employees
      - Gas station—100 customers per day for gas plus 1 employee
      - Cafeteria—2 customers per seat per day plus 4 employees
    - Water usage taken from literature tabulations (Table 3.24)
      - 50 gal/day per guest
      - 20 gal/day per employee
      - 7 gal/day per gas customer
      - 13 gal/day per café customer

3.92

- Average daily wastewater flow rates

$$Q_A = \sum_{j=1}^{n} \left\{ \sum_{i=1}^{n} [(N_S)(N_U)(Q_U)]_i \right\}_j \quad (3.6)$$

$$\text{Motel } Q_A = \left(\frac{50\,\text{rooms}}{\text{motel}}\right)\left(\frac{2\,\text{guests}}{\text{room}}\right)\left(\frac{50\,\text{gal/day}}{\text{guest}}\right) + \left(\frac{2\,\text{empl.}}{\text{motel}}\right)\left(\frac{20\,\text{gal/day}}{\text{empl.}}\right)$$
$$= 5040\,\text{gal/day}$$

$$\text{Gas station } Q_A = \left(\frac{0.5 \times 200\,\text{cust.}}{\text{day}}\right)\left(\frac{7\,\text{gal/day}}{\text{cust.}}\right) + \left(\frac{1\,\text{empl.}}{\text{gas station}}\right)\left(\frac{20\,\text{gal/day}}{\text{empl.}}\right)$$
$$= 720\,\text{gal/day}$$

$$\text{Cafeteria } Q_A = \left(\frac{50\text{ seats}}{\text{cafeteria}}\right)\left(\frac{2\,\text{cust.}}{\text{seat-day}}\right)\left(\frac{13\,\text{gal/day}}{\text{cust.}}\right) + \left(\frac{4\,\text{empl.}}{\text{cafeteria}}\right)\left(\frac{20\,\text{gal/day}}{\text{empl.}}\right)$$
$$= 1380\,\text{gal/day}$$

$$\text{Total } Q_A = 5040 + 720 + 1380 = 7140 \;\; \text{gal/day}$$

3.93

- Average concentrations of BOD_5 and TSS in the raw wastewater
  * Development sources include contributing activities and events similar to those occurring in a dwelling unit. But there could be elevated levels of some parameters like BOD_5 and TSS due to high toilet and urinal use in the gas station as well as high food preparation and cleaning uses in the cafeteria
  * The relative flow contributions from the different sources can be helpful to estimate the likely average concentrations of pollutants:
    – Motel Q_A = 5040 gal/day/7140 gal/day = 70.5 %
    – Gas station Q_A = 720 gal/day/7140 gal/day = 10.1 %
    – Cafeteria Q_A = 1380 gal/day/7140 gal/day = 19.3 %
  * We need to estimate the BOD_5 and TSS concentrations in each of these three contributing sources
    – Often, concentrations for raw wastewaters will not be available but septic tank effluent concentrations may be (Table 3EP.1)

3.94

Table 3EP.1 Septic tank effluent concentrations for relevant commercial sources (Lowe et al. 2007)

Source represented	Relevant available characterization data		BOD_5 (mg/L)	TSS (mg/L)
Motel	STE from multiple domestic sources	Median	184	62.4
		Average	169	66.4
		SD	44.0	20.3
		Range	63–229	27–99
Gas station	STE from non-medical institutions (i.e., nonresidential sources that do not have significant food service or medical services)	Median	561	41.8
		Average	620	50.9
		SD	443	28.5
		Range	74–2820	13.8–150
Cafeteria	STE from food services (i.e., nonresidential sources that have significant food service such as a restaurant)	Median	244	110.4
		Average	267	274
		SD	261	710
		Range	28–1537	12–4775

3.95

* Use of septic tank effluent data to estimate raw wastewater concentrations requires an adjustment factor for the removal efficiencies in typical septic tank units (Eq. 3.9)
  – For multiple domestic sources, the ratio of median concentrations in raw wastewater to septic tank effluent is given as (see Table 3.27):

    $BOD_5 = 1.62$ (equiv. to 38 % removal of BOD_5)

    $TSS = 4.3$ (equiv. to 77 % removal of TSS)

  – These ratios can be used, but it is important to recognize they are very imprecise
* Mass balance estimates of average BOD_5 and TSS in raw wastewater from the entire development can be made as shown on the following page (after Eqs. 3.10 and 3.9)

3.96

$$\mathrm{BOD_5} = 1.62 \times \left[\frac{(\mathrm{Q_A\,motel})(\mathrm{BOD_5\ motel}) + (\mathrm{Q_A\,gas\ station})(\mathrm{BOD_5\ gas\ station}) + (\mathrm{Q_A\,cafe})(\mathrm{BOD_5\ cafe})}{(\mathrm{Q_A\,motel}) + (\mathrm{Q_A\,gas\ station}) + (\mathrm{Q_A\,cafe})}\right]$$

$$= 1.62 \times \left[\frac{(5040)(169) + (720)(620) + (1380)(267)}{(5040) + (720) + (1380)}\right]$$

$$= 1.62 \times \left[\frac{(851,760) + (446,400) + (368,460)}{(7140)}\right] = 378\ \mathrm{mg/L}$$

$$\mathrm{TSS} = 4.3 \times \left[\frac{(\mathrm{Q_A\,motel})(\mathrm{BOD_5\ motel}) + (\mathrm{Q_A\,gas\ station})(\mathrm{BOD_5\ \ gas\ station}) + (\mathrm{Q_A\,cafe})(\mathrm{BOD_5\ cafe})}{(\mathrm{Q_A\,motel}) + (\mathrm{Q_A\,gas\ station}) + (\mathrm{Q_A\,cafe})}\right]$$

$$= 4.3 \times \left[\frac{(5040)(66.4) + (720)(50.9) + (1380)(274)}{(5040) + (720) + (1380)}\right]$$

$$= 4.3 \times \left[\frac{(334,656) + (36,648) + (378,120)}{(7140)}\right] = 451\ \mathrm{mg/L}$$

Note: These data for commercial source are higher than the results of Problem 3EP.4 for multiple domestic sources: $\mathrm{BOD_5} = 378$ vs. 273 mg/L and TSS = 451 vs. 285 mg/L, respectively.

3.97

Chapter 4

Water Use Efficiency and Waste Stream Source Separation

4-1. Scope

This chapter describes how the contemporary characteristics of indoor water use and wastewater generation that are often viewed as normal can be modified using water efficient fixtures and appliances and waste stream source separation approaches. These modifications can yield substantial direct and indirect environmental and economic benefits by reducing water use and wastewater generation and the associated energy use and greenhouse gas emissions and enabling recovery of water, organic matter, nutrients, and energy.

4-2. Key Concepts

- Indoor plumbing systems have substantially evolved in the United States and elsewhere over the past 100 years, with water supply and wastewater drainage systems becoming commonly available in urban and rural buildings by the middle of the 20th century.
- Modernization of indoor plumbing during the latter half of the 20th century led to increasing indoor water use and wastewater generation rates due to water-consuming fixtures and appliances and water use behaviors that were generally not constrained by water supply limitations, wastewater treatment and disposal impacts, high energy costs, or concerns over sustainability.
- Beginning in the 1970s, water shortages and droughts revealed there were in fact potentially serious water supply limitations. Subsequently there were growing concerns over energy use and sustainability that

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spurred interest in approaches that could help make water use and wastewater generation more sustainable.

- Two modern approaches to achieve more sustainable water use and wastewater generation include: (1) use of water efficient fixtures and appliances and (2) separation of the waste streams generated by water using activities and events based on their differing characteristics.
  - These approaches and the physical components can be implemented (1) as part of and in support of a decentralized system design or (2) as part of and in support of sustainability efforts in buildings in cities and urban areas that are served by centralized infrastructure.

- Basic water efficient fixtures and appliances began to appear in the 1970s due to droughts and a growing awareness of the benefits of water efficiency in residential and commercial settings.
  - Examples of basic water efficient fixtures and appliances include: water efficient toilets ($<$1.6 gal/use), clothes washers ($<$20 gal/use) and showerheads ($<$2.0 gal/min).
  - Use of basic water efficient fixtures and appliances in an individual dwelling unit, along with diligent attention to leak prevention, can reduce the daily indoor water use and wastewater flow to 100 gal/day/DU or less. Compared to traditional fixtures and appliances (e.g., typical of the 1980s and 1990s), efficiency improvements can yield a 60 % reduction in indoor water use and wastewater flow and a 45 % reduction in hot water use.
  - The reduced water use and lower wastewater flows can result in a concomitant reduction in energy use and greenhouse gas emissions associated with water treatment and distribution, water heating, and wastewater conveyance and treatment.

- Advanced minimum-flow fixtures and appliances began to emerge in the 1980s and developments have continued and commercial availability has grown.
  - Examples of advanced minimum-flow fixtures and appliances include: waterless toilets (0 gal/use), vacuum-flush toilets (0.25 gal/use), and air-assist showerheads (0.5 gal/min).
  - Use of minimum-flow fixtures and appliances in residential buildings and developments can reduce the indoor water use to as low as 50 gal/day/DU, about 50 % lower than that achieved with basic water efficient fixtures and appliances. Compared to traditional fixtures and appliances, advanced efficiency improvements can amount to nearly a 70 % reduction in indoor water use and wastewater flow and a 50 % reduction in hot water use.

- Use of minimum-flow fixtures and appliances in nonresidential buildings and developments can yield even greater reductions compared to residential applications.

- Waste stream source separation emerged as an approach to enhance decentralized treatment and enable resource recovery during the 1970s, but widespread use has not yet occurred. Examples of waste stream source separation approaches include:
  - Elimination of garbage disposers and handling garbage through solid waste composting for use as a soil amendment.
  - Separation and treatment of graywater for water reuse in toilet flushing and turf irrigation.
  - Diversion of urine with recovery and conditioning for use as a fertilizer rich in nutrients.

- Graywater consists of wastewaters from one or more basins, sinks, and appliances but always excludes toilet wastes. Graywater flow and composition varies depending on the sources included. In a residential building, mixed graywater includes wastewaters from all basins, sinks, and appliances excluding toilet wastes while light graywater excludes kitchen sink and dishwasher wastewaters. In a nonresidential building, graywater can vary from light to mixed. All graywaters contain pollutants and potentially pathogenic microorganisms and decentralized treatment systems and reuse plans have to account for this.

- Blackwater consists of excreta (urine and feces) plus dilution water and toilet tissue. Excreta are rich in nutrients (N, P, K) and contain high levels of pathogenic microorganisms. Decentralized use of excreta to recover nutrients can be safely accomplished if proper handling and management procedures are followed to mitigate infectious disease risks.

- The potential benefits from source separation include reduced wastewater pollutant loads, simplified and enhanced wastewater treatment options, enabled water reuse plans, and facilitated recovery of organic matter and nutrients. For example, recovery of $>$80 % of the N and $>$50 % of the P is possible through urine diversion out of residential wastewaters.

- For a particular application, the benefits actually realized through water efficient fixtures and appliances or waste stream source separation depends on the attributes of the building or development (e.g., residential dwellings vs. commercial businesses; retrofitting an existing building vs. installation in a new building), the attitudes and behaviors of the residents and users, and the nature of local utility services (e.g., availability and cost of water, wastewater, energy services).

- Successful long-term implementation of water efficient fixtures and appliances or source separation approaches requires that the involved and affected users be satisfied with the aesthetics, function, operational needs, and cost-benefit attributes of the approaches and systems deployed. The greater the departure from the attributes of traditional water using fixtures and appliances and wastewater management systems, the more challenging it can be for long-term sustainable use.
- Unintended consequences can occur if implementation is not well planned and executed. Example consequences include: (1) user dissatisfaction and reversion to traditional fixtures and appliances and wastewater management systems; (2) reduced water supply water quality in some settings due to low flow rates in distribution piping caused by water efficient fixtures and appliances; or (3) adverse health or environmental effects caused by improper reuse of graywater or nutrient recovery from excreta.

4-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 4 are presented in the Slides section.

4-4. Terminology

Terminology introduced and used in Chap. 4 is defined below.

Blackwater—Wastewaters from water-flush toilets and potentially including wastewaters from kitchen sink and dishwasher uses.

Excreta—In the context of human waste, excreta refers to human urine and feces.

Fecal sludge—For the purposes of this book, fecal sludge is defined as the mixture of human wastes combined with a small volume of water that accumulates in a vault, lined pit, or similar containment structure due to the use of ultra low-volume water-flush toilets. Other definitions of fecal sludge can be broader and encompass combinations of excreta and blackwater, with or without graywater (e.g., pit latrines, septic tanks, aqua privies, and dry toilets).

Fecal sludge management (FSM)—Fecal sludge management encompasses the removal of fecal sludge from a waterless toilet or ultra low-volume water-flush vault toilet (definition varies, see Fecal sludge) followed by the proper management for its treatment and disposal or beneficial recovery.

Graywater—Wastewaters produced by water use in basins, sinks and appliances in residential and nonresidential buildings. Mixed graywater includes food preparation related wastewaters (e.g., kitchen sink and dishwasher) while light graywater excludes food preparation wastewaters and possibly laundry wastewaters. All types of graywater exclude toilet wastewaters, which contain human excreta. Graywater can also be spelled as greywater.

Minimum flow fixtures and appliances—Fixtures and appliances that use little or no water but still function properly.

Source separation—In decentralized systems, refers to the separation and separate management of individual wastes and waste streams. For example using dual plumbing systems, blackwater comprised of toilet wastes and kitchen sink wastewaters can be separated from graywater produced by basins, other sinks, and appliances. Another example is the diversion of urine from fecal wastes using a urine-diverting toilet to enable urine processing and use as a fertilizer.

Toilet wastewater—Toilet wastewater consists of urine and feces plus toilet tissue.

Water recycling—The process of reusing reclaimed water for a function within the source responsible for the wastewater that was treated to produce the reclaimed water (e.g., graywater produced within an office building is treated and the reclaimed water is used for toilet flushing in that building).

Water use efficiency—Water use efficiency can encompass water use conservation measures with traditional fixtures and appliances (e.g., showering less frequently and for a shorter duration) or water efficient fixtures and appliances (e.g., a toilet with a lower flush volume per use).

Yellow water—Term that can be used to represent human urine.

4-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 4 are listed below.

BOD	Biochemical oxygen demand
BW	Blackwater
BWEFA	Basic water efficient fixtures and appliances
BWR	Basic water requirement
cap	Capita (or persons)
CFU	Colony forming units
CO_2	Carbon dioxide
COD	Chemical oxygen demand
CW	Clothes washer
d	Days

DU	Dwelling unit
DW	Dishwasher
EPAct	United States Energy Policy Act
F	Faucet
Fecal coli.	Fecal coliform bacteria
FOG	Fats, oils, and greases
g	Gram
gal	Gallon
GW	Graywater
K	Potassium
kg	Kilogram
kL	Kiloliter
kWh	Kilowatt-hour
L	Liter
MFFA	Minimum flow fixtures and appliances
min	Minute
N	Nitrogen
NH_4^+	Ammonium nitrogen
NO_3^-	Nitrate nitrogen
NO_x	Nitrous oxides (NO, NO_2)
P	Phosphorus
PO_4^{-3}	Phosphate
psi	Pounds per square inch
REUWS1	Residential end uses of water study 1
REUWS2	Residential end uses of water study 2
S	Shower
SO_2	Sulfur dioxide
TF	Toilet flush
TKN	Total Kjeldahl nitrogen
TSS	Total suspended solids
U.S.	United States of America
USEPA	U.S. Environmental Protection Agency
V	Volume of each activity or event (e.g., 5 gal per toilet flush)
WERF	Water Environment Research Foundation
WHO	World Health Organization
WRA	Water Reuse Association
WRF	Water Reuse Foundation

C_i	Concentration of a constituent in a particular waste stream
C_{T*}	Concentration of a particular constituent in the average daily flow after a source separated stream is removed
F_H	Fraction of indoor water use that is hot water use
F_R	Fractional reduction in use from water efficient fixtures and appliances

F_{R-H}	Fractional reduction in hot water use due to water efficient fixtures and appliances
F_U	Frequency of use (e.g., 2 urinal flushes per person per day)
i	Sources contributing to the flow being estimated in one building (e.g., restrooms, locker room, laundry services)
j	Different buildings that are present in a development (e.g., office building, restaurant)
M	Minutes of usage per day (e.g., 8 min of faucet use per person per day)
M_i	Mass in a particular stream
M_{SS}	Mass of a particular constituent in the average daily flow of the source separated waste stream
M_T	Mass of a particular constituent in the average daily flow of a combined wastewater stream
N	Number of activities or events (e.g., 4 toilet flushes per person per day)
N_U	Number of users (e.g., 6 males using a urinal)
Q	Flow rate by fixture or appliance use (e.g., 2.5 gal/min for a showerhead)
Q_A	Average daily indoor use in a DU with traditional plumbing
ΔQ_A	Savings due to water efficient fixtures and appliances
Q_{A-FA}	Average daily indoor use contribution of fixtures and appliances
Q_{A-Hot}	Average daily indoor hot water use in a DU with traditional fixtures and appliances
ΔQ_{A-Hot}	Hot water savings (i.e., avoided use) due to MFFA
Q_{A-R}	Average indoor use in a DU with efficient fixtures and appliances
Q_i	Flow rate of a particular waste stream
Q_L	Average daily water use contribution due to leakage
Q_O	Average daily water use contribution by other activities and events
Q_{SS}	Average daily flow of the source separated waste stream
Q_T	Average daily flow in a combined wastewater stream
Q_U	Flow rate during a water use (e.g., 2.5 gal/min during showering)
T_U	Time used during an activity (e.g., 8 min per shower)
V_U	Water volume used per water use (e.g., 1 gal per urinal flush)

4-6. Problems

4.1. What is a single person's basic water requirement (BWR) for drinking water and sanitation? How does this compare to the average daily indoor water use of a person in the United States?

4.2. In a typical city in the United States, approximately what fraction of the drinking water produced at the city's water treatment plant is wasted due to the combined effects of losses during water distribution, losses due to leaking plumbing fixtures, and drinking water use for flushing toilets: about 5 %, about 15 %, about 50 %, or over 75 %?

4.3. What are three potential benefits to utility services (e.g., water, wastewater, energy) from the use of modern water efficient fixtures and appliances?

4.4. For an apartment complex that was built in Denver in the 1970s, which of the following is the most likely maximum reduction in indoor water use that could be achieved if the apartments were retrofitted with high efficiency water-using fixtures and appliances: 10, 50, or 90 %?

4.5. A regression equation for indoor water use at a dwelling unit in the United States was developed through monitoring completed in the 1990s (Mayer et al. 1999). This equation (Eq. 3.2) was fit to water use data collected at households with traditional water-using fixtures and appliances typical of that period. How might Eq. 3.2 be modified to make it applicable to households with modern minimum flow fixtures and appliances?

4.6. Renovation is planned for an existing highway rest area that generates an average daily wastewater flow rate of 5000 gal/day (minimum day = 1000 gal/day and maximum recurring day = 15,000 gal/day). The rest area was built in 1978 and currently has the original water using fixtures. During the renovation project, minimum flow fixtures (faucets, urinals, toilets) will be installed. Estimate the average daily wastewater flow rate (Q_A) (in gal/day) that would be generated from the rest area after the planned renovation.

4.7. A subdivision of 300 homes located in Denver, Colorado was built during the 1970s. Assuming an average residency of 2.6 persons per home based on census data, if the pre-1980 toilets, showerheads and faucets were replaced by post-1994 water efficient fixtures and appliances (with maximum water usage as stipulated in the U.S. Energy Policy Act of 1992), what might the benefits be for the subdivision in terms of: (1) reduction in water use (in gal/year), (2) reduction in energy use for water and wastewater treatment (in kWh/year), and (3) reduction in greenhouse gases emitted by water and wastewater treatment (in lb/year of CO_2, NO_x plus SO_2)?

4.8. An indoor water use rate of 20 gal/day per person has been set as a target that is suggested as being achievable for dwelling units equipped with minimum flow fixtures and appliances. To achieve this target, what are the fixtures and appliances that could be used and at what utilization rates (i.e., events/day and volume per event or minutes per use and gal/min)? How realistic is it to sustainably achieve this target at a majority of dwelling units in a city where there are 50,000 houses?

4.9. Which of the following are examples of waste stream source separation (check all that apply): (1) elimination of a garbage grinder, (2) solids wasting from a bioreactor, (3) urine diversion and handling to recover N and P nutrients, (4) segregation and separate treatment of graywater?

4.10. Graywater can be comprised of different individual wastewater streams from a building. How might graywater comprised of restroom sink wastewaters in an office building compare to graywater comprised of wastewaters from sinks, basins and appliances (excluding toilet and kitchen sink waste) in an apartment building? Which graywater would likely present the greater risk related to the presence of pathogens?

4.11. Lookout Prairie is a new condominium complex being built near Eagle, Colorado that will have four buildings, each of which will have 6 condominium units. Each condominium will have two bedrooms and the developer is projecting on average, 22 of the condos will be occupied and each occupied condo will have an average of 2.5 residents in it. To help obtain a LEED rating, the building will be equipped with modern water-efficient fixtures and appliances, which are projected to bring the average water use down to 35 gal/day per person. A source separation strategy involving urine diversion is also being considered to recover nitrogen for use in fertilizing the property grounds (6-acre property and there will be 4 acres of Kentucky bluegrass turf). The anticipated nitrogen fertilizer requirements are listed below (in lb-N per 1000 ft^2). What is the likely maximum amount of nitrogen that could be recovered in the urine each year (in lb-N/year)? What percentage of the nitrogen requirements for turf fertilization could be satisfied by the nitrogen recovered from the urine diversion? If there will be too little or too much nitrogen available in the urine compared to what is required for fertilization (annually or seasonally), how would you handle this deficiency or excess?

Estimated fertilizer requirements in lb-N per 1000 ft^2 of landscape:					
Mid-November to Mid-March (18 weeks)	Mid-March to April (7 weeks)	May and June (8 weeks)	July to Mid-August (6 weeks)	Mid-August to September (7 weeks)	October to Mid-November (6 weeks)
0	0.5	1	0	1	1

4.12. Provide a list of five biogenic compounds (e.g., hormones and pharmaceutical residues) that can be found in urine that is diverted for recovery.

4.13. Long-term sustainable use of minimum flow fixtures and appliances or source separation schemes is critical to realizing the benefits projected and to avoid unintended consequences. Explain an untended consequence that could be of concern with respect to a decentralized system that is designed and implemented at a house where a composting toilet will be used and the graywater will be simply treated and reused for irrigation of a lawn and garden.

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Slides of Chapter 4

Decentralized Water Reclamation

Chapter 4: Water Use Efficiency and Waste Stream Source Separation

Contents

4.1

4-1. Introduction

- Evolution of indoor water use and wastewater generation
  - Indoor plumbing didn't become available in U.S. homes until after World War I and it took several decades for widespread availability
  - Economic development during the latter decades of the 20th century spurred modernization of indoor plumbing systems
    - Increasing numbers, types, usage of indoor fixtures and appliances
      - Higher flow rates and volumes per use
      - Increased frequency of use (e.g., bathing and washing)
      - Increased level of use of synthetic consumer products
    - This led to higher water use and wastewater generation—why?
      - Desire for user convenience and comfort, while achieving public health protection
      - Water supply was viewed as plentiful and there were limited concerns over wastewater treatment, energy use, and greenhouse gas emissions

4.2

- But how much water do humans really need?
  - True minimum need for daily living survival
    - 0.8–1.3 gal/day/cap of clean water
  - Basic needs for drinking water and sanitation
    - Recommendations of the U.S. Agency for International Development, World Bank, and World Health Organization
      - 5.3–10.6 gal/day/cap
    - Basic water requirement (BWR) proposed by Gleick (1996)
      - 13.2 gal/day/cap (50 L/day/cap) allocated as follows:
        - Drinking water = 1.3 gal/day/cap
        - Sanitation = 5.3 gal/day/cap
        - Bathing = 4.0 gal/day/cap
        - Cooking (excl. water to grow food) = 2.6 gal/day/cap

4.3

- Worldwide, billions of people do not have access to a BWR of 13.2 gal/day per person (50 L/day/cap)
  - For the year 2000, over two billion people in about 62 countries were estimated to have domestic water use that was less than the BWR of 13.2 gal/day/capita
  - Even large rapidly modernizing nations like India and China have average water use that is close to a BWR of 13.2 gal/day/capita
- In the United States and other developed countries
  - Indoor water use is much greater than a BWR of 13.2 gal/day/cap
  - For example, in the United States, the average daily indoor water use is generally greater than 50 gal/day/cap (refer to Chap. 3)

4.4

- Water use and wastewater generation in the 21st century
  - Sustainability concerns have grown in the United States and other developed countries spurring changes
  - Improved water use behaviors are becoming more prevalent with existing fixtures and appliances
    - Malfunctioning fixtures and appliances (e.g., leaks) are being fixed
    - Water efficient practices associated with water use and waste generation are being adopted
  - Advanced approaches and associated technologies are emerging and becoming more available
    - Indoor water use efficiency through low flow to minimum flow fixtures and appliances
    - Waste stream source separation through separation of fixture and appliance waste streams of differing characteristics

4.5

4-2. Water Use Efficiency

- During the 1970s, water shortages raised conservation awareness (Fig. 4.1)
  - Research and development occurred into:
    - Water use and conservation methods
    - Water efficient fixtures and appliances[a] (Fig. 4.2)
  - Nearly 30 years ago, the benefits of water efficient fixtures and appliances were recognized, including the potential for:
    - Lower water use
    - Reduced wastewater flow
    - Less energy use

[a]*Note*: Fixtures use water without requiring power; appliances use water but require power to function.

Fig. 4.1 Photograph of a dried up inland lake during a 1970s drought

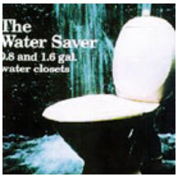


Fig. 4.2 Brochure for low flow toilet fixtures

4.6

- In 1983 the benefits of what were termed "minimum flow plumbing fixtures" were described (Table 4.1)

Table 4.1 Potential effects on indoor water use for houses equipped with plumbing systems employing minimum flow fixtures and appliances as proposed around 1980 (Siegrist 1983)

Fixture or appliance	Normal usage around 1980	Minimum flow plan A		Minimum flow plan B	
		Type	Usage	Type	Usage
Toilets	5 gal/use	Low flow flush toilet	1 gal/use	Air-assisted toilet	0.5 gal/use
Showers	5 gal/min	Low flow showerhead	1.8 gal/min	Air-assisted shower	0.5 gal/min
Clothes washers	37 gal/use	Front-loading washer	21 gal/use	Front-loading washer	21 gal/use
Faucets	3.2 gal/min	Aerators	1.6 gal/min	Aerators	1.6 gal/min
Total indoor water use incl. all uses	50 gal/day/cap	59 % lower indoor water use		67 % lower indoor water use	
Hot water use	15.9 gal/d/cap	45 % lower hot water use		52 % lower hot water use	

4.7

- Evolution of basic water efficient fixtures and appliances
  - Early requirements due to the U.S. Energy Policy Act of 1992
    - National water use efficiency requirements—three major provisions
      - * Maximum water use set for toilets, urinals, showerheads, and faucets manufactured after January 1994 (Table 4.2)
      - * Labeling required to clearly indicate water use volume or rate
      - * Encouragement of state and local incentive programs for replacement of old water consuming fixtures and appliances

Table 4.2 Water use efficiency requirements set for fixtures as a result of the U.S. Energy Policy Act of 1992 (Vickers 1993)

Fixture	EPAct 1992 maximum water use
Toilets	1.6 gal per flush[a]
Urinals	1.0 gal per flush
Showerheads	2.5 gal/min (at 80 psi)
Faucets and aerators	2.5 gal/min (at 80 psi)

[a]3.5 gal/flush allowed for some commercial units until 1 Jan 1997.

4.8

- Benefits of more water-efficient fixtures required by the U.S. Energy Policy Act of 1992
  - Estimates made around 1993 projected that there could be major reductions in indoor water use, energy use, and greenhouse gas emissions due to the increased water use efficiency of new toilets, showerheads and faucets (Table 4.3)
  - It was also projected that it would take over 30 years to achieve widespread use and fully realize the potential benefits
    - In 1993, it was estimated it would take until 2026 for the pre-1994 generation of fixtures to be replaced with post-1994 fixtures
    - After widespread use occurred dramatic benefits were projected as presented in Table 4.4

4.9

Table 4.3 Projected beneficial effects of increased water use efficiency in residential units (Vickers 1993)

Toilets, showerheads, faucets used in residential units	Water use (gal/day/cap)	Energy use (kWh/year per capita)	Greenhouse gases: CO_2, NOx, SO_2 (lb/year)
Before 1980	54.5	57	110.7
1980–1994	33.9	35	68.7
After 1994	21.4	22	43.4
Basis	Average daily usage per person: 4.0 toilet flushes, 4.8-min. shower, 4.0-min faucet use	Combined use for water treatment (1500 kWh per 10^6 gal/day) and wastewater treatment (1400 kWh per 10^6 gal/day)	Emissions per kWh: 1.89 lb CO_2, 0.00914 lb NOx, 0.0195 lb SO_2

Table 4.4 Projected per capita savings by a complete change to water-efficient fixtures in DUs as estimated in the early 1990s (Vickers 1993)

Savings from change in toilets, showerheads, and faucets used in residential units	Per capita savings (%)		
	Indoor water use	Energy use	Greenhouse gases: CO_2, NOx, SO_2
Post-1994 compared to 1980–1994	36.9 % ↓	37.1 % ↓	36.8 % ↓
Post-1994 compared to Pre-1980	60.7 % ↓	61.4 % ↓	60.8 % ↓

4.10

- Continuing and growing requirements for water-efficient fixtures and appliances
  - Potential for increased efficiency in water use was increasingly reflected in plumbing standards and specifications (Table 4.5)
    - U.S. Energy Policy Act of 2005 standards
    - American Society of Mechanical Engineers standards
    - USEPA Energy Star or WaterSense specifications

Table 4.5 Examples of efficiency requirements for residential applications

Residential fixture	Maximum water use volume or rate		
	Standard in the Energy Policy Act of 1992	Current standard or specification as of 2011	Proposed/future standard or specification
Toilets	1.6 gal per flush	1.6 gal per flush	1.28 gal per flush
Urinals	1.0 gal per flush	1.0 gal per flush	0.5 gal per flush
Showerheads	2.5 gal/min (at 80 psi)	2.5 gal/min (at 80 psi)	2.0 gal/min
Faucets/aerators	2.5 gal/min (at 80 psi)	2.2 gal/min (at 60 psi)	1.5 gal/min

Source: http://www.allianceforwaterefficiency.org/Codes_and_Standards_Home_Page.aspx.

4.11

- Water use efficiency has improved over time
  - Studies have revealed that indoor water use in U.S. urban areas has declined from about 177 gal/day/DU in the 1990s to about 138 gal/day/DU in the 2010s (Table 4.6)
    - Over about 20 years the decline in the average indoor water use among a service area of dwelling units was about 22.2 %
      - A majority of the decline was attributed to increasing use of more efficient toilets and clothes washers
    - A decline >22 % might have occurred but many DUs in the study areas were still using older less water efficient fixtures and appliances
  - Vickers (1993) projected a 36.9 % decline would occur by 2026 due to improvements in toilets, showers, and faucets (Table 4.4)
    - According to this projection and assuming a linear change with time, there would be about a 23.5 % decline by 2014, similar to the 22.2 % noted above and shown in Table 4.6

4.12

Table 4.6 Reductions in the indoor water use contributions of different activities and events due to increasing use of water efficient fixtures and appliances (DeOreo 2014)

Indoor water use by type	Contribution to indoor water use (gal/day per DU)			Potential overall change [Col. 3 versus Col. 1]
	Measured during REUWS1 (Mayer et al. 1999) [Col. 1]	Measured during REUWS2 (DeOreo 2014) [Col. 2]	Projected future water use (DeOreo 2014) [Col. 3]	
Toilet use	45.2	33.1	16.5	−63.5 %
Clothes washer use	39.3	22.7	11.7	−70.2 %
Shower use	30.8	28.1	23.3	−24.4 %
Faucet use	26.7	26.3	24.8	−7.1 %
Bathtub use	3.2	3.6	4.2	+31.2 %
Dishwasher use	2.4	1.6	2.0	−16.7 %
Other uses	7.4	5.3	1.5	−79.7 %
Leaks	21.9	17.0	4.3	−80.4 %
Total	176.9	137.7	88.3	−50.1 %

4.1

- Use of basic water efficient fixtures and appliances should continue to lower water use and wastewater generation over time
  - As projected by Vickers in 1993, widespread use of water efficient toilets, showerheads and faucets would require nearly 30 years to realize due to the time to retrofit existing fixtures
    - Widespread use could yield a 60.7 % or greater reduction in indoor water use compared to pre-1980 fixtures (Tables 4.3 and 4.4)
  - As projected by DeOreo in 2014, average indoor water use within residential service areas in the future could decline by about 50 % from that measured in the 1990s (Table 4.6)
    - Declines would be mostly due to increasing use of water efficient toilets and clothes washers combined with reducing other uses and minimizing water leakage at more and more dwelling units in the service area

4.14

- Evolution of minimum flow fixtures and appliances
  - Beyond the basic water efficient fixtures and appliances just discussed, there are minimum flow options (Fig. 4.3)
    - General characteristics include:
      * Ultra low to zero water use
      * Comparable user service level
      * Compatible with existing utilities
      * Need for little or no maintenance
    - Example minimum flow options include:
      * Waterless urinals (0 gal per use)
      * Composting dry toilets (0 gal per use)
      * Vacuum flush toilets (0.25 gal per use)
      * Air-assist showerheads (0.5 gal/min)

Fig. 4.3 Examples of advanced minimum flow fixtures: (**a**) vacuum flush toilet, (**b**) composting dry toilet, and (**c**) waterless urinal

4.15

- Application of minimum flow fixtures and appliances can yield extremely high degrees of water use efficiency (Table 4.7)

Table 4.7 Per capita indoor water use in a single DU through application of minimum flow fixtures and appliances compared with basic water efficient fixtures and appliances

Indoor water use by type	Assumed uses per day per person	Basic water efficient flow		Minimum water flow	
		Water use volume or flow rate	Indoor water use (gal/day/cap)	Water use volume or flow rate	Indoor water use (gal/day/cap)
Toilet	4/day	1.6 gal/use	6.4	0.5 gal/use	2.0
Clothes washer	0.36/day	20 gal/use	7.2	15 gal/use	5.4
Shower	0.6/day @8 min	2.0 gal/min	9.6	0.75 gal/min	3.6
Faucets	8 min/day	1 gal/min	8	0.5 gal/min	4.0
Bathtub	0.15/day	20 gal/use	3	20 gal/use	3.0
Dishwasher	0.10/day	10 gal/use	1	4 gal/use	0.4
Other uses	–	3 gal/day/DU	1.2[a]	1.5 gal/day/DU	0.6[a]
Leaks	–	10 gal/day/DU	4[a]	5 gal/day/DU	2[a]
Total gal/day/cap	–	–	40.4		21.0

The assumptions and projections made in this table are proposed by the author.
[a]Per capita flows due to other uses and leaks is based on 2.5 capita per DU.

4.16

- Water use efficiency potential under different conditions
  - Figure 4.4 illustrates measured and projected usage in a DU during different periods with increasing water use efficiency

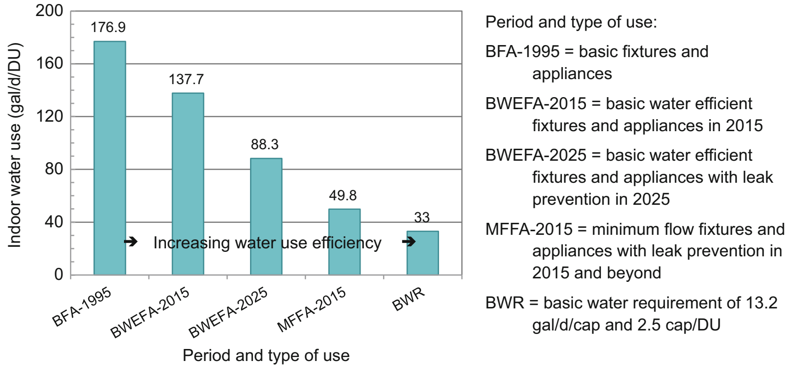


Fig. 4.4 Average indoor water use for an individual dwelling unit in the United States during different periods with different sets of fixtures and appliances

4.17

4-3. Source Separation

- Source separation can be achieved through two strategies
  - Eliminating materials and waste products that are often added during normal water using activities and events
    - This has also be referred to as pollutant load reduction
      - * The elimination of phosphate in laundry detergents during the 1990s is an example of pollutant load reduction with respect to phosphorus
    - Eliminating materials and waste products can also be used to achieve resource recovery (e.g., composting garbage rather than kitchen sink disposer use)
  - Separation of waste streams from different activities
    - Separation is based on contrasting flow and composition
    - Separation can enable treatment as well as resource recovery
  - While enabling treatment and resource recovery, source separation can also achieve water use efficiency

4.18

- Source separation by eliminating added materials
  - Examples of approaches are listed in Table 4.8

Table 4.8 Examples of waste material elimination approaches

Approach	Example	Enabling option	Effect on wastewater
Use pollutant lean products	Use of low-P biodegradable detergents	Not applicable	Reduce levels of P
Avoid adding waste materials	Avoid flushing chemicals and medicines down the toilet or sink (Fig. 4.5)	Dispose at a solid or hazardous waste facility; dispose at a pharmaceutical drop off	Reduce levels of toxic and hazardous substances
	Avoid use of kitchen sink garbage disposers	Compost food waste or use for biogas	Reduce levels of BOD, TSS, and FOG
Use of appliance treatment devices	Use of discharge filter bags on clothes washing machines (Fig. 4.6)	Not applicable	Reduce discharge of fabric fibers and TSS

Fig. 4.5 Examples of consumer products and medicines that can enter the wastewater stream

Fig. 4.6 Example of a bag filter for a clothes washer discharge

4.19

- Separating waste streams in residential dwellings
  - Nearly 40 years ago, it was recognized that there could be benefits from segregating waste streams generated in residential settings
    - Segregation and separate management in the United States was called out as illustrated in Fig. 4.7

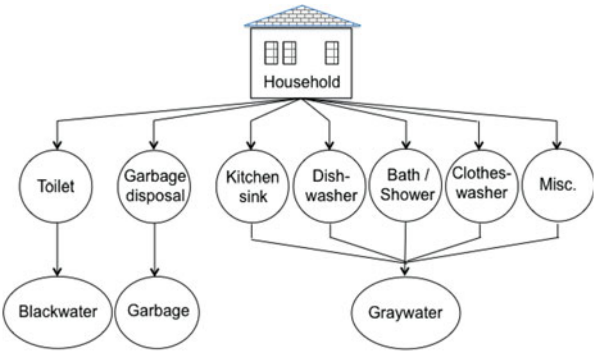


Fig. 4.7 Illustration of a waste segregation scheme proposed in 1978 (Siegrist 1978)

4.20

- Source separation to enable enhanced treatment
  - This was intended to exploit the contrasting characteristics of graywater and blackwater and enable enhanced treatment options
    - * Data tabulations for the concentrations and mass loadings in graywater and blackwater are presented in Tables 4.9, 4.10, 4.11, and 4.12

Table 4.9 Typical pollutant and pathogen loads contributed from the segregated graywater and blackwater streams (Siegrist 1978, USEPA 1980)

Parameter	Daily flow volume (e.g., gal/day/cap) or mass load (e.g., gal/day/cap)	
	Graywater stream	Blackwater stream
Flow volume	60–75 %	22–30 %
Organics (BOD_5) load	63 %	37 %
Suspended solids load	39 %	61 %
Nitrogen load	18 %	82 %
Phosphorus load	36 %	64 %
Pathogen load	Some . . .	Vast majority. . .

4.21

Table 4.10 Average concentrations of BOD, COD, N and P in graywater measured in Norway and reported elsewhere in Europe (Todt et al. 2015)

Location	BOD_5 (mg/L)	COD (mg/L)	Nitrogen (mg N/L)	Phosphorus (mg P/L)
Norway	140–160	250–300	16–19	1.3–1.6
Great Britain	146	451	8.7	1.4
Sweden	418	588	10	7.5
Germany	–	640	27.2	9.8
Netherlands	–	724	7.2	26
Average Europe	205–449	350–783	6.7–22	0.4–8.2
Average literature	100–400	200–700	8–30	2–7
NSF 2012	100–300	200–500	3–6	1–4

4.22

Table 4.11 Average concentrations of BOD, COD, N and P in blackwater measured in Norway and reported elsewhere in Europe with gravity flush (WC) and vacuum toilets (VC) (Todt et al. 2015)

Location	BOD_5 (g/L)	COD (g/L)	Nitrogen (g N/L)	Phosphorus (g P/L)
Norway	3.1–3.6	8.9–11.4	1.4–1.7	0.15–0.2
Sweden, low flush WC	–	3.0	0.16	0.028
Netherlands, VC	–	9.5–19	1.9	0.11–0.28
Turkey, WC	0.3	1.2	0.18	0.025
Germany, VC	–	8.0	1.5	0.175
Germany	0.3	0.7	0.28	0.029
Average WC	0.3–0.6	0.9–1.5	0.1–0.3	0.020–0.040

4.23

Table 4.12 Mass loadings per capita for nutrients in graywater (GW) and blackwater (BW) as measured in Norway and reported elsewhere in Europe (Todt et al. 2015)

Location	Organics (g COD/day/cap)		Nitrogen (g N/day/cap)		Phosphorus (g P/day/cap)	
	BW	GW	BW	GW	BW	GW
Norway	68–83	31–37	10–12	2–2.5	1.1–1.4	0.15–0.2
Sweden	85	39	4.6	0.6	1.5	0.5
Sweden	–	–	12	1.4	1.4	0.4
Germany	40	39	7.5	1.7	0.9	0.6
Netherlands	57–119	–	11.4	–	0.7–1.7	–
Typical Europe	75	46	11.9	1.5	1.5	0.5
Turkey	90	25	19.6	0.7	3.7	0.8

4.24

- Source separation to enhance resource recovery
  - Beyond benefits to treatment, source separation was also thought to help manage nutrient loadings and aid resource recovery of nutrients (Table 4.13)

Table 4.13 Water and nutrient loads contributed in separated sources with no dilution for urine and feces (Otterpohl et al. 2003)

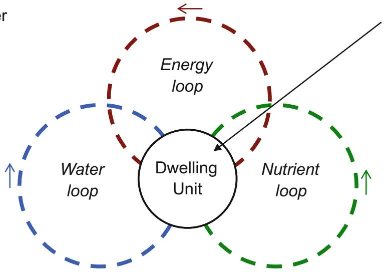
Parameter	Units	Approximate annual per capita contributions			
		Total	Graywater	Urine only	Feces only
Volume	gal/year/cap	6750–26,660	6600–26,420	132	13.2
	kL/year/cap	25–100	25–100	0.5	0.05
	% of total	100 %	99 %	<1 %	≪1 %
Nitrogen	kg N/year/cap	4–5	0.12–0.15	3.5–4.4	0.4–0.5
	% of total	100 %	3 %	87 %	10 %
Phosphorus	kg P/year/cap	0.75	0.08	0.38	0.30
	% of total	100 %	10 %	50 %	40 %
Potassium	kg K/year/cap	1.8	0.61	1.0	0.22
	% of total	100 %	34 %	54 %	12 %

4.25

- During the past 30 years, source separation has evolved
  - There have been developments and applications in Norway (Fig. 4.8), Sweden, Germany, Australia and other countries
  - In the United States growing interest and applications materialized during the past decade with a primary focus (so far) on separation and separate management of graywater
  - Approaches to source separation are also viewed as crucial to sustainable development worldwide
    - In 2006, the World Health Organization (WHO) published guidelines for the safe use of excreta and graywater (WHO 2006)
    - Several key conclusions and recommendations are summarized in Table 4.14
- Current approaches to source separation and separate management in the United States and similar industrialized nations are illustrated in Fig. 4.9

4.26

The goal is to: reduce water consumption in a home by 50%, facilitate near complete recycling of plant nutrients for agriculture, produce soil amendments, and produce energy from bio-resources



Dwelling unit has source separated waste streams. The energy loop involves heat, power and fuel. The water loop involves reclamation and recycling for irrigation. The nutrient loop involves nutrient recovery for plant production.

Fig. 4.8 Source separation to enable ecological sanitation as proposed in Norway (after Alsén and Jenssen 2005)

4.27

Table 4.14 Summarized findings concerning flow and composition of source separated wastes and management approaches (after WHO 2006)

Source and comments
Excreta (feces plus urine) ○ Estimated default values for excreted nutrients are 10.02 lb/year/cap of nitrogen and 1.21 lb/year/cap of phosphorus ○ The relative amounts of nutrients in urine and feces depend on diet. Digested nutrients are mainly in urine while undigested fractions are in feces. Approximately 88 % of the excreted nitrogen and 67 % of the excreted phosphorus are found in urine and the rest are in feces ○ Theoretically, 1/3 of the world's use of mineral nitrogen could be replaced by excreta. Similarly, 22 % of the world's use of mined phosphorus could be replaced by phosphorus from excreta ○ Urine is a quick-acting fertilizer that can be used for most vegetables ○ Feces may contain high concentrations of pathogens and treatment is critical to ensure safe use
Gray water ○ Gray water volumes produced per day: Poor areas with hand-carried water: 5.3–7.9 gal/day/cap; Developing countries: 7.9–26.4 gal/day/cap; Industrialized countries: 26.4–52.8 gal/day/cap ○ Concentrations of N, P, and K and pathogens of health concern are low in graywater and it will mainly provide a source of water for recycling ○ Bacterial indicators can grow in graywater and tend to overestimate the fecal load by 100–1000× ○ Microbial contamination of graywater is significant and must be accounted for in calculating risks and selecting treatment methods

4.28

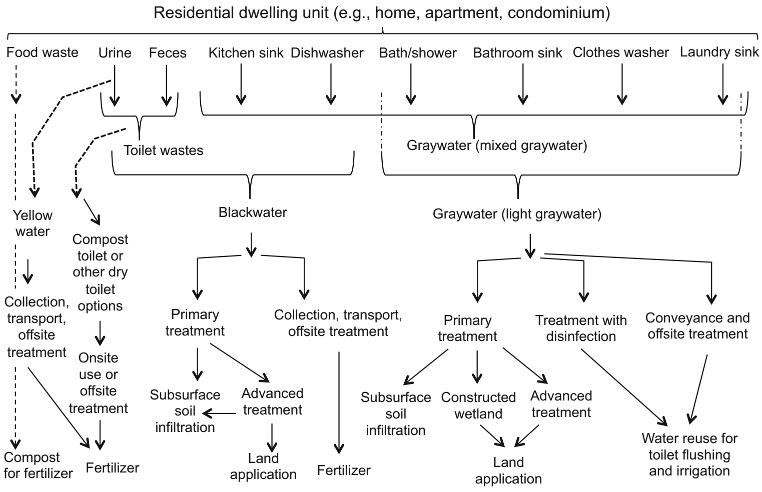


Fig. 4.9 Illustration of modern approaches to source separation and management

4.29

- Examples of source separation in residential applications
  - An example of source separation of blackwater and graywater in a building in an urban area is shown in Fig. 4.10

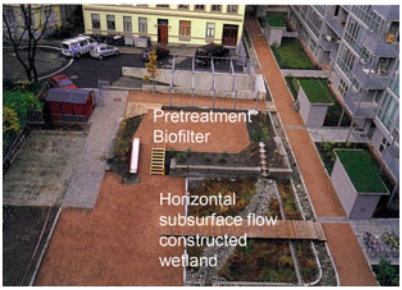


Fig. 4.10 Example of source separation for blackwater and graywater at a 33-unit apartment building in Oslo Norway (Jenssen 2015).
Blackwater—Feces and urine are transported via ultra low flush vacuum toilets connected to a composting bioreactor for generation of agricultural fertilizer.
Graywater—Collection and onsite treatment using a biofilter and subsurface flow wetland in the court yard area (area required = 10.8 ft^2 per person) with effluent use for irrigation

4.30

- An example of source separation involving urine diversion is shown in Fig. 4.11
  - ○ Urine is diverted and conditioned offsite for use as agricultural fertilizer
  - ○ Feces and graywater are separately collected for treatment and disposal/reuse

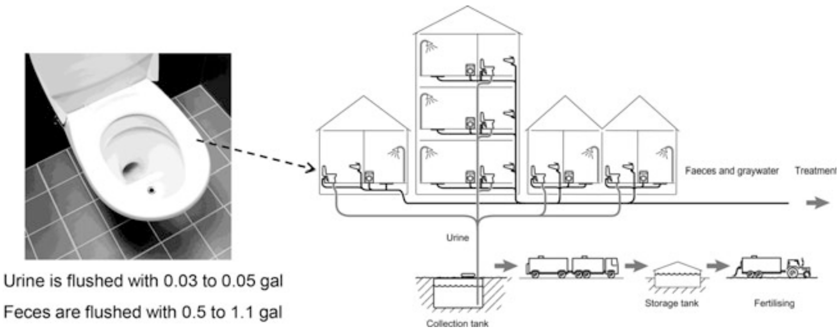


Fig. 4.11 Example of urine diversion in a residential development using urine diverting toilets with nutrient recovery via agricultural fertilizer (Jønsson et al. 2000)

4.31

■ Separating waste streams in nonresidential buildings

- While initially conceived for residential applications, application of source separation can also be valuable for nonresidential buildings and developments
- In nonresidential sources the proportion of different waste streams can be dramatically different from that of residential dwellings
  - ○ For example:
    - * A self-service laundry could be >90 % graywater
    - * A highway rest stop could be >90 % blackwater
- The extreme proportions of one or another waste stream in nonresidential sources can make source separation for enhanced treatment or resource recovery even more attractive than it can be in residential applications

4.32

4-4. Predicting Effects and Benefits

- Water use efficiency and source separation can offer major benefits
- Potential effects and benefits can include:
  - Reduced indoor water use and water supply demands
  - Reduced wastewater flows and pollutant loads
  - More cost-effective wastewater treatment
    - Smaller systems with more reserve capacity
    - Different types of systems with lower risks
  - Other sustainability benefits
    - Enabling resource recovery
    - Lowering energy and chemical use
    - Lowering greenhouse gas emissions

4.33

- Type and magnitude of effects and benefits depend on various factors and considerations
  - The type and nature of the building or development, for example:
    - Residential dwelling versus a nonresidential building
    - Individual source versus a cluster of sources or a city service area
    - New construction versus an existing older source
    - The attitudes and water use behaviors of residents and users
  - The features of the existing or planned utility services, for example:
    - Availability and costs of water supply and wastewater treatment
    - Availability and cost of energy
    - Options for, and value of, fertilizer use of N and P from excreta

4.34

- Predicting the effects of water efficient fixtures and appliances
  - Residential dwelling units and buildings
    - To estimate indoor water use efficiency in a single DU or residential building, a lumped approach can be used employing Eqs. 4.1 and 4.2

$$Q_{A-R} = (1 - F_R)(Q_A) \tag{4.1}$$

$$\Delta Q_A = Q_A - Q_{A-R} \tag{4.2}$$

Where:

Q_{A-R} = average indoor use in a DU with efficient fixtures and appliances (gal/day)

F_R = fractional reduction in use from water efficient fixtures and appliances (–)

For DUs, F_R is dependent on the fixtures and appliances used and the basis for comparison; F_R can vary from ~0.3 to 0.6 (30–60 % reduction)

Q_A = average daily indoor use in a DU with traditional plumbing (gal/day)

Note: Q_A can be estimated using Eqs. 3.2–3.5 and can be expressed per person or per dwelling unit or per cluster of dwelling units.

ΔQ_A = savings due to water efficient fixtures and appliances (gal/day)

4.35

- To estimate the savings in hot water use in a dwelling unit, Eqs. 4.3–4.5 can be used

$$Q_{A\text{-Hot}} = (F_H)(Q_A) \tag{4.3}$$

$$Q_{A\text{-RHot}} = (F_{R\text{-H}})(Q_{A\text{-Hot}}) \tag{4.4}$$

$$\Delta Q_{A\text{-Hot}} = Q_{A\text{-Hot}} - Q_{A\text{-RHot}} \tag{4.5}$$

Where:

Q_{A-Hot} = average daily indoor hot water use in a DU with traditional fixtures and appliances (gal/day)

F_H = fraction of indoor water use that is hot water use (–)

Q_A = average daily indoor use in a DU with traditional plumbing (gal/day)

Note: Q_A can be estimated using Eqs. 3.2–3.5

F_{R-H} = fractional reduction in hot water use due to water efficient fixtures and appliances (–)

For DUs, F_{R-H} varies from ~0.45 to 0.55 (45–55 % reduction)

ΔQ_{A-Hot} = hot water savings (i.e., avoided use) due to MFFA (gal/day)

4.36

- Estimating indoor water use efficiency in a multi-unit development requires consideration of the variability in traditional water use and wastewater generation among DUs as discussed in Chap. 3
  - For clusters of 10–20 or more DUs or a similar size multi-unit building, estimated flows tend to approach the average value
  - If the goal is to estimate the normalized efficiency per DU in the development, need to estimate the percentage of DUs using water efficient fixtures and appliances and the attributes of those being used
  - Where <100 % of DUs are using modern water efficient fixtures and appliances or some are older traditional versions
    - Need to scale the estimated indoor water use and waste generation to account for this <100 %

4.37

- Sources of information for Eqs. 4.1–4.5 are summarized in Table 4.15

Table 4.15 Typical sources of information for use in Eqs. 4.1–4.5

Parameter	Example sources of information	Example data
Q_A	• See Chap. 3	See Chap. 3
F_R	• Published data (e.g., Siegrist 1983, Vickers 1993) • Assumptions and calculations of fixture usage, rates and volumes	$F_R = 0.59$ for 1 gal/flush toilets, 1.8 gal/min showerheads, 21 gal/use clothes washers, and 1.6 gal/min faucets (Siegrist 1983) $F_R = 0.67$ for 0.5 gal/flush toilets, 0.5 gal/min showerheads, 21 gal/use clothes washers, and 1.6 gal/min faucets (Siegrist 1983) $F_R = 0.37$ for post-1994 to pre-1994 to $F_R = 0.61$ for post-1994 to pre-1980 usage (Vickers 1993)
F_{R-H}	• Published data (e.g., Siegrist 1983) • Assumptions and calculations of fixture usage, rates and volumes	$F_{R-H} = 0.45$ for 1 gal/flush toilets, 1.8 gal/min showerheads, 21 gal/use clothes washers, and 1.6 gal/min faucets (Siegrist 1983) $F_{R-H} = 0.52$ for 0.5 gal/flush toilets, 0.5 gal/min showerheads, 21 gal/use clothes washers, and 1.6 gal/min faucets (Siegrist 1983)

4.38

- Another approach accounts for the contribution of each fixture and appliance
  - Equation 4.6 captures the usage of common fixtures and appliances

$$Q_{\text{A-FA}} = \sum \begin{bmatrix} (N_{TF})(V_{TF}) + (N_{CW})(V_{CW}) + (N_S)(M_S)(Q_S) + \\ (M_F)(Q_F) + (N_{DW})(V_{DW}) \end{bmatrix} \tag{4.6}$$

Where:

Q_{A-FA} = average daily indoor use contribution of fixtures and appliances (gal/day)

Note: fixtures and appliances can all be traditional or all minimum flow or some combination of both; : average daily water use can be expressed per person or per dwelling unit or per cluster of dwelling units

N = number of activities or events (e.g., 4 toilet flushes per person per day)

V = volume of each activity or event (e.g., 5 gal per toilet flush)

M = minutes of usage per day (e.g., 8 min of faucet use per person per day)

Q = flow rate by fixture or appliance use (e.g., 2.5 gal/min for a showerhead)

Subscripts on N, V, M, Q:

TF = toilet flush, CW = clothes washer, S = shower, F = faucet, DW = dishwasher

4.39

- Equations 4.7 and 4.8 capture the fixture and appliance water use plus that associated with other events and leakage
- To estimate indoor hot water use, hot water use would replace indoor water use when employing Eqs. 4.6–4.8

$$Q_A = (Q_{A-FA}) + (Q_O + Q_L) \tag{4.7}$$

$$Q_{A-R} = (Q_{A-FA}) + (Q_O + Q_L) \tag{4.8}$$

Where:

Q_A = average daily indoor use in a DU, building or group of buildings with traditional plumbing including other and leakage use (gal/day)

Q_{A-R} = average daily indoor use in a DU, building or group of buildings with minimum flow fixtures and appliances including other and leakage use (gal/day)

Q_{A-FA} = average daily indoor use contributions of fixtures and appliances (gal/day)

Note: fixtures and appliances can all be traditional or all minimum flow or some combination of both

Q_O = average daily water use contribution by other activities and events (gal/day)

Q_L = average daily water use contribution due to leakage (gal/day)

4.40

- Sources of information for Eqs. 4.6–4.8 are given in Table 4.16

Table 4.16 Typical sources of information for use in Eqs. 4.6–4.8

Use	Uses/day per person at home	Water volume or flow rate during use: Unit	Example typical values[a]
Toilet	$N_{TF} = 4$	V_{TF} = gal/flush	Trad. old = 5, Trad. new = 1.6, Minimum flow = 0.5, Waterless = 0
Shower	$N_S = 0.6$	Q_S = gal/min	Trad. old = 5, Trad. new = 2.5, Minimum flow = 0.75
	M_S = 8 min		
Bath	$N_B = 0.15$	V_B = gal/bath	Variable, but can be around 20
Faucets	M_F = 8 min	Q_F = gal/min	Trad. old = 5, Trad. new = 2.5, Minimum flow = 0.75
Dishwasher	$N_{DW} = 0.10$	V_{DW} = gal/load	Trad. old = 12, Trad. new = 7, Minimum flow = 4
Clothes washer	$N_{CW} = 0.35$	V_{CW} = gal/load	Trad. old = 40, Trad. new = 27, Minimum flow = 21
Other	Q_O = gal/day per person		1.5–3[b]
Leakage	Q_L = gal/day per person		Trad. old = 7–12; achievable new = 0

[a]Trad. Old = Pre-1994, Trad. New = Post-1994, Minimum flow represents modern water-efficient fixture and appliances.
[b]Other includes water use for water conditioning equipment etc.

4.41

- Nonresidential buildings and sources
  - Equation 4.9 captures the water use contributions of traditional or minimum flow fixtures and appliances and Eqs. 4.7 and 4.8 can be used to capture other events and leakage

$$Q_{A-FA} = \sum_{j=1}^{n} \left\{ \sum_{i=1}^{n} \left[(N_U)(U_U)(V_U) + (N_U)(U_U)(Q_U)(T_U) \right]_i \right\}_j \tag{4.9}$$

Where:
Q_{A-FA} = average daily water use (e.g., gal/day or similar)
Note: fixtures and appliances can all be traditional or all minimum flow or some combination of both
N_U = number of users (e.g., 6 males using a urinal)
F_U = frequency of use (e.g., 2 urinal flushes per person per day)
V_U = water volume used per water use (e.g., 1 gal per urinal flush)
Q_U = flow rate during a water use (e.g., 2.5 gal/min during showering)
T_U = time used during an activity (e.g., 8 min. per shower)
i = sources contributing to the flow being estimated in one building (e.g., restrooms, locker room, laundry services)
j = different buildings that are present in a development (e.g., office building, restaurant)

4.42

- Predicting the effects of source separation
  - Estimating the effects of source separation depends on the separation scheme being considered and what data are of interest
    - For some applications, the flow and composition of a particular source separated waste stream are of interest, for example:
      * Composition of light graywater generated in an office building
      * Volume and nutrient composition in human urine
    - For some applications, the effects of removing a waste stream from the balance of the wastewater generated in a dwelling unit or nonresidential building may be of interest, for example:
      * Flow and composition of residential wastewater after light graywater is removed
      * Flow and composition of residential wastewater after urine is diverted

4.43

- For several common source separated streams, characterization data are available and may be relevant and appropriate for use
  - Data are given in Tables 4.10, 4.11, 4.12, and 4.13 for:
    * Graywater flow and composition
    * Blackwater or urine volume and composition
- For some applications, estimates of flow and composition can be made using mass balance equations as depicted in Fig. 4.12

4.44

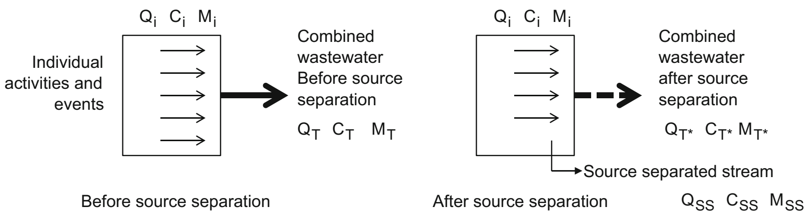


Fig. 4.12 Example mass balance schematics for application before (*left*) or after (*right*) source separation

- Concentrations in a wastewater after a source separated stream is removed can be estimated using Eqs. 4.10 and 4.11

$$C_{T^*} = \frac{(M_T - M_{SS})}{(Q_T - Q_{SS})} \qquad (4.10)$$

$$M_i = (Q_i)(C_i) \qquad (4.11)$$

Where:
C_{T^*} = concentration of a particular constituent in the average daily flow after a source separated stream is removed (mg/L or similar)
M_T = mass of a particular constituent in the average daily flow of a combined wastewater stream (mg/day or similar)
M_{SS} = mass of a particular constituent in the average daily flow of the source separated waste stream (mg/day or similar)
Q_T = average daily flow in a combined wastewater stream (L/day or similar)
Q_{SS} = average daily flow of the source separated waste stream (L/day or similar)
M_i = mass in a particular stream (mg/day or similar)
Q_i = flow rate of a particular waste stream (L/day or similar)
C_i = concentration of a constituent in a particular waste stream (mg/L or similar)
Note: Units can vary and conversion factors can be used in a given calculation

4.45

- Sustainably achieving predicted effects and benefits
  - Achieving long-term sustainable implementation of water efficient fixtures and appliances or approaches for source separation
    - This requires that the involved and affected users are satisfied with the aesthetics, function, operational needs and cost-benefit attributes of the approaches and systems deployed
  - The greater the departure from the attributes of traditional water using fixtures and appliances and conventional wastewater systems, the more challenging it can be for long-term use
    - An example of increasing departure from traditional might be:
      - Traditional water flush toilet (4–6 gal/use)
      - Water efficient flush toilet (1.6 gal/use)
      - Urine diverting water-flush toilet (0.05–1.0 gal/use)
      - Composting dry toilet (0 gal/use)

Increasing departure from a traditional fixture

4.46

- Unintended consequences and potential concerns
  - Application of water efficient fixtures and appliances or source separation in some situations can yield unintended consequences
    - A few potential consequences and concerns are discussed below
    - Applications done properly can help avoid these and other unintended consequences
  - Effects on waste conveyance in conventional sewerage
    - Changing the flow and composition of wastewater through water efficient fixtures and appliances or source separation could cause problems in a conventional gravity sewerage system
      - For example, if toilet wastes alone are discharged to a gravity sewer when graywater is treated and reused onsite, there could be solids accumulations and blockages

4.47

- Effects of highly reduced water use on tapwater supply quality
  - The quality of potable water may deteriorate during extended residence times in conventional water supply piping when minimum flow fixtures and appliances are used
    - Example effects include:
      - Loss of chlorine residual in the water
      - Increased levels of piping impurities leaching into water
  - Potential effects are most likely to occur in older plumbing systems and in buildings that are remote from the source of their water supply
  - Water supply piping systems and operational procedures should be able to minimize or eliminate these effects

4.48

- Source separation involving graywater
  - All graywater is an impaired water (i.e., a wastewater), but the degree of impairment depends on the activities and events contributing to it (see Fig. 4.9)
  - Depending on its composition, graywater will require some degree of treatment to ensure safe discharge or reuse (Table 4.14)
  - With graywater removed, blackwater management requires special attention due to its composition
- Source separation involving urine diversion
  - It can be difficult to install a separated drainline for the diverted urine within multi-story buildings and difficult to keep it functioning
  - With respect to many pollutants and pathogens, urine is normally relatively easily conditioned for nutrient recovery and use
  - But, concerns have emerged about the levels and fate of biogenic compounds in urine (e.g., hormones, pharmaceuticals) and how this might constrain recovery and beneficial use

4.49

4-5. Summary

- Indoor plumbing systems evolved during the past 100 years
  - Plumbing systems and behaviors do not yet fully account for sustainability concerns about water supply, waste management, energy use, and greenhouse gas emissions
- Modern approaches and technologies can help support more sustainable water use and wastewater generation
  - Water efficient fixtures and appliances (e.g., toilets, clothes washers, showerheads) can reduce water demand and wastewater flows by 50 % or more and reduce hot water use
  - Waste stream source separation (e.g., separate graywater from excreta; divert and recover urine) can enable wastewater treatment and water reuse and help achieve recovery of organic matter and nutrients
  - The effects and benefits realized depend on the application and ensuring sustainable long-term implementation

4.50

4-6. Example Problems

- 4EP-1. Estimating the water use effects of retrofitting minimum flow fixtures and appliances
  - Given information
    - Alpine Meadows is a condominium complex that was built in the foothills near Golden, Colorado in 1990
    - Alpine Meadows has a total of 32 dwelling units (DU) (4 DUs in each of 8 buildings) and each DU has two bedrooms and conventional plumbing fixtures and appliances from the 1980s
    - Alpine Meadows is considering retrofitting minimum flow fixtures including toilets, clothes washers, and showers
  - Determine
    - Estimate the average daily indoor water use and wastewater flow (Q_A in gal/day) if the dwelling units were retrofitted with new minimum flow plumbing fixtures and appliances (gal/day)
    - State whether the daily consumption of hot water would be reduced and if so, by how much (gal/day)

4.51

- Solution
  - Select the apartment occupancy based on census data for Denver, which shows an average of 2.2 persons per rental unit
  - Estimate the average daily water use under "normal" conditions with traditional fixtures and appliances using Eqs. 3.2 and 3.5

$$Q_A = 69.2 + 37.2N_P \tag{3.2}$$

$$Q_A = 69.2 + 37.2(2.2) = 151\,\text{gal/day}$$

$$Q_A = \left(\frac{Q_A}{DU}\right)(N_{DU}) \tag{3.5}$$

$$Q_A = (151\,\text{gal/day})(32\,\text{DU}) = 4832\,\text{gal/day}$$

4.52

- Estimate the reduction in average daily water use and hot water use under minimum flow conditions with water efficient fixtures and appliances
  * Different approaches can be taken for estimating the indoor water use and hot water use savings
  * Fixture and appliance utilization rates can be selected and used for water efficient conditions versus normal conditions
  * Alternatively, a rough estimate can be made using calculations presented in the literature
    – According to one minimum flow plan (Plan A in Siegrist 1983), on average, indoor water use is reduced by 59 % by installing low flush toilets (1 gal per use), showerheads (1.8 gal/min) and automatic clothes washers (21 gal per use)
    – This also yields a 45 % lower use of hot water based on hot water use being equal to 30 % of indoor water use

4.53

- Calculations made using the Plan A approach are shown below
  * Reduction in average daily indoor water use

$$Q_{\text{A-R}} = (1 - F_{\text{R}})(Q_{\text{A}}) \tag{4.1}$$

$$Q_{\text{A-R}} = (1 - 0.59)(4832\ \text{gal/day}) = 1981\ \text{gal/day}$$

$$\Delta Q_{\text{A}} = Q_{\text{A}} - Q_{\text{A-R}} \tag{4.2}$$

$$\Delta Q_{\text{A}} = Q_{\text{A}} - Q_{\text{A-R}} = 4832 - 1981 = 2851\ \text{gal/day}$$

  * Reduction in average daily hot water use

$$Q_{\text{A-Hot}} = (F_{\text{H}})(Q_{\text{A}}) = (0.30)(4832\ \text{gal/day}) = 1450\ \text{gal/day} \tag{4.3}$$

$$Q_{\text{A-RHot}} = (F_{\text{R-H}})(Q_{\text{A-Hot}}) = (0.45)(1450\ \text{gal/day}) = 652\ \text{gal/day} \tag{4.4}$$

$$\begin{aligned}\Delta Q_{\text{A-Hot}} &= Q_{\text{A-Hot}} - Q_{\text{A-RHot}} = (1450\ \text{gal/day} - 652\ \text{gal/day}) \\ &= 798\ \text{gal/day}\end{aligned} \tag{4.5}$$

4.54

- 4EP-2. Estimating the effects of urine diversion on wastewater concentrations and nutrient recovery
  - Given information
    - A source separation strategy involving urine diversion is being evaluated for Lookout Mesa, a new apartment building near Denver, Colorado. The apartment building has 8 dwelling units and each is expected to have an average occupancy of two persons
    - Each apartment will have water efficient fixtures and appliances and a urine diversion toilet setup which is expected to yield an average daily indoor water use of 40 gal/day per person
  - Determine
    - Estimate the average daily flow of wastewater (Q_A in L/year)
    - If urine diversion were implemented, what would be the total nitrogen (mg-N/L) and phosphorus concentration (mg-P/L) in the building wastewater (i.e., in Q_A)
    - Estimate how much nitrogen and phosphorus could be recovered (kg/year) from urine diversion at Lookout Mesa

4.55

- Solution
  - Based on literature values for wastewater generation data:
    - Total N = 4.5 kg/year per capita with 87 % from urine
    - Total P = 0.75 kg/year per capita with 50 % in urine
  - Wastewater flow rate with water efficient fixtures and appliances

$$Q_A = \left(\frac{Q_A}{P}\right)(N_P) \tag{3.3}$$

$$\frac{Q_A}{P} = (40\ \mathrm{gal/cap/day})\left(3.785\frac{\mathrm{L}}{\mathrm{gal}}\right) = 151.4\ \mathrm{L/cap/day}$$

$$Q_A = (151.4\ \mathrm{L/cap/day})(16\,\mathrm{cap})(365\ \mathrm{day/year}) = 884{,}176\ \mathrm{L/year}$$

4.56

- Total N and P concentrations and nutrient recovery
  - N concentration in the building wastewater (mg-N/L)

$$C_{T*} = \frac{(M_T - M_{SS})}{(Q_T - Q_{SS})} = \frac{(16\ \text{cap})[(4.5\,\text{kg/cap/year}) - (0.87)(4.5\text{kg/cap/year})]}{(884,176\ \ \text{L/year} - (16\ \ \text{cap})(500\ \ \text{L/year}))} \quad (4.12)$$

$$C_{T*} = 1.068 \times 10^{-5}\,\text{kg/L}$$

$$C_{T*} = \left(1.068 \times 10^{-5}\,\text{kg/L}\right)\left(10^{6}\,\text{mg/kg}\right) = 10.7\,\text{mg-N/L}$$

  - N recovered in diverted urine during the year (kg)

$$M_{SS} = (16\ \text{cap})(0.87)(4.5\ \ \text{kg/cap/year}) = 62.6\,\text{kg-N/year}$$

Note: Estimated wastewater concentrations of N and P and nutrient mass recovered assumes 100 % urine diversion is sustained year-round.

4.57

- P concentration in the building wastewater (mg-N/L)

$$C_{T*} = \frac{(M_T - M_{SS})}{(Q_T - Q_{SS})} = \frac{(16\ \text{cap})[(0.75\,\text{kg/cap/year}) - (0.50)(0.75\text{kg/cap/year})]}{(884,176\ \ \text{L/year} - (16\ \text{cap})(500\ \ \text{L/cap/year}))} \quad (4.12)$$

$$C_{T*} = 6.85 \times 10^{-6}\,\text{kg/L}$$

$$C_{T*} = \left(6.85 \times 10^{-6}\,\text{kg/L}\right)\left(10^{6}\ \text{mg/kg}\right) = 6.8\ \text{mg-P/L}$$

- P recovered in diverted urine during the year (kg)

$$M_{SS} = (16\ \text{cap})(0.50)(0.75\ \text{kg/cap/year}) = 6\ \text{kg-P/year}$$

4.58

Chapter 5

Alternative Wastewater Collection and Conveyance Systems for Decentralized Applications

5-1. Scope

Wastewaters generated in a building typically flow out of the building in drainage piping which is connected to a sewer system that leads to the site (s) where treatment and discharge or reuse can occur. For decentralized infrastructure situations where there are multiple buildings or sources in lower density developments, alternative sewer systems can be used. This chapter describes the features and principles and processes of four major alternatives: septic tank effluent gravity (STEG) and pressure sewers (STEP), grinder pump sewers, and vacuum sewers. This chapter then describes the design and implementation of STEG and STEP systems due to their widespread usage within decentralized infrastructure.

5-2. Key Concepts

- Decentralized systems serving a single house or business require building drainage piping that connects to a building sewer that conveys wastewater to a local site for treatment and discharge or reuse. For decentralized systems that serve multiple buildings (e.g., a subdivision of houses), wastewater often needs to be collected and conveyed some distance to the site of a treatment system for discharge or reuse.
- Conventional wastewater collection systems as typically used in urban areas with centralized wastewater treatment systems involve sewers that rely on gravity-based flow of untreated wastewaters. These sewers are characterized by larger diameter pipes that are installed with a desired slope to maintain scouring velocities, which commonly leads to requirements for periodic pumping stations and frequent cleanout

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manholes. Conventional sewer systems can be costly in decentralized applications and they are susceptible to infiltration and inflow.

- Alternative wastewater collection systems were developed and increasingly used in the 1970s. These systems were developed to collect and convey wastewaters using smaller diameter pipes that can be installed at variable grade lines with much shallower installations, fewer access points, and lower clean water infiltration and inflow. There are four major types of alternative sewers:
  - Septic tank effluent gravity sewers (STEG)—Convey septic tank effluent under gravity.
  - Septic tank effluent pressure sewers (STEP)—Convey septic tank effluent under pressure.
  - Grinder pump pressure sewers—Convey ground up wastewater under pressure.
  - Vacuum sewers—Convey untreated wastewater under vacuum.

- Design flow rates for sewer pipe sizing are based on equivalent dwelling units (EDUs).
  - One EDU is equal to an average daily flow from a building divided by a flow of typically 150 to 250 gal/day, the specific value selected by a designer or prescribed by a regulation. EDUs are calculated for segments within a sewer system and they accumulate as you go from the upstream segments of a sewer system toward the downstream outlet.
  - The design flow rate (Q_{DP}) is the peak flow rate at a specific location in the sewer system based on the number of EDUs contributing flow to it at that location. Based on experience, Q_{DP} is often set equal to a minimum flow rate from a single building plus a flow rate for each EDU contributing times the number of EDUs contributing (e.g., $Q_{DP} = 15 + 0.5N_{EDU}$).

- Alternative sewer systems are designed based on hydraulics of flow in plastic pipelines. For STEG and STEP systems, pipe diameters are initially chosen and then energy grade line slope and pipeline velocity calculations are made to assess suitability. Pipe sizes can be changed as necessary to keep the slope and velocity in line with good practice. For STEG and STEP systems there is no minimum velocity required since STE is being conveyed. Maximum velocities can be limited to 5 ft/s to control damage to piping and valve components. For STEG systems the energy grade line slope is determined by site topography. For STEP systems the energy grade line slope is determined by the attributes of the pumps used and pipeline sizing can help keep the slope in a low range (e.g., 0.5–1.5 %) to avoid wasting energy.

- STEG systems are often selected where topography enables gravity flow. STEG systems can have sections with negative slope as long as there is an overall positive energy grade line slope to the outlet from the sewer system. Effluent from a septic tank at each connection flows by gravity into the STEG system. Features of typical STEG systems include a service lateral size of 1.5–2 in., a main sewer pipeline size of 3–8 in., a depth of installation of 2–3 ft (with freezing protection as needed) and cleanouts at the end of lines and major changes in pipe diameters. During STEG design, the energy grade line available to drive flow is largely determined by site topography.
- STEP systems are often selected in hilly topography where pressure is required to lift effluent and maintain a positive energy grade line slope to the outlet of the sewer system. High-head submersible pumps are used in combination with a septic tank at each connection. Features of typical STEP systems include: service lateral size = 1–1.25 in., main sewer pipeline size = 2–8 in., depth of installation = 2–3 ft (with freezing protection as needed) and cleanouts at the end of lines and major changes in pipe diameters. During STEP design, the energy grade line available to drive flow is determined by the pump selection and not constrained by topography.
- STEP and STEG systems can be configured in hybrid systems that include combinations of system types such as:
    - A STEG system serving most of a subdivision with a STEP system serving a low-lying area that discharges into the STEG system. The combined STE flow could then be conveyed to a site for treatment and discharge or reuse.
    - A STEP system serving an area and conveys STE to the outlet where it is discharged into a nearby conventional gravity sewer that conveys the STE plus untreated wastewater to a site for treatment and discharge or reuse.
- Special considerations in design of STEG and STEP systems include: assurance that the septic tanks in the system are watertight, provision of air release and pressure sustaining valves at key locations, and provision of corrosion and odor control as needed.
- Routine operation and maintenance of the electrical and mechanical components of the STEG or STEP system is required (e.g., any pumps, valves, controls). In addition inspection of the individual septic tanks in the system is required with periodic removal and proper management of solids. If the septic tanks have effluent screens these require periodic cleaning. Experience has shown that cleaning of one or more segments of a STEG or STEP system is rarely needed, even in systems that have been operating for decades.

5-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 5 are presented in the Slides section.

5-4. Terminology

Terminology introduced and used in Chap. 5 is defined below.

Alternative sewers—Sewer systems that convey untreated or treated wastewaters utilizing gravity or pressure (positive pressure or vacuum) in smaller diameter pipelines with watertight joints that can be laid on variable grades with few access points and low infiltration and inflow. Alternative sewers include: septic tank effluent gravity (STEG), septic tank effluent pressure sewers (STEP), raw wastewater grinder pump sewers, and raw wastewater vacuum sewers.

Building drainage—Piping within a building that conveys wastewaters generated by usage of fixtures and appliances and typically connects to a building sewer that conveys the combined wastewater out of the building.

Building sewer—A sewer line that is connected to the building drainage piping and is used to convey wastewater to a treatment system located onsite or into a sewer system for collection and conveyance to the site of a nearby decentralize system or further away to a more remote centralized system.

Cluster—Term that refers to combining the wastewater flows from more than one building (e.g., multiple houses or several businesses) using a collection system so the combined flow can be treated for a chosen discharge or water reuse option.

Cluster system—A term used to describe decentralized infrastructure that is used to serve a group of buildings or other sources. A cluster system is often comprised of an alternative sewer system connected to a decentralized treatment system for effluent discharge or water reuse.

Conventional sewers—Sewers that include larger diameter pipes that are used to convey untreated wastewaters from multiple buildings under gravity flow (aided as needed by pumping stations if excavation depths get too great, to lift wastewater up for continued gravity flow) to a centralized facility for treatment and discharge or reuse.

Development—A term that typically refers to a group or cluster of buildings such as a subdivision of houses or a commercial center comprised of several businesses.

Drainage fixture unit (DFU)—A unit of measure used to size drainage piping in buildings. One drainage fixture unit is defined as equal to a discharge flow rate of 7.5 gal/min and various fixtures and appliances are allocated a certain number of DFUs based on their respective discharge flow rates.

Equivalent dwelling unit (EDU)—An equivalent dwelling unit is a construct used to normalize the discharges from different types of sources connected to a sewer. An EDU is based on a selected average daily flow rate. Designers or authorities can decide on the gal/d per EDU and values of 150–250 gal/day per EDU are typical.

Energy grade line (EGL)—The energy grade line represents the total head available to the liquid that is flowing in a pipe. For a liquid that is flowing without any energy losses due to friction (major losses) or components (minor losses), the energy grade line would be at a constant level. In practice, however, the energy grade line decreases along the pipeline due to friction and component losses.

Flow rate (Q)—(1) A measure of the volume of liquid that flows through a pipe during a certain period of time (e.g., gal/min, ft^3/s). (2) A measure of the volume of water used or wastewater generated during use of an appliance or fixture in a building (e.g., gal per laundry load, gal per toilet flush).

Hydraulic grade line (HGL)—The hydraulic grade line represents the total head available to a liquid flowing in a pipe minus the velocity head. In STEG and STEP systems, the velocity head is usually negligible so the HGL is approximately equal to the EGL.

Source—Source is defined as the origin of the wastewaters that are generated and will be treated for discharge or reuse. A source can include an individual dwelling unit, an apartment building, a cluster of dwelling units, a commercial or institutional building, a development of residential and/or commercial buildings, a portion of a city-wide service area, etc.

Scouring—A term that refers to the removal of solids that could accumulate in a sewer pipe during wastewater flow through it. Scouring velocity is that velocity which is sufficient to transport the solids and mitigate their deposition and accumulation. In a conventional gravity sewer for untreated wastewater a scouring velocity is typically 2 ft/s. In a STEG or STEP sewer system a scouring velocity can be near zero since these sewers convey septic tank effluents that have no gross solids or debris and only very low levels of suspended solids.

Sewer—A pipeline that is typically located below ground and used to convey wastewaters (untreated or treated) from one location to another.

Sewer system—A network of sewer lines that collect and convey wastewaters (untreated or treated) from one or more sources to the site(s) where treatment and discharge or reuse will occur. Depending on the type of sewer system, there can also be pumps, pump basins, controls, valves, cleanouts and other components that are part of the system and needed for the system to function properly.

Slope (S)—(1) A measure of the change in elevation along segments of pipe within a STEG system or the change in the elevation of the energy grade line along segments of pipe within a STEP system. (2) A measure of the change in elevation of a surface with distance (e.g., land surface where a soil treatment unit is installed, water surface in a constructed wetland).

Velocity (V)—A measure used to describe the speed of motion of a liquid in a pipe, channel or basin in units of length per time (e.g., ft/s).

Wastewater collection—Process and physical facilities involved in collecting wastewaters from individual sources using a sewer system.

Wastewater conveyance—Refers to the process of transporting wastewater (untreated or treated) under gravity or pressure forces from one location to another.

5-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 5 are listed below.

DFU	Drainage fixture unit
D&I	Design and implementation
EDU	Equivalent dwelling unit
EGL	Energy grade line
FOG	Fats, oils and greases
HGL	Hydraulic grade line
ID	Inside diameter
STE	Septic tank effluent
STEG	Septic tank effluent gravity sewer
STEP	Septic tank effluent pressure sewer
TDH	Total dynamic head
TSS	Total suspended solids

C	Hazen-Williams coefficient
D	Diameter
h_e	Change in elevation between the water level in the septic tank and the service lateral connection point
h_f	Head loss due to friction
h_{hv}	Friction losses in the discharge assembly of a STEP system pumping unit
h_l	Friction losses in the service lateral
h_p	Pressure head needed to transport Q_{DP} flow in the main sewer line of a STEP system
N_{EDU}	Number of EDUs
Q_{CAP}	Flow capacity of a segment of a sewer system
Q_{EDU}	Discharge rate for each EDU contributing to a sewer system
Q_{MIN}	Discharge rate from a single connection to a sewer system

Q_{DP}	Design flow rate for sizing a segment of a sewer system
R	Hydraulic radius
S	Slope of a sewer line
V	Velocity of flow in a sewer line

5-6. Problems

5.1. In the United States, alternative wastewater collection systems were developed beginning around the 1970s. What was the motivation for development and implementation of alternatives to the conventional gravity collection systems that had traditionally been used in urban areas?

5.2. What is the fundamental reason why alternative sewers can successfully be used to collect and convey wastewaters from buildings in relatively small diameter pipelines?

5.3. Is it reasonable to consider the use of small diameter pressure sewers for collection and conveyance of just the graywater (excluding kitchen wastes) that is produced in homes in a subdivision of 100 houses? If your answer is yes, would you suggest the graywater be treated before collection and conveyance and if so, by what method?

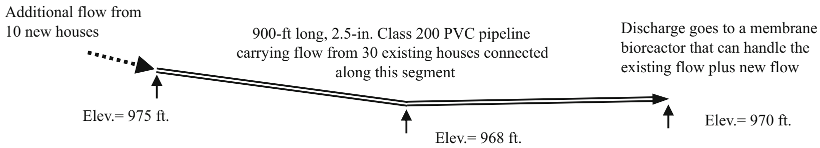
5.4. When estimating the peak flow rate for sizing a segment of a septic tank effluent pressure sewer the minimum flow rate used can be determined based on the type of pump(s) used. Explain the reason for this.

5.5. Installation of septic tank effluent gravity or pressure sewers can be installed using trenching equipment and placed at shallow depths below the ground surface. What concern(s) would you have about this type of installation in a cold climate location and what might you do to mitigate the concern(s)?

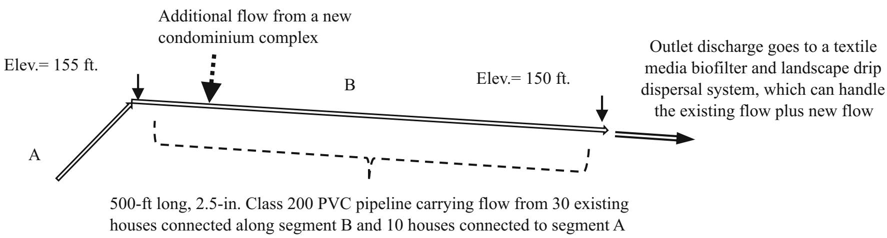
5.6. Select which type of alternative wastewater collection system, a STEG or STEP, best fits each of the following statements: (1) In which system will the pipeline capacity (Q_{CAP}) be determined by landscape topography and the available slope? (2) Which system would most likely be used to convey septic tank effluent from 100 homes located along a lake shore (elev. = 930 ft) for treatment in a recirculating sand filter system located about 3 miles away from the lake (elev. = 980 ft)? (3) Which system would most likely be used to convey STE from 20, 4-unit condominium buildings located at a resort (elev. = 1200 ft) to a nearby site (elev. = 1180 ft) where water reuse for landscape irrigation was planned following treatment in a membrane bioreactor?

5.7. Within a relatively new subdivision development there is a plan to complete the development by finishing 10 new houses and connecting them to an existing STEG collection system (see schematic below). Is

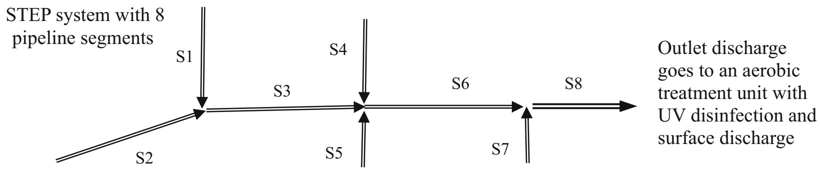
the existing pipeline shown below capable of handling the total flow, which includes that from the 10 new houses in addition to that from the existing 30 houses?
Given information and assumed values: 1 house = 1 EDU. Q_{DP} (in gal/min) = $15 + 0.5(N_{EDU})$.



5.8. Near an existing subdivision development there is a plan to expand the development by adding a condominium complex. The developer wants to connect the new complex to an existing STEG collection system serving the existing subdivision (shown below). How many condominiums can be added in the new complex without exceeding the capacity of the STEG pipeline segment B leading to the outlet?
Given information and assumed values: $Q_A = 225$ gal/d/house and 180 gal/d/condo. 1 EDU = 250 gal/d. $Q_{DP} = 15 + 0.5N_{EDU}$ for all STEG segments. C = 130.

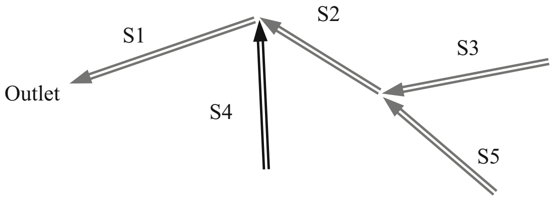


5.9. A planned subdivision development will have a STEP collection system that will deliver flow to a recirculating sand filter (see schematic below). Based on the information provided in the schematic below and given here, what is the design flow rate for sizing STEP segment S6?
Given information and assumed values: Houses connected to each pipeline segment: S1 = 8, S2 = 8, S3 = 6, S4 = 5, S5 = 5, S6 = 4, S7 = 8, S8 = 0. $Q_A = 225$ gal/d/house. 1 EDU = 150 gal/d. $Q_{DP} = 15 + 0.5N_{EDU}$ for all STEP segments.



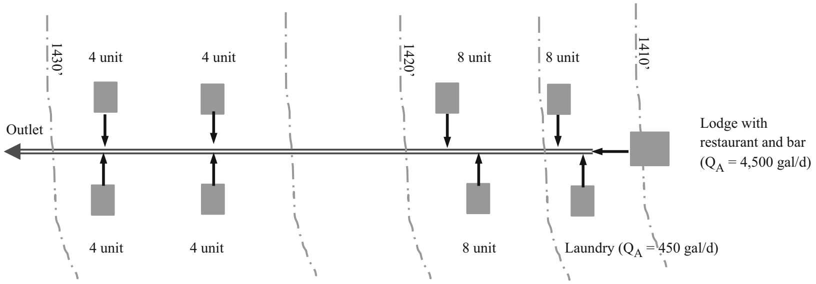
5.10. A STEP system is being designed to serve a development. One of the pipeline segments is being sized to carry the flow from a building with 12 apartments, a small restaurant with bar, and a 30-room motel. What is the design peak flow rate (Q_{DP}) that you would use to size the pipeline segment?
Given information and assumed values: $Q_A = 250$ gal/d for each apartment, 2500 gal/d for the restaurant, and 125 gal/d per room at the motel. Q_A of 250 gal/day = 1 EDU. $Q_{DP} = 15 + 0.5N_{EDU}$ for the STEP segment.

5.11. For the STEP system presented below, what is the total dynamic head (in ft) at each of the following three locations: (1) outlet from the system, (2) junction of pipeline segments S1, S2 and S4, (3) junction of pipeline segments S2, S3 and S5?
Given information and assumed values: Pipeline segment flow data are given in the table below. The elevations derived from a topographic map are: outlet elevation = 150 ft, junction of pipeline segments S1, S2, and S4 = 120 ft, junction of pipeline segments S3 and S5 = 120 ft.

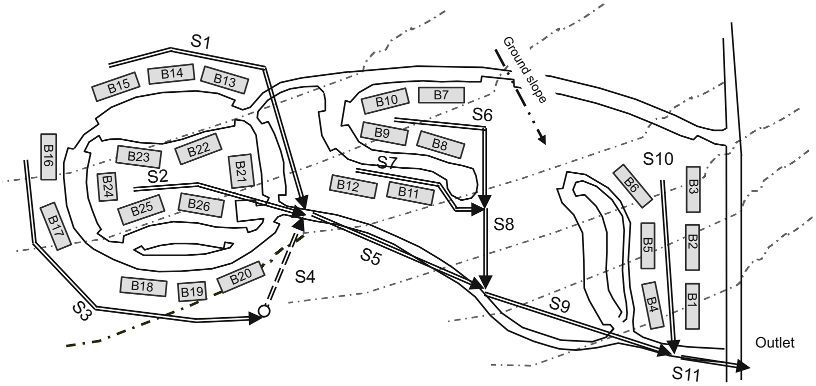


Pipeline no.	Cum. EDUs	Q_{DP}, (gal/min)	Length (ft)	Nom. diam. (in.)	True I.D. (in.)	Slope of EGL (%)	Cross-sectional area (in^2)	Pipeline velocity (ft/s)	Head loss due to flow (ft)
S1	111	55.5	1500	3.0	3.166	0.61	7.87	2.26	9.1
S2	67	33.5	1500	2.5	2.601	0.62	5.31	2.02	9.4
S3	21	10.5	1000	2.0	2.149	0.18	3.63	0.93	1.8
S4	20	10.0	1000	2.0	2.149	0.17	3.63	0.88	1.7
S5	19	9.5	1000	2.0	2.149	0.15	3.63	0.84	1.5

5.12. To serve a small resort development, A STEP collection system is being designed. The development has 1 lodge (with a restaurant/bar), 1 laundry, and 7 condominium buildings (4 or 8 units in each). Each building is served by a septic tank, which is connected to a 500-ft long pipeline that needs to convey the effluent to the outlet of the development. Determine the size of the pipeline so it is suitable based on EGL slope and flow velocity guidelines ($S = 0.5$–$1.5\,\%$, $V = <5$ ft/s).
Given information and assumed values: Each condo unit $Q_A = 300$ gal/day. 1 EDU = 150 gal/day. $Q_{DP} = 15 + 0.5N_{EDU}$. Schedule 40 PVC pipe is used with $C = 130$.

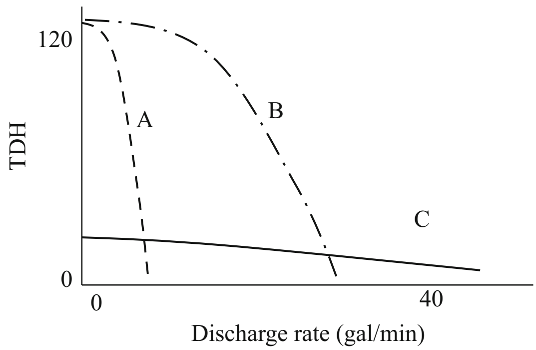


5.13. For the Mines Park housing development located on the Colorado School of Mines campus in Golden, Colorado you are tasked with completing hydraulic calculations as part of the design of a STEG collection system that will convey septic tank effluent from each building to the location of a decentralized membrane bioreactor system. Using the layout shown and the data provided in the tables below, for STEG segments S2, S5, S10, and S11, select a trial pipe diameter and determine the following: EGL slope (%), flow velocity (ft/s), and pipeline capacity (Q_{CAP} in gal/min) and then compare the ratio of Q_{DP} to Q_{CAP}. Comment on the suitability of the trial pipe size for each segment based on your calculations (but do not repeat any sizing calculations). Given information and assumed values: Q_A per DU = 0.85(69.2 + 37.2N_P/DU); 0.85 is used since there are no clothe washers in the DUs. 1 EDU = 150 gal/day. $Q_{DP} = 15 + 0.5N_{EDU}$ for all STEG segments. Schedule 40 PVC pipe is used with C = 130 and a minimum pipeline size = nominal 2-in. diameter. Elevation and length data were taken from a topographic base map and represent the beginning and end of each pipeline segment. Segment S4 requires a pump basin and pump to lift the effluent from segment S3 up and into segment S5.



Bldg. no.	DUs per bldg.	BRs per DU	N_P per DU	Seg. no.	Bldg. no.	Elev. (ft)	L (ft)	Seg. no.	Bldg. no.	Elev. (ft)	L (ft)
B1–B6	8	1	1	S1	B13–B15	5947–5910	810	S7	B11–B12	5915–5907	320
B7–B12	8	2	3	S2	B21–B26	5923–5910	490	S8	B7–B12	5910–5905	150
B13–B20	8	2	2	S3	B16–B20	5930–5900	940	S9	B7–B26	5905–5895	520
B21–B26	8	3	4	S4	B16–B20	5900–5910	270	S10	B1–B6	5910–5895	450
				S5	B13–B26	5910–5905	450	S11	B1–B26	5895–5892	100
				S6	B7–B10	5934–5907	490				

5.14. Which of the three pumps shown in the figure below (A, B or C) is most likely to be suitable for use at each house in a development with a STEP system if the TDH is about 90 ft?



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Slides of Chapter 5

Decentralized Water Reclamation

Chapter 5: Alternative Wastewater Collection and Conveyance Systems for Decentralized Applications

Contents

5.1

5-1. Introduction

- Wastewaters generated in buildings need to be collected and conveyed for treatment and discharge or reuse
- There are varied scenarios and collection and conveyance options
  - A few scenarios and options are highlighted in Fig. 5.1
  - Basic features of collection and conveyance options include:
    - Each building has internal drainage piping that collects wastewaters from fixtures and appliances
    - Wastewater conveyed out of a building can be handled alone or combined with the wastewaters collected from other buildings
    - Wastewater exits a building in a building sewer and can be fully treated and discharged or reused onsite, or be conveyed offsite to a nearby decentralized system, or be conveyed further away to a more centralized system
    - Wastewater flows in sewers under gravity or pressure forces

5.2

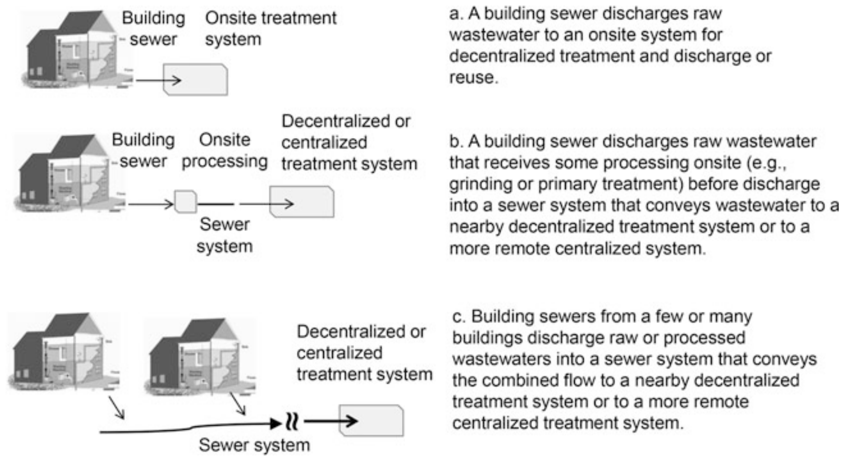


Fig. 5.1 Illustration of several scenarios with different wastewater conveyance options

5.3

- Building drainage piping
  - Fixtures and appliances are normally connected to drainage piping located within a building
  - Building drainage piping systems are sized to handle intermittent discharges of fixture and appliance wastewaters
  - Sizing is typically based on drainage fixture units (DFUs)
    - 1 DFU is defined as a 7.5 gal/min discharge flow rate
  - DFUs are assigned to each fixture or group of fixtures, e.g.:
    - Toilet—1.6 gal gravity tank
      - Private building = 3.0 DFUs
      - Public building = 4.0 DFUs
    - Clothes washer—automatic
      - Private or public = 3.0 DFUs
  - Pipe diameters and lengths depend on DFUs contributing

5.4

- Drainage piping for buildings with source separation
  - With source separation, modifications to conventional building drainage piping can be needed
    - Two drainage networks are needed for:
      - Urine diversion from a total wastewater stream
      - Separating graywater and blackwater
    - Three drainage networks are needed for:
      - Separating graywater and blackwater plus urine diversion
  - Installing modified drainage piping systems in buildings
    - Easiest in new development or during major renovations
    - In existing buildings, installation is possible, but more difficult to accomplish

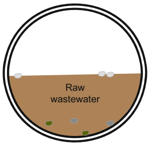
5.5

- Wastewater collection and conveyance in a sewer system
  - A sewer system is comprised of pipelines, basins, pumps, controls, and other components that are used to collect and convey untreated or treated wastewaters from individual buildings to the site of treatment and discharge or reuse
  - Conventional versus alternative sewer systems
    - Conventional sewers—traditionally involves the collection and conveyance of raw or untreated wastewaters under gravity forces to a remotely located centralized treatment system for discharge or reuse
    - Alternative sewers—typically involves collection and conveyance of processed or primary treated wastewaters under gravity or pressure forces to a decentralized treatment system for discharge or reuse or to a more remote centralized facility

5.6

- Features of conventional gravity sewer systems
  - Developed and used in the United States since the 1870s
  - General design features include:
    - Design capacity is based on the sewer flowing only half full (Fig. 5.2) for solids conveyance and cleaning equipment
    - Minimum diameter is usually >8-in. diameter
    - Installed to provide a consistent slope with velocities > 2 ft/s
      - Can lead to deep excavations and need for pumping stations to maintain slopes in areas with varied topography
    - Access ports (manholes) for inspection and cleaning are provided at each change in slope or alignment, but usually no further apart than 400 ft.

Fig. 5.2 Cross-section of a typical gravity sewer



5.7

- Installation of conventional gravity sewers can be very disruptive and costly as illustrated in Fig. 5.3

Source: www.nrsengineers.com.

Source: www.denvergov.org/WMDDesign/template22667.asp.

Fig. 5.3 Photographs of conventional sewer line installation

5.8

- Features of alternative sewer systems
  - Alternatives were developed and used in the United States starting in the 1970s to reduce the high cost of conventional gravity sewers in rural and peri-urban areas
    - The alternatives include two general features:
      - * Wastewater processing or treatment near buildings so wastewater solids are removed or reduced in size
      - * Use of small diameter pipes with watertight joints that are installed at shallow depths below the ground surface
    - There are four alternatives that include:
      - * Septic tank effluent gravity sewers
      - * Septic tank effluent pressure sewers
      - * Raw wastewater grinder pump pressure sewers
      - * Raw wastewater vacuum sewers
    - Features of these alternatives are highlighted in Table 5.1 and briefly described in the following pages

5.9

Table 5.1 Representative features of alternative sewer systems

Features	Septic tank effluent gravity	Septic tank effluent pressure	Grinder pump pressure	Vacuum
Source of flow to the sewer system and the connection discharge rate (gal/min)	Semi-continuous gravity flow of septic tank effluent	Intermittently pumped septic tank effluent	Intermittently pumped raw wastewater out of a grinder pump sump	Intermittently discharged raw wastewater out of a vacuum sump
	0.1–1[a]	0 to 5–20[b]	5–25[b]	> 100
Service lateral pipe diameter (in.)	1.5–2	1–2	1–2	2–4
Main sewer pipe diameter (in.)	3–8	2–8	2–12	4–10
Main sewer line slope and velocity desired	S > 0 % V < 5 ft/s	S = 1 to 1.5 % V < 5 ft/s	S = 1 to 1.5 % V = 2 to 5 ft/s	S > 0.2 % between lifts V > 3 ft/s
Maximum lift to the outlet of the system (ft)	0 ft	100 ft or more	100 ft or more	15 to 20 ft
Trench depth (ft)[c]	2–3	2–3	2–3	3–5
Cleanouts	Located at the end of pipelines or at major changes in pipeline size			

[a]In systems with pumps, the discharge rate is determined by the type and size of pump used [b]The discharge rate is for a septic tank serving a single house. Rates for septic tanks serving an apartment building or commercial establishment would be higher [c]Minimum trench depth is based on climate and frost depth unless piping is insulated or heat-traced. Maximum trench depth is based on pipe type and strength.

5.10

- Septic tank effluent gravity sewers
  - Wastewater from a source is treated in a septic tank near the source and then septic tank effluent (STE) in collected and conveyed in a small diameter gravity sewer system
    - These effluent sewers have several names, including:
      - Septic tank effluent gravity sewers (STEG), small-bore sewers, effluent drains
      - In the United States they are called STEG systems
  - General design features include:
    - Sewers can be designed to flow full or nearly full (Fig. 5.4) and small diameter pipes can be used (e.g., 2 in.)
    - Installation can be shallower and at variable slopes and
    - $\geq$2 ft/s scouring velocities are not needed
    - Watertight joints and fewer access points can yield low I&I

5.11

- Septic tank effluent pressure sewers
  - Wastewater from a source is treated in a septic tank near the source and then STE is pumped into a small diameter pressurized sewer system for collection and conveyance
    - These effluent sewers are referred to as STEP systems
  - General design features include:
    - Sewers can be designed to flow full (Fig. 5.4) and small diameter pipes can be used (e.g., 1.5 in.) (Fig. 5.5)
    - Installation can be shallower and at variable slopes and $\geq$ 2 ft/s scouring velocities are not needed
    - Watertight joints and fewer access points can yield low I&I
    - STEP systems can be favorable over STEG systems in hilly terrain where gravity is not capable of moving STE to a location targeted for treatment operations

5.12

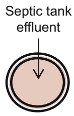


Fig. 5.4 Cross-section of a STEG sewer

Fig. 5.5 Photographs illustrating the installation of a septic tank effluent pressure sewer (STEP) in a rural development area. Installation is made using a continuous trencher (*left*) and insulation can be added for shallow burial in cold climates (*top*). (Photographs courtesy of R.J. Otis)

5.13

- Grinder pump pressure sewer systems
  - Wastewater flows by gravity from a building into a nearby sump containing a grinder pump (Fig. 5.6)
    - The grinder pump is turned on periodically by float controls and it chops up the raw wastewater and reduces the heterogeneous mix of solids and objects into a slurry
    - The pump discharges the wastewater slurry into a small diameter pressure sewer system
  - General design features include:
    - Sewers can be designed to flow full and small diameter pipes can be used (e.g., 2 in.)
    - Installation can be shallower and at variable slopes but the velocity achieved in the pressure sewer has to achieve scouring and solids conveyance (e.g., $\geq$2 ft/s)
    - Watertight joints and fewer access points can yield low I&I

5.14

- Vacuum sewer systems
  - A vacuum pump in a remote collection station maintains a vacuum of 15–20 in. Hg on the main sewer lines (Fig. 5.6)
    - Wastewater from a building flows to a sump separated from a vacuum sewer main by a vacuum valve
    - When a volume of wastewater accumulates, a vacuum valve is opened and air and wastewater is sucked into the main sewer
    - The wastewater flow is two phase (air and water), which can break down larger solids and aid conveyance
  - General design features include:
    - Sewers are designed to flow partially full at velocities that achieve scouring and solids conveyance (e.g., $\geq$ 2 ft/s)
    - Small diameter pipes (e.g., 4 in.) are installed at shallow depths
    - Watertight joints and fewer access points can limit I&I

5.15

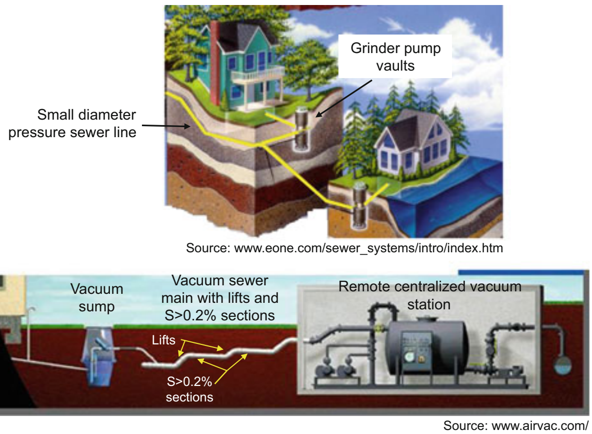


Fig. 5.6 Illustration of the key components of a grinder pump pressure sewer system (*top*) and a vacuum sewer system (*bottom*)

5.16

- Conditions that are well-suited for alternative sewers
  - The existing or new development is located in an area with level or varied topography, shallow bedrock, or high ground water
  - Wastewater needs to be collected from a cluster of buildings, a housing development, or town that has:
    - Larger lot sizes (e.g., >0.5 acres) or limited connections per mile of sewer line (e.g., <100) (Fig. 5.7)
    - Existing or potential for onsite treatment or processing

Fig. 5.7 Examples of a small town (*left*) and lower density rural housing development (*right*) where there are larger lot sizes and would be limited connections per mile of sewer line

5.17

5-2. Principles and Processes

- Conveyance using small-bore sewers
  - Small-bore sewer systems do not handle gross solids and debris or high concentrations of total suspended solids (TSS) and fats, oils and greases (FOG)
  - In STEG and STEP systems, a septic tank (Chap. 6) is used for primary treatment at each building in a development to produce an effluent that is suitable for small-bore conveyance
  - Small-bore sewers can also be used for other wastewaters
    - Wastewater effluents with low solids contents that are produced by unit operations other than septic tanks (e.g., aerobic unit, porous media biofilter, etc.)
    - Graywater (e.g., untreated or after a settling basin)
  - Chapter 5 is focused on small-bore sewers for conveyance of septic tank effluent (STEG and STEP systems)

5.18

- Alternative sewer system types and layouts
  - The outlet is the terminus of the sewer system that collects wastewater effluent from the buildings within a development
  - The outlet elevation relative to the elevation within a development guides suitability for each system (Fig. 5.8)

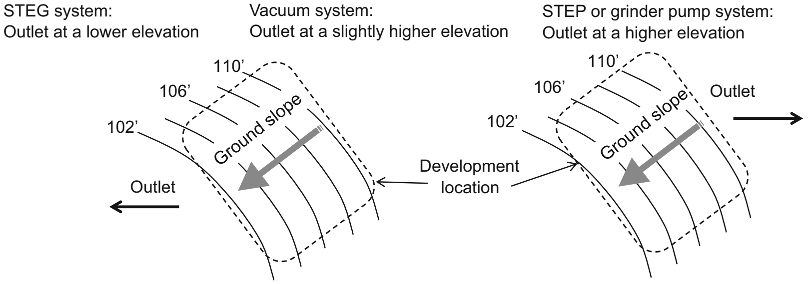


Fig. 5.8 Illustration of topographic conditions and the collection network outlet location as it relates to the suitability of using a particular alternative sewer system

5.19

- Development of effluent sewer system layouts
  - Topographic base maps enable system layout and design
  - Based on the outlet location, a system type is chosen
  - Sewer line layouts can then be selected to suit the development
    - Consider landscape features and try to minimize disruption to structures, roads, and other features
    - Consider options for dividing the collection network into segments
      - Segments are used to enable rational upsizing of sewer pipe diameters as more flow is accumulated toward the outlet
      - A new segment is also typically used downstream of the junction of multiple segments
  - Figures 5.9 and 5.10 illustrate a STEG and STEP system as laid out for the Mines Park development of 26 apartment buildings located on the Colorado School of Mines campus

5.20

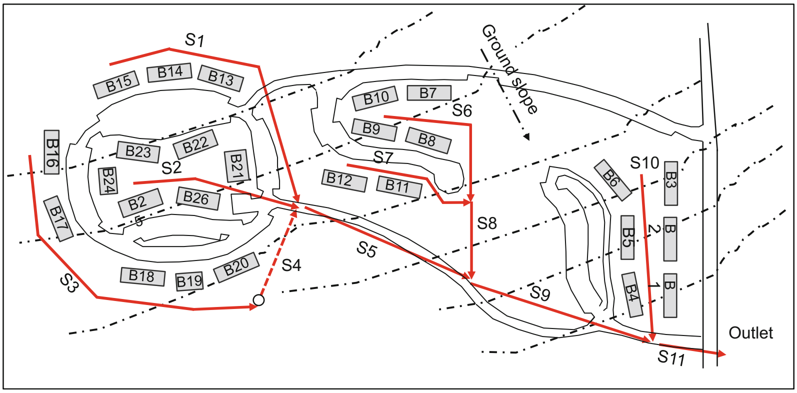


Fig. 5.9 Illustration of a STEG system serving a development of 26 apartment buildings (denoted by B1, B2, etc.). *Note*: The STEG system has 11 segments denoted as S1, S2, etc.

5.21

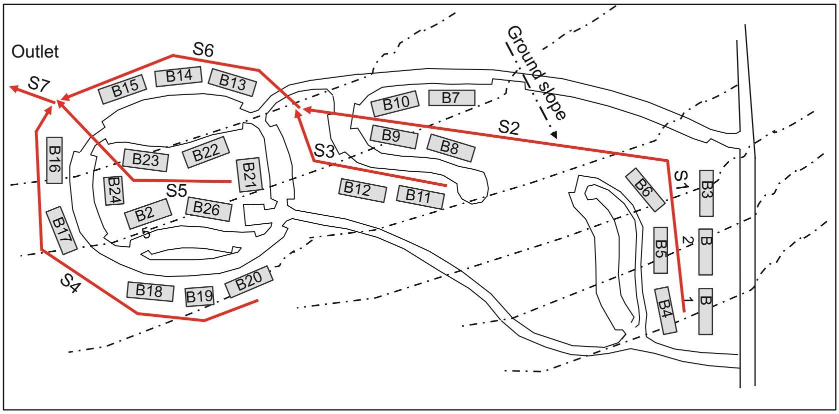


Fig. 5.10 Illustration of a STEP system serving a development of 26 apartment buildings (denoted by B1, B2, etc.). *Note*: The STEP system has 7 segments denoted as S1, S2, etc. Segment S1 and S2 could be combined into a single segment if the majority of the flow originates from the upstream end

5.22

- Cross sections are used to show elevation and length as illustrated in Fig. 5.11.

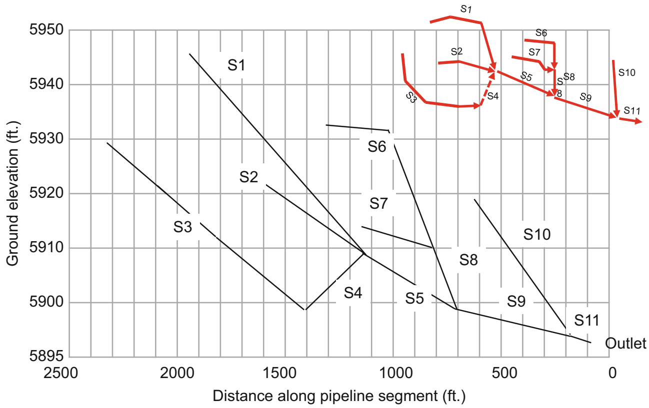


Fig. 5.11 Topographic cross-section of the 11 segments within a STEG system serving a development of 26 apartment buildings as shown in Fig. 5.9

5.23

■ Equivalent dwelling units (EDU)

- The EDU concept is used to normalize the discharges from different types of sources connected to a sewer
  - For example, there can be houses, restaurants, schools, etc.
  - An EDU is based on a selected average daily flow rate
    - Designers or authorities can decide on the gal/d per EDU
    - Based on Q_A for a DU, 150–250 gal/day per EDU is typical
- Calculating the number of EDUs
  - For a single building or other source, Eq. 5.1 is used

$$\mathrm{EDU} = \frac{Q_A}{Q_{EDU}} \tag{5.1}$$

Where:

EDU = number of equivalent dwelling units (no.)

Q_A = average daily flow from a building or other source (gal/d)

Q_{EDU} = defined daily flow per EDU (gal/d per EDU) (e.g., 150–250 gal/day)

5.24

- For multiple buildings or other sources, Eq. 5.2 can be used

$$N_{EDU} = \sum_{B=1}^{n} B = \left(\frac{Q_{A1} + Q_{A2} + \cdots Q_{An}}{Q_{EDU}}\right) \qquad (5.2)$$

Where:
N_{EDU} = number of equivalent dwelling units (no.)
B = building or other source
$Q_{A1...An}$ = average daily flow from a building or other source (gal/d)
Q_{EDU} = defined daily flow per EDU (gal/d per EDU)(e.g., 150–250 gal/day)

- Example calculation results are shown in Table 5.2

Table 5.2 Results of EDU calculations for several building and source conditions

Example building or source	Eq. no.	N_{EDU} w/Q_{EDU} = 150 gal/d	N_{EDU} w/Q_{EDU} = 250 gal/d
1 dwelling unit with Q_A = 300 gal/d	5.1	2	1.2
10 dwelling units with Q_A = 300 gal/d each	5.2	20	12
12 dwelling units (Q_A = 200 gal/d each) each plus 1 lodge that generates Q_A = 1500 gal/d	5.2	26	15.6
1 motel with Q_A = 1500 gal/d plus a restaurant with Q_A = 3000 gal/d	5.2	30	18

5.25

- Determining the number of EDUs for sizing a particular segment within a sewer system
  - EDUs for sizing a segment include the EDUs from any upstream segments plus those from buildings attached to the segment
  - Equation (5.3) can be used to calculate the number of EDUs contributing to the segment being sized

$$N_{EDU} = \sum_{S=1}^{n} \text{Upstream EDU} + \sum_{B=1}^{n} \text{Segment EDU} \qquad (5.3)$$

Where:
N_{EDU} = number of equivalent dwelling units contributing to a segment (no.)
Upstream EDU = EDUs contributing via an upstream segment ($S_{1...n}$)
Segment EDU = EDUs of buildings attached to the segment ($B_{1...n}$)
$B_{1...n}$ = building or other source connected to the segment being sized
$S_{1...n}$ = upstream segment(s) contributing to the segment being sized

5.26

- For the example segment shown in Fig. 5.12, $N_{EDU} = 134$ and this would be used to size the segment

$$N_{EDU} = \sum_{S=1}^{n} \text{Upstream EDU} + \sum_{B=1}^{n} \text{Segment EDU} \tag{5.3}$$

$$N_{EDU} = 100 + \left[\left(\frac{8 \times 400}{150}\right) + \left(\frac{600}{150}\right) + \left(\frac{2 \times 4 \times 150}{150}\right)\right] = 133.3 \Rightarrow 134$$

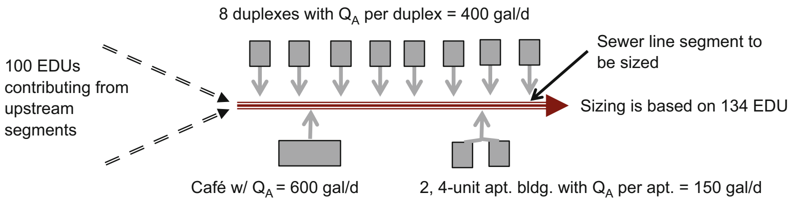


Fig. 5.12 Example calculation of the number of EDUs contributing to a sewer line segment where there are upstream EDUs contributing plus EDUs from buildings connected directly to the segment

5.27

- EDUs accumulate as you go from one segment to the next toward the outlet as illustrated in Fig. 5.13
  - EDUs accumulating in an effluent sewer system is analogous to water flows increasing as you go downstream in a river system

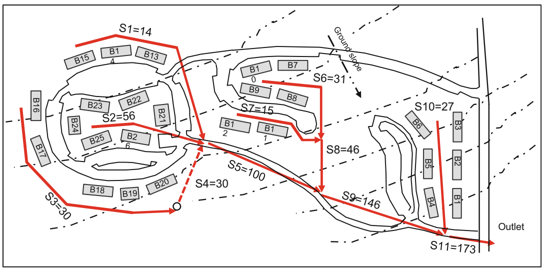


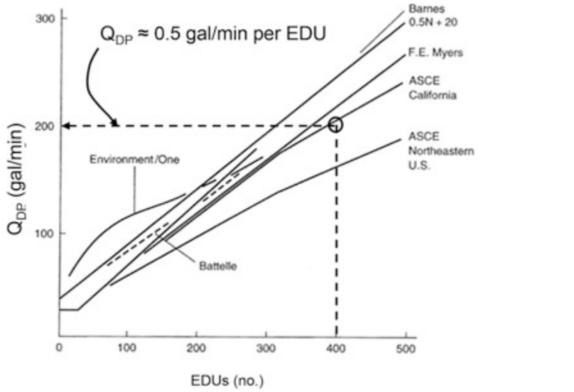
Fig. 5.13 Illustration of how EDUs accumulate in a STEG system serving a development of 26 apartment buildings as shown in Fig. 5.9. *Note*: There are 11 segments and the N_{EDU} shown are equal to the accumulated number at the end of each segment including the N_{EDU} from all upstream segments plus the N_{EDU} from buildings connected to the segment

5.28

- Q_{DP} values for sizing a segment
  - Q_{DP} = design flow rate (gal/min) for sizing a segment if it has a certain number of EDUs contributing to it
  - Q_{DP} values can be affected by the type of sewer system, e.g.:
    - STEG systems
      - * Effluent leaves a septic tank by gravity and enters the sewer
      - * Discharges from a septic tank are usually <1 gal/min, often in the 0.3–0.6 gal/min range, with periods of zero flow
    - STEP systems
      - * Effluent is periodically pumped into the sewer
      - * In STEP systems, commonly used pumps discharge in the range of 5–20 gal/min
      - * Each connection has long periods of zero flow with periodic bursts of 5–20 gal/min flow
      - * Need to consider the particular pump's discharge rate plus the probability of multiple pumps discharging simultaneously

5.29

- Q_{DP} values for sizing alternative sewer systems
  - * Data has been obtained by monitoring operating STEP systems serving residential developments and small towns (Fig. 5.14)



Notes:
- These Q_{DP} data are for STEP systems.
- Data are based on monitoring of systems with older water-using fixtures and appliances contributing to the total wastewater flows.
- It is uncertain how this might change for sources with modern water efficient plumbing or just graywater

Fig. 5.14 Q_{DP} values versus cumulative EDUs. *Source*: Fig. 2.12 in USEPA 1991 shown as Fig. 6.3. Crites and Tchobanoglous 1998

5.30

- Estimating Q_{DP} for sizing a segment
  - Q_{DP} might be estimated from monitoring data for existing systems
  - It is more common to make predictions of Q_{DP} using Eq. 5.4

$$Q_{DP} = Q_{MIN} + Q_{EDU}(N_{EDU}) \tag{5.4}$$

Where:

Q_{DP} = design peak flow at a location in the sewer system (gal/min)

Q_{MIN} = minimum discharge rate from one EDU (gal/min)

= values of 10–20 gal/min are often used to account for the peak flow rate from one or a few buildings

Q_{EDU} = discharge per EDU for the development size (gal/min per EDU)

= values of 0.5 gal/min are typically used for systems with 50 or more connections

N_{EDU} = number of EDUs contributing to Q_{DP} at a particular location

Notes: The actual Q_{MIN} for a STEG system equals the likely peak rate of gravity discharge from a single septic tank that can be 1 gal/min or less depending on the building being served. The actual Q_{MIN} for a STEP or grinder pump system is often in the 10 to 25 gal/min range based on the type of pump used and its discharge rate. Also, flow controllers in a discharge line can maintain the pump discharge in the range of 10 gal/min. The actual Q_{MIN} from a vacuum vault can be 100 gal/min but this flow rate is rapidly attenuated in the vacuum sewer line.

5.31

- Examples of Q_{DP} for sizing sewer line segments with varied numbers of EDUs contributing
- The Q_{DP} values shown in the Table 5.3 are calculated using Eq. 5.4

Table 5.3 Examples of Q_{DP} values for sewer line segments with varied EDUs

Number of EDUs contributing to a sewer line segment[a]	Estimate of Q_{DP} (gal/min)	
	Q_{DP} parameter values[b]	Peak flow rate
1	$Q_{MIN} = 15$	15.5
10	$Q_{EDU} = 0.5$	20
20		25
50		40
100		65
500		265
1000		515

[a]EDUs contributing includes the EDUs from upstream segments plus those for buildings connected to the segment being designed.
[b]Q_{DP} parameter values are given as examples only.

5.32

- Hydraulics of flow to handle the design peak flow
  - Calculating velocity and head loss
    - Velocity calculations can be made using the Hazen-Williams equation (Eq. 5.5) with substitutions for R (Eq. 5.6) and S (Eq. 5.7) for round pipes flowing full:

$$V = 1.318(C)\left(R^{0.63}\right)\left(S^{0.54}\right) \tag{5.5}$$

$$R = \frac{D}{4} \qquad S = \frac{h_f}{L} \tag{5.6,5.7}$$

Where:
V = velocity of flow (ft/s)
C = Hazen-Williams coefficient (–) (depends on pipe type and condition: $C = 150$ for new PVC pipe; $C = 120$–140 is used to account for aging)
R = hydraulic radius (flow area divided by wetted perimeter) (ft)
S = slope of energy grade line (ft/ft)
D = true inside pipe diameter (ft)
h_f = head loss due to friction (ft)
L = length of pipeline (ft)

5.33

- Head loss calculations
  * To calculate head loss (h_f) due to flow in a pipeline segment, flow rate needs to be calculated (Eq. 5.8)

$$Q_{CAP} = (V) \times (A) = \left[(1.318C)\left(\frac{D}{4}\right)^{0.63}\left(\frac{h_f}{L}\right)^{0.54}\right] \times \left(\frac{\pi D^2}{4}\right) \tag{5.8}$$

Where:
Q_{CAP} = flow rate capacity for a given pipe size and EGL slope (ft^3/s)
V = flow velocity (ft/s)
A = pipe inside cross-sectional area (ft^2)
D = true inside pipe diameter (ft)
h_f = head loss due to friction (ft)
L = length of pipeline (ft)
C = Hazen-Williams coefficient (–) (depends on pipe type and condition: $C = 150$ for new PVC pipe; $C = 120$–140 is used to account for aging)

5.34

* Head loss (h_f) can be calculated by rearranging Eq. 5.8 to yield Eq. 5.9:

$$h_f = 4.72(L)\left(\frac{Q_{CAP}}{C}\right)^{1.85}(D)^{-4.87} \tag{5.9}$$

Equation (5.10) applies when the flow rate is in gal/min and the pipe diameter is in inches:

$$h_f = 10.5(L)\left(\frac{Q_{CAP}}{C}\right)^{1.85}(D)^{-4.87} \tag{5.10}$$

Where:

h_f = head loss (ft)

L = length of pipeline segment (ft)

Q_{CAP} = flow rate capacity for a given pipe size and EGL slope (gal/min)

C = Hazen-Williams coefficient (–) (depends on pipe type and condition: C = 150 for new PVC pipe; C = 120 to 140 is used to account for aging)

D = true inside pipe diameter (in.)

5.35

- Selection of a pipe type and diameter for a segment
  - Tables and charts relate Q_{CAP} for pipe sizes to S and V
    * For example, if sizing a STEP segment where Q_{DP} = 60 gal/min, new 3-in. diameter Class 200 pipe appears okay since S = 0.7 % and V = 2.45 ft/s (Table 5.4)

Table 5.4 Q_{DP} values for different slope and velocity values to aid initial pipe size selection

Nominal pipe size (in.)	Class 200 ID (in.)	Cross-section area (ft^2)	R = D/4 (ft)	Velocity (ft/s) at EGL S (%):			Capacity Q_{CAP} (gal/min) at EGL S (%):		
				0.1	0.7	2.0	0.1	0.7	2.0
1	1.189	0.0077	0.0248	0.46	1.32	2.33	1.6	4.6	8.1
1.5	1.720	0.0161	0.0358	0.58	1.66	2.93	4.2	12	21
2	2.149	0.0252	0.0448	0.67	1.92	3.38	7.6	22	38
2.5	2.601	0.0369	0.0542	0.76	2.16	3.81	12	36	63
3	3.166	0.0547	0.0660	0.86	2.45	4.31	21	60	106
4	4.072	0.0904	0.0848	1.00	2.87	5.05	41	116	205
6	5.993	0.1959	0.1249	1.28	3.66	6.45	113	322	567
8	7.805	0.3322	0.1626	1.51	4.32	7.62	225	644	1135

Note: Velocity and capacity calculations were made using Eqs. 5.5 and 5.8 with C = 150.

5.36

- ○ Calculations for pipe sizing need to account for pipe attributes
  - * Need to use true inside diameters which vary based on the nominal size and type of pipe (Table 5.5)
  - * Need to use roughness coefficients that reflect pipe aging
    - – For example use Hazen-Williams C values of 120 to 140 rather than 150, which applies to new plastic pipe

Table 5.5 True inside diameters of pipes of different types and sizes

Nominal pipe size (in.)	Outside diameter (in.)	True inside diameter (in.)		
		Schedule 40 PVC	Schedule 80 PVC	Class 200 PVC
1	1.315	1.049	0.957	1.189
1.5	1.900	1.610	1.500	1.720
2	2.375	2.067	1.939	2.149
2.5	2.875	2.469	2.323	2.601
3	3.500	3.068	2.900	3.166
4	4.500	4.026	3.826	4.072
6	6.625	6.065	5.761	5.993
8	8.625	7.981	7.625	7.805

5.37

- Hydraulic and energy grade lines
  - ○ Hydraulic grade line (HGL)
    - * Represents the potential energy of a liquid in a pipeline
  - ○ Energy grade line (EGL)
    - * Represents potential plus kinetic energy along a pipeline
    - * Kinetic energy is determined by the velocity of flow (V)
    - * In most systems, V is low and contributes little to the EGL
      - – For example with $V = 10$ ft/s, the velocity head $= 1.5$ ft.
  - ○ For alternative sewers, the HGL $\approx$ EGL
    - * Velocity head can usually be ignored
    - * STEG systems—HGL is fixed based on ΔElevations
    - * STEP, grinder pump, vacuum systems—HGL depends on pump or vacuum discharge characteristics
  - ○ During flow in a pipe, friction causes head losses that reduce the head available for flow

5.38

- Head loss during flow at different rates in different pipe sizes is shown in Table 5.6
  - For example: In a new 3-in. pipeline at S = 0.7 %, the Q_{CAP} = 60 gal/min and the headloss h_f = 7.0 ft per 1000 ft of length

Table 5.6 Headloss during flow in a full section of a sewer pipe

Nominal pipe size (in.)	Class 200 PVC ID (in.)	Cross-section area (ft^2)	Capacity Q_{CAP} (gal/min) at EGL S (%):			Headloss h_f at the flow rate listed	
			0.1 %	0.7 %	2.0 %	Q (gal min^{-1})	h_f (ft/1000 ft)
1	1.189	0.0077	1.6	4.6	8.1	5	8.4
1.5	1.720	0.0161	4.2	12	21	15	10.5
2	2.149	0.0252	7.6	22	38	25	9.2
2.5	2.601	0.0369	12	36	63	40	8.6
3	3.166	0.0547	21	60	106	60	7.0
4	4.072	0.0904	41	116	205	120	7.4
6	5.993	0.1959	113	322	567	325	7.2
8	7.805	0.3322	225	644	1135	650	7.1

Note: Head loss calculations were made using Eq. 5.10 with C = 150.

5.39

- Development of the EGL
  - An EGL can be developed for STEG, STEP and grinder systems but vacuum systems are more complicated due to the flow regime
  - The EGL is based on calculated head losses (h_f) during flow in the segments of the STEG, STEP or grinder pump system
    - For example, for a 3 segment system where segment 1 leads to the outlet:
      - The h_f required to transport Q_{DP} in Segment 1 is added to the elevation of the static head, which equals the outlet elevation
      - The h_f for Q_{DP} in Segment 2 is added to the elevation of the upstream end of Segment 1
      - The h_f for Q_{DP} in Segment 3 is added to the elevation of the upstream end of Segment 2
      - This process is used to establish the EGL pressure head in the main sewer line and in STEP or grinder pump systems, the total dynamic head (TDH) required for a pumping unit
  - Examples of EGLs for a STEG system and STEP system are shown in Figs. 5.15 and 5.16

5.40

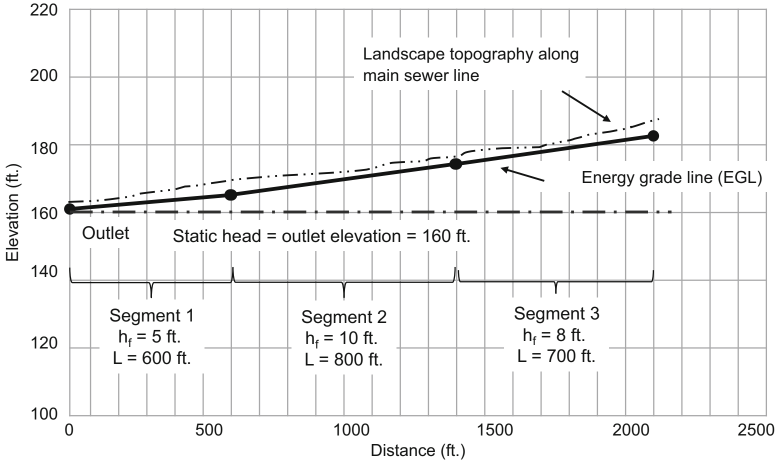


Fig. 5.15 Illustration of an EGL along 3 segments of a STEG system

5.41

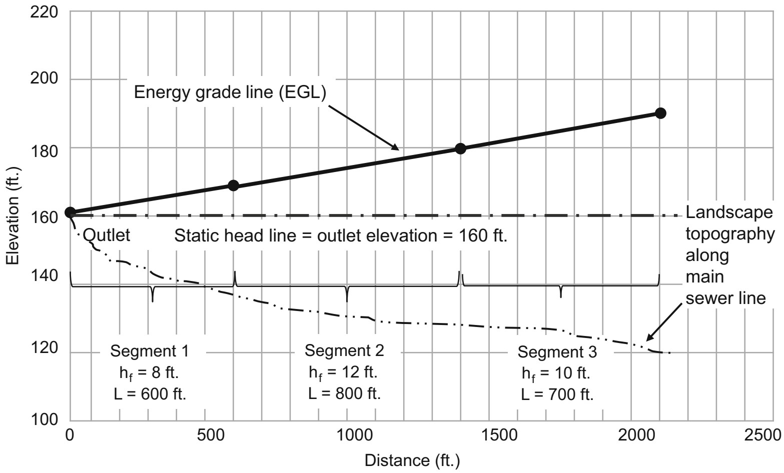


Fig. 5.16 Illustration of an EGL along 3 segments of a STEP system

5.42

- Pumps used in STEP or grinder pump systems
  - Type of pump
    - * STEP systems and grinder pump systems utilize high-head, low-flow rate pumps
    - * This ensures the pumps can pressurize the sewer system at levels sufficient to transport the Q_{DP} in the main sewer lines and achieve needed velocities
    - * Often one type and size of pump can be used throughout a sewer system serving a development
  - Pumps operate intermittently, based on float controls
    - * Pumps may cycle on 3 to 5 +/− times a day
    - * Discharge of 50 gal will draw down the water surface in a tank with 20 ft^2 of surface area by only 4 in.
  - Pumps used in STEP systems are selected and set up to discharge at low rates (e.g., 5–20 gal/min)
    - * Low pumping rates out of the septic tanks help ensure limited turbulence in the tank and minimum solids carry over

5.43

- The TDH for a pumping unit is illustrated in Fig. 5.17 and calculations can be made using Eq. 5.11

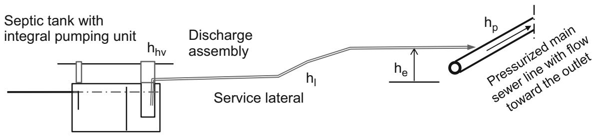


Fig. 5.17 Illustration of TDH components in a STEP system

$$\mathrm{TDH} = h_p + (h_e + h_{hv} + h_l) \tag{5.11}$$

Where:

TDH = total dynamic head for a particular pumping unit (ft)

h_p = pressure head needed to transport Q_{DP} flow in the main sewer line at the point where the service lateral connects to it (ft)

h_e = change in elevation between the water level in the septic tank and the service lateral connection point at the main sewer line (ft)

h_{hv} = friction losses in the discharge assembly at a pumping unit (ft)

h_l = friction losses in the service lateral (ft)

Note: $h_{hv} + h_l$ are ~5 to 10 ft typ.

5.44

- The TDH that a pump could have to discharge against along 3 segments of a STEP system is illustrated in Fig. 5.18

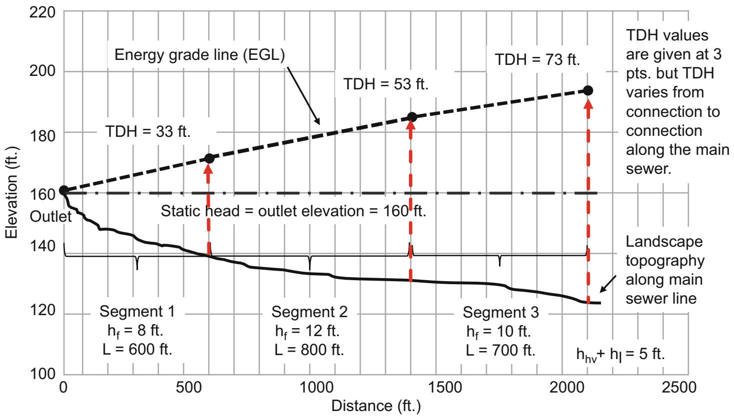
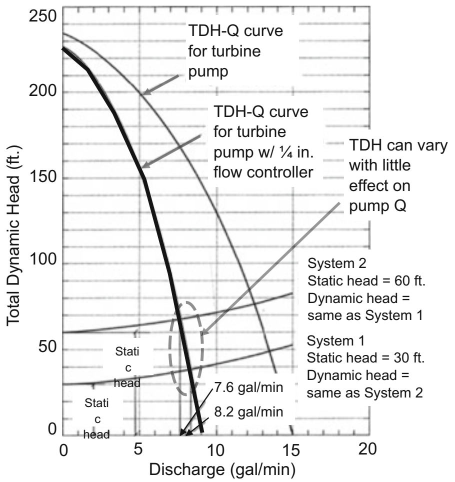


Fig. 5.18 Illustration of TDH along 3 segments within a STEP system

5.45

- A representative head-discharge curve for a high head pump used in a STEP system is shown in Fig. 5.19

Fig. 5.19 Illustration of the head—discharge curve for a ½-hp high head submersible pump such as used in a STEP system. *Note*: Pumps are typically 0.5 to 1.5 hp. The pump discharge can be limited to 5 to 10 gal/min using a 1/4-in. flow controller in the discharge line. Also, a 1/8-in. bypass hole allows expulsion of trapped air and recirculation when the main sewer line pressure is too high. *Source*: From Orenco Systems®, Inc. shown as Fig. 6.9 in Crites and Tchobanoglous 1998



5.46

5-3. Design and Implementation

- Design and implementation (D&I) considerations for STEG and STEP systems
  - Development features
  - Typical design parameter values
  - Type of pipe
  - Approach to pipeline sizing
  - Pipe size suitability
  - System components
  - Hybrid systems
  - Special considerations
  - System installation
  - System O&M

Note: This section is focused on STEG and STEP systems since they are most widely used in decentralized infrastructure applications. Elements of the design process used for STEP systems also applies to grinder pump systems but vacuum sewer systems are more complicated. Design and implementation of STEP, grinder pump, and vacuum sewer systems is often completed with the involvement of one of the major technology vendors and equipment manufacturers (e.g., Orenco Systems®, Inc., Environment One, AIRVAC®, respectively).

5.47

- D&I considerations—Development features
  - Land use and development attributes
    - Type and number of buildings and wastewater sources
    - Density of the development
    - Planned development growth in the future, if any
    - Location of the site for treatment and discharge or reuse
  - Topography
    - Level or gently sloping vs. hilly and mountainous terrain
  - Subsurface characteristics
    - Depth of soil and presence of shallow bedrock
    - Depth to ground water
    - Depth of freezing zone (if applicable)

5.48

■ D&I considerations—Typical design parameters

- Listed in Table 5.7 are the range of values that can be used for different design parameters for STEG and STEP systems

Table 5.7 Representative values for different design parameters for STEG and STEP systems

Design parameters	STEG	STEP
Service connection discharge rate (gal/min)	0–1.0	0 to 5–20[a]
Q_{DP} for segment sizing based on N_{EDU} (gal/min) (Eq. 5.4)	$15+0.5(N_{EDU})$[b]	$15+0.5(N_{EDU})$
Service lateral pipeline diam. (in.)	1.5–2.0	1.0–2.0
Main sewer pipeline diameter (in.)	3–8	2–8
Trench depth (ft)[c]	2–3	2–3
Cleanouts	Located at the end of pipelines or at major changes in pipeline size	

[a]Discharge rate is determined by the type and size of pump used in the STEP system.
[b]Designers often set Q_{MIN} equal to the same value for both STEG and STEP systems (e.g., $Q_{MIN} = 15$ gal/min) even though a septic tank serving a single house will typically discharge at 1 gal/min or less. However, for septic tanks receiving higher flows (e.g., apartment building or commercial establishment) discharge rates can be higher. Also using a smaller value for Q_{MIN} (e.g., 2 gal/min) would not change the pipe sizing due to minimum pipe sizes used.
[c]Minimum trench depth is based on climate and frost depth unless insulated or heat-traced piping is used. Maximum trench depth is based on pipe type and strength.

5.49

■ D&I considerations—Type of pipe

- Different types of plastic pipe can be used in STEG and STEP systems (Table 5.8)

Table 5.8 Features of different types of plastic pipe used for STEG and STEP systems

Nominal pipe diameter[a] (in.)	Type of plastic pipe used[a]
< 3 in.	Schedule 80 PVC is used but Class 200 PVC pipe is also commonly used
> 3 in.	Class 200 PVC is typically used including locations with shallow burial depths or difficult access

[a]Nominal outside diameter is the same for all types of PVC pipe, however the wall thickness varies. For example, Schedule 80 PVC has a thicker wall than Schedule 40 PVC and so the inside diameter of Schedule 80 PVC is less than Schedule 40 PVC. High density polyethylene pipe is also available.

5.50

- D&I considerations—Approach to pipeline sizing
  - STEG systems
    - EGL slope is determined by gravity given the site topography
    - Pipe diameters have to be large enough so that under a given S the sewer line Q_{CAP} exceeds the Q_{DP}
    - Typically the Q_{DP} will be only a fraction of the capacity available (Q_{CAP}), so at times the pipe may not flow full
  - STEP systems
    - EGL slope is determined by the pumping units selected to transport the Q_{DP} and the TDH during system operation
    - Typically, the Q_{CAP} will be equal to the design peak flow rate the system is designed for (i.e., Q_{DP}) and the pipe will flow full

5.51

- Steps followed during sizing a segment in a STEG system
  - Determine the N_{EDU} contributing and calculate the Q_{DP}
  - Determine the length of the sewer line segment
  - Determine the drop in elevation and slope
  - Select a trial pipe diameter to handle Q_{DP} at S and suitable V
  - Calculate the velocity assuming the pipe is flowing full (check that V is near or < 5 ft/s)
  - Calculate the pipe cross-sectional area
  - Calculate the pipe capacity (Q_{CAP}) when flowing full
  - Compare Q_{DP} to the flowing full capacity, Q_{CAP}
    - $Q_{DP}/Q_{CAP} < 1$: pipeline segment flows only partially full
    - $Q_{DP}/Q_{CAP} > 1$: surcharge flow occurs during peak Q
    - $Q_{DP}/Q_{CAP} >> 1$: repeat calculations with a larger pipe diameter

5.52

- Steps followed during sizing a segment in a STEP system
  - Determine the N_{EDU} contributing and calculate Q_{DP}
    - Note that the required $Q_{CAP} = Q_{DP}$
  - Determine the length of the sewer line segment
  - Select a trial pipe diameter to handle Q_{DP} at suitable S and V
  - Calculate the slope of the energy grade line (EGL) for the pipe segment (confirm that S is reasonable, e.g., 0.5–1.5 %)
  - Calculate the pipe cross-sectional area
  - Determine the velocity by dividing the Q_{CAP} by the cross-sectional area (confirm that V is reasonable, near or $<$ 5 ft/s)
  - Calculate the head loss due to friction (h_f) based on S and L
  - Plot the EGL on the system profile to determine the TDH each pump would have to discharge against

5.53

- D&I considerations—Pipe size (diameter) suitability
  - Suitability based on EGL slope
    - For many systems, designs can yield an EGL $S = 0.5$–1.5 %
      - Minimum slopes
        - The system EGL S to the outlet must be $>$ 0.0 %
        - If S for a segment is too low, it could indicate the pipe size is bigger than needed
      - Maximum slopes
        - For some STEG systems, S can be high (e.g., $>$1.5 %)
        - If S is too high in STEP systems, it indicates the pipe diameter is too small and energy due to pumping is being wasted
    - S can be controlled by topography
      - For a STEG system, S is largely controlled by landscape topography and it can be $>$1.5 %
      - For a STEP system, topography will not control S since pumps can be selected to yield a desired S

5.54

- Suitability based on flow velocity (V)
  - For many systems, designs can yield $V < 5$ ft/s
    - Minimum velocities
      - Technically a minimum V is not required
      - But some regulatory agencies can require one

        e.g., V minimum = 1.0–1.5 ft/s during peak Q
    - Maximum velocities
      - To avoid excessive friction losses and damage to fittings and valves, particularly in STEP systems, try to keep $V < 5$ ft/s
  - V can be affected by topography
    - For STEG systems, V can be controlled in part by landscape topography which controls S
    - For STEP systems, topography won't control V since pumps can be selected to yield a desired S and V

5.55

- D&I considerations—System components
  - STEG system components
    - Onsite components
      - Septic tank (potentially including an effluent screen)[a]
      - Service laterals from each septic tank to the sewer main
    - Collection system components
      - Check valves and shutoff valves on the service lateral
      - Small diameter gravity flow sewer line segments
      - Cleanouts at certain locations
      - Vents and and combinations of air release/vacuum valves
      - Corrosion and odor control options

[a]Septic tank units are described in Chap. 6. One septic tank can serve multiple buildings. If septic tanks are already existing but are old, they may need to be replaced with watertight, properly sized and installed units.

5.56

- An illustration of STEG onsite system components in Fig. 5.20

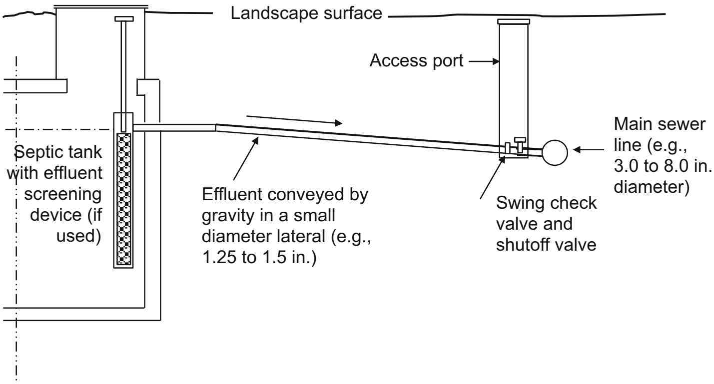


Fig. 5.20 Illustration of the onsite system components of a STEG system

5.57

- STEP system components
  - Onsite components
    * Septic tank with effluent screen[b]
    * Pumping unit with pressurized lateral to a sewer main
  - Collection system components
    * Check valves and shutoff valves on the service lateral
    * Small diameter pressurized sewer lines
    * Cleanouts at certain locations
    * Valves placed throughout the system
      – Air release valves
      – Pressure sustaining valves
    * Corrosion and odor control options

[b]*Notes*: One septic tank can serve multiple buildings. If septic tanks are already existing but are old, they may need to be replaced with watertight, properly sized and installed units.

5.58

- Illustrations of STEP onsite system components are shown in Figs. 5.21 and 5.22

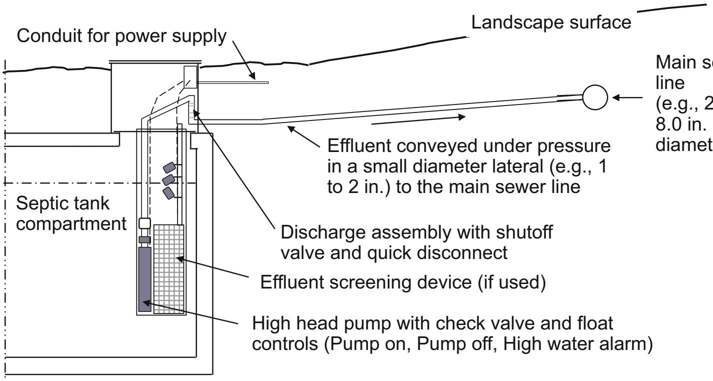


Fig. 5.21 Illustration of the onsite system components of a STEP system (Orenco Systems®, Inc.)

5.59

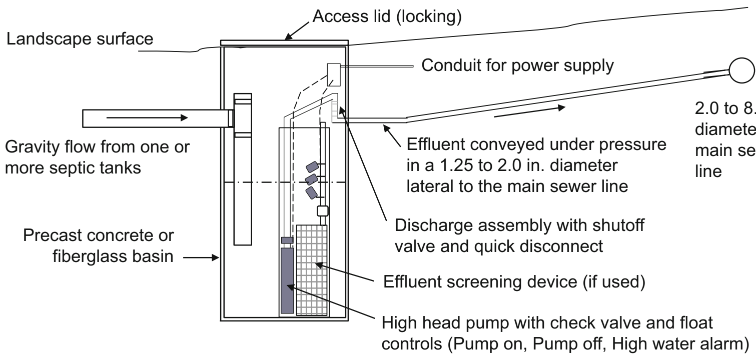
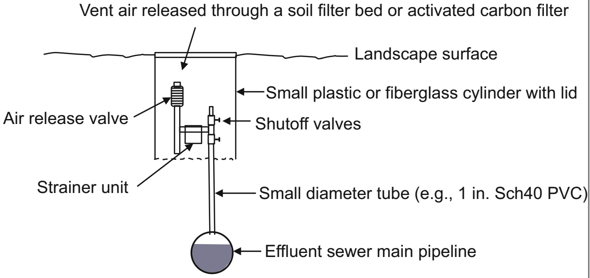


Fig. 5.22 Illustration of an external pumping unit for a STEP system

5.60

- Collection system components
  - Air release valves (Fig. 5.23)
    - * Used at high points to release air that can accumulate
  - Pressure sustaining valves
    - * Used to maintain upstream static pressures in those portions of a STEP system which are higher in elevation than the outlet

Fig. 5.23 Illustration of an air release valve setup



5.61

- Cleanouts
  - * In STEG or STEP systems cleanouts can facilitate cleaning of one or more pipeline segments if needed (Fig. 5.24)
  - * However, cleaning has rarely been required even in systems that have been operating for decades
  - * Cleanouts are typically installed at just a few locations, e.g.:
    - – Ends of terminal pipe segments
    - – Pipe junctions or pipe size changes

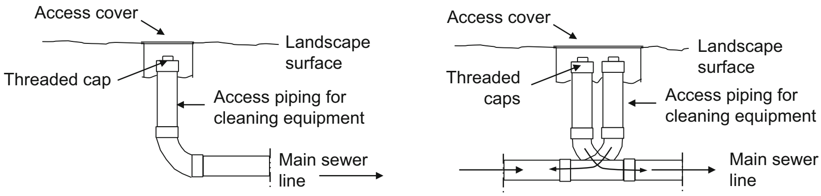


Fig. 5.24 Illustration of cleanouts used in STEG systems at the end of a terminal segment (*left*) and along a segment (*right*)

5.62

- D&I considerations—Hybrid systems
  - Developments can be served by individual STEG or STEP systems or hybrids
  - Examples of hybrid systems include:
    - A STEG system used for most of a subdivision development with a STEP system used for part of it (based on area-wide topography), the combination of which are conveyed to the treatment site
    - A STEG or STEP system, which discharges into a conventional sewer system with gravity collection and conveyance of STE and raw wastewater to the treatment site
    - A STEG system with a pump station at the outlet and pressurized force main that conveys STE to the treatment site

5.63

- D&I considerations—Special considerations
  - Air binding and air release valves
    - Air can become entrapped in STEG or STEP sewers based on the variable grades that are used
    - Air release valves are important to mitigate the air entrapment and its effects on flow capacity
    - Air release valves are typically needed at high points of sewer lines where air can accumulate
    - Air release valves can be manual or automatic
    - The air released through an air release valve is typically treated in a soil bed or activated carbon filter before release to the atmosphere
    - Air release valves are typically inspected on an annual basis

5.64

- Corrosion and odor
  - STE that is conveyed in STEG or STEP sewers is low in dissolved oxygen and can contained reduced compounds such as H_2S
  - When STE contacts air—such as during flow through a manhole and connection to a conventional gravity sewer or discharge into a treatment unit—there is potential for both corrosion and odor (sulfur compounds are often involved)
  - Corrosion control options include:
    * Limit exposure to air
    * Provide corrosion resistance equipment such as plastic, stainless steel, or coated products
      – Corrosion protection is especially important for electrical wiring
  - Odor control devices can also be used

5.65

- D&I considerations—Installation
  - Installation practices are illustrated in Figs. 5.25, 5.26 and 5.27

Fig. 5.25 Photographs of effluent sewer main installation using a continuous trencher (*left*) and directional drilling (*right*) methods. (Photographs courtesy of R. J. Otis)

5.66

Fig. 5.26 Photographs of a septic tank/pump vault installation along a lakeshore development (*left*) and a service lateral run from a residence to the main sewer line (*right*). (Photographs courtesy of R. J. Otis)

5.67

Fig. 5.27 Photograph of insulated pipe used for shallow burial of sewer pipe in a cold climate. (Photograph courtesy of R. J. Otis)

5.68

- D&I consideration—Operation and maintenance
  - For STEG and STEP systems
    - Septic tank operation and maintenance (refer to Chap. 6)
      - Inspect tanks and measure solids accumulation at 1–3 years to estimate a solids removal frequency
        - Solids that are removed periodically need to be properly managed
      - Screening devices (if used) need to be cleaned annually
    - Cleaning of a segment of the sewer system could be required to remove accumulating solids, though this is rarely needed
  - For STEP systems (in addition to the above)
    - The pumping equipment needs attention if an alarm condition occurs due to loss of power, blockage or other cause
    - Pump replacement is needed at a frequency based on the pump design life (e.g., 20 years)

5.69

5-4. Summary

- Decentralized infrastructure can require collection and conveyance of wastewaters
  - Alternative sewer systems convey septic tank effluent from one or multiple sources under gravity (STEG) or pressure (STEP) or convey raw wastewater using grinder pumps and pressurized sewers or without grinding in vacuum sewers
- STEG or STEP systems are widely used and offer potential benefits
  - Use passive primary treatment of raw wastewater at a source
  - Convey STE in small-diameter pipe with watertight joints laid at variable grades in shallow trenches without manholes
  - Enable clustering of multiple sources and more efficient treatment and discharge or reuse based on economies of scale

5.70

5-5. Example Problems

- 5EP-1. Design of a STEG system for Mines Park
  - Given information
    - A housing development is located on the Colorado School of Mines campus in Golden, Colorado
      - * The development characteristics are revealed in an aerial photograph (Fig. 5EP.1) and a topographic base map
        - – There are 26 buildings with different numbers of apartments and occupants (Table 5EP.1)
      - * The effluent collected from the buildings will be discharged at the outlet shown to a membrane bioreactor with disinfection for nonpotable reuse within the development
    - One EDU is defined as $Q_A = 150$ gal/day
  - Determine
    - Layout a STEG collection system for the development and complete the sizing calculations for Segment 1 of the system

5.71

Fig. 5EP.1 Aerial photograph of the Mines Park housing development located on the campus of the Colorado School of Mines in Golden, Colorado USA

5.72

- Solution
  - Assumptions made:
    - * Building information is shown in Table 5EP.1
    - * Assume Q_A per DU = 80 % of 69.2 + 37.2N_P/DU (Eq. 3.2) where 80 % is used here since there are no clothes washers in the DUs
    - * Use $Q_{DP} = 15 + 0.5N_{EDU}$ for all STEG segments
    - * Use Schedule 40 PVC pipe with C = 130 to account for aging; minimum pipe size = 2 in. (true ID = 2.067 in.)

Table 5EP.1 Building information for the Mines Park development

Building no.	DUs per building	Bedrooms per DU	Persons per DU	gal/d per DU	gal/d per bldg	Q_{EDU} (gal/d)	EDU per building
B1–B6	8	1	1	85.1	680.8	150	4.5
B7–B12	8	2	3	144.6	1156.8		7.7
B13–B20	8	2	2	114.9	919.2		6.1
B21–B26	8	3	4	174.4	1395.2		9.3

5.73

- Layout a STEG collection system divided into segments
  - * One proposed layout is shown in Fig. 5EP.2

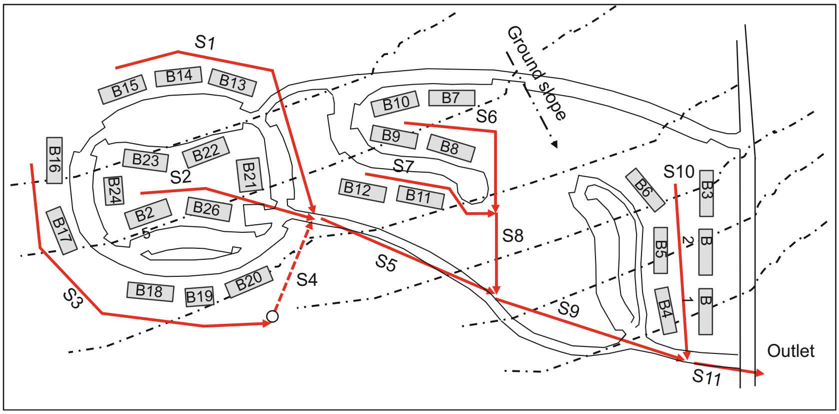


Fig. 5EP.2 STEG collection system layout for the Mines Park development. (*Note*: there are other layouts that could also work.)

5.74

- Contributing EDUs and design peak flows for each segment
  - Determine the N_{EDU} contributing based on building information and layout (Table 5EP.1)
    - N_{EDU} = upstream EDUs plus segment buildings EDUs
  - Determine the design peak flows using Eq. 5.4 with Q_{MIN} = 15 gal/min and Q_{EDU} = 0.5 gal/min (Table 5EP.2)

$$Q_{DP} = 15 + 0.5(N_{EDU}) \tag{5.4}$$

Table 5EP.2 EDUs contributing to each STEG segment in the Mines Park development

Seg. no.	Bldgs.	N_{EDU}	Q_{DP} (gal/min)	Seg. no.	Bldgs.	N_{EDU}	Q_{DP} (gal/min)
1	B13–B15	18.3	24.2	7	B11–B12	15.4	22.7
2	B21–B26	55.8	42.9	8	B7–B12	46.2	38.1
3	B16–B20	30.5	30.2	9	B7–B26	150.8	90.4
4	B16–B20	–	–	10	B1–B6	27	28.5
5	B13–B26	104.6	67.3	11	B1–B26	177.8	103.9
6	B7–B10	30.8	30.4				

5.75

- Complete the pipeline sizing calculations for Segment 1
  - Segment 1 is highlighted in Fig. 5EP.3

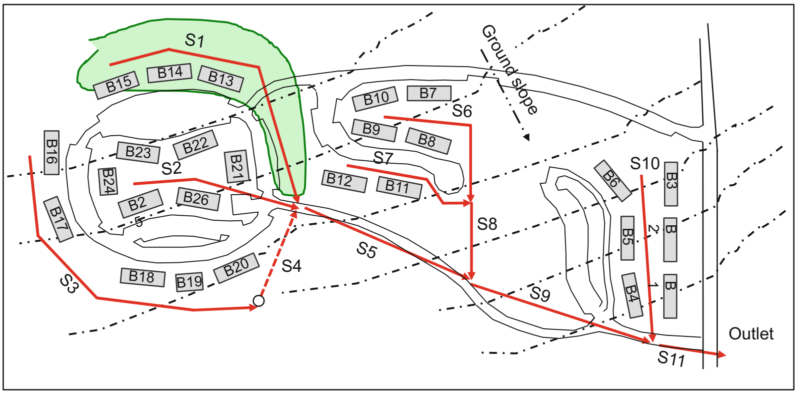


Fig. 5EP.3 STEG system layout for the Mines Park development with Segment 1 highlighted. (*Note*: there are other layouts that could also work.)

5.76

* Select a trial pipe diameter for Segment 1
  – Use the calculated Q_{DP} for Segment 1 (Table 5EP.2)
  – Using a topographic base map:
    Determine the length of Segment 1
    Determine the elevation drop over the pipe length
  – Calculate the slope available
  – Select a pipe diameter that appears able to handle Q_{DP} (for example using Table 5.4)

Table 5EP.3 Design peak flow and slope applicable for Segment 1 leading to a trial pipe diameter

Segment no.	Cum. EDUs	Q_{DP} (gal/min)	Length (ft)	Elev. drop from upstream end to downstream end (ft)	Slope (S) (ft/ft)	Trial pipe diam. (in.)
1	18.3	24.2	810	5947 to 5910 = 37	0.046	2.0 (True ID = 2.149)
Basis	Table 5EP.2	Table 5EP.2	Estimated based on a topo-graphic base map		Calculated from the elevation drop	Table 5.4 based on Q_{DP} and S

5.77

* Calculate the velocity of flow in Segment 1
  – Use the slope and trial pipe diameter shown in Table 5EP.3 and the Hazen-Williams equation (Eq. 5.5)

$$V = 1.318(C)\left(R^{0.63}\right)\left(S^{0.54}\right) \tag{5.5}$$

$$V = 1.318(130)\left[\left(\frac{2.067/12}{4}\right)^{0.63}\right](0.046)^{0.54}$$

$$V = 1.318(130)(0.138)(0.1896)$$

$$V = 4.48\ \text{ft/s}$$

5.78

* Calculate the segment capacity, Q_{CAP}, and compare it to the design flow rate, Q_{DP} (Table 5EP.4)

$$Q_{CAP} = V \times A \tag{5.8}$$

$$Q_{CAP} = (4.48\ \text{ft./s})\left(\frac{3.63\ \text{in}^2}{144 in./\text{ft}^2}\right)$$

$$Q_{CAP} = 0.113\ \text{f}t^3/\text{s}$$

$$Q_{CAP} = 0.113\ \text{ft}^3/\text{s}\left(\frac{449\ \text{gal/min}}{1\ \text{ft}^3/\text{s}}\right) = 50.7\ \text{gal/min}$$

Table 5EP.4 Segment 1 flow rate capacity compared to the design peak flow contributing to it

Seg. no.	Q_{DP} (gal/min)	Slope (ft/ft)	Velocity (ft/s)	Cross-sectional area (in^2)	Q_{CAP} (gal/min)	Ratio of Q_{DP} to Q_{CAP} (–)
1	24.2	0.046	4.48	3.63	50.7	0.48

5.79

* Assessing suitability of pipeline sizing
  – A 2-in. diameter pipe was selected for Segment 1 since:
    It appears to have the capacity to handle the Q_{DP} in the normal range of S (0.5–1.5 %) and V (<5 ft/s)
    2-in. diameter pipe is a minimum size often used for STEG mains
  – The slope determined for the Segment 1 was higher than the normal 0.5–1.5 % due to topography at the site
  – Due to the high slope and 2-in diameter size, the pipeline segment V is high (5.2 ft/s)
* Suitability?
  – Reducing the diameter (say to 1.5 in.) would reduce V and A and reduce Q_{CAP} and thereby increase the utilization of the pipeline capacity (i.e., increase the ratio of Q_{DP} to Q_{CAP} from 0.48 toward 1.0)
  – But, given the 2-in. diameter minimum sizing, pipeline sizing for Segment 1 is okay (even though it is somewhat oversized)

5.80

- 5EP-2. Design of a STEP system for Mines Park
  - Given information
    - A housing development is located on the Colorado School of Mines campus
      - The development characteristics are revealed in an aerial photo (Fig. 5EP.1) and a topographic base map
      - There are 26 buildings with different numbers of apartments and occupants (Table 5EP.1)
      - The effluent collected from the buildings will be discharged at the outlet to a soil-based treatment system located in the open space
    - One EDU is defined as $Q_A = 150$ gal/day
  - Determine
    - Layout a STEP collection system for the development and complete the sizing calculations for Segment 1 of the system

5.81

- Solution[c]
  - Layout a STEP collection system divided into segments (Fig. 5EP.4)

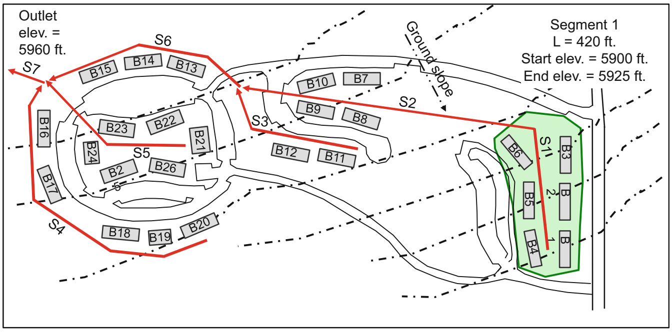


Fig. 5EP.4 STEP collection system layout for the Mines Park development with Segment 1 highlighted. (*Note*: there are other layouts that could also work.)

[c]Assumptions made are the same as for Problem 5EP-1 except class 200 pipe is used

5.82

- Determine the design peak flows for each pipeline segment
  - For a STEP system, determine Q_{DP} using Eq. 5.4 with $Q_{MIN} = 15$ gal/min and $Q_{EDU} = 0.5$ gal/min (Table 5EP.5)

$$Q_{DP} = 15 + 0.5(N_{EDU}) \tag{5.4}$$

Table 5EP.5 Design peak flows for each segment of the STEP system in the Mines Park development

Seg. no.	Buildings connected[a]	N_{EDU}	Q_{DP} (gal/min)
1	B1–B6	27	28.5
2	B1–B10	57.8	43.9
3	B11–B12	15.4	22.7
4	B16–B20	30.5	30.2
5	B21–B26	55.8	42.9
6	B1–B15	118.5	74.25
7	B1–B26	177.8	103.9

[a]Refer to Table 5EP.1 for information on each building.

5.83

- Select a trial pipe diameter for Segment 1
  - Select a trial pipeline diameter to handle $Q_{DP} = 28.5$ gal/min
  - Using Table 5.4, select a trial pipe size
    - New 2-in. Class 200 pipe appears to have capacity to handle Q_{DP} at a suitable slope (0.5–1.5 %) and velocity (<5 ft/s)
- Calculate the slope (S) of the energy grade line (EGL) needed to convey the Q_{DP} in Segment 1 using Eqs. 5.7 and 5.10

$$S = \frac{h_f}{L} = 10.5\left(\frac{Q_{CAP}}{C}\right)^{1.85}(D)^{-4.87} \tag{5.7, 5.10}$$

$$S = 10.5\left(\frac{28.5 \ \text{gal/min}}{130}\right)^{1.85}(2.149)^{-4.87}$$

$$S = 10.5(0.0603)(0.0241)$$

$$S = 0.0153 \ \text{ft./ft.} = 1.5\%$$

5.84

- Determine the velocity of flow in Segment 1
  - Calculate the pipe cross-sectional area
  - Divide the Q_{DP} (which = Q_{CAP}) by the cross-sectional area and determine V

$$A = \frac{\pi d^2}{4} = \frac{3.1414\left(\frac{2.149\ \text{in.}}{12\ \text{in./ft.}}\right)^2}{4} = 0.0252\ \text{ft}^2$$

$$V = \frac{Q_{CAP}}{A} = \frac{28.5\ \text{gal/min}}{\left(\frac{449\ \text{gal/min}}{\text{ft}^3/\text{s}}\right)}\left(\frac{1}{0.0252\ \text{ft}^2}\right) = 2.52\ \text{ft/s}$$

5.85

- Assessing the suitability of Segment 1 pipeline sizing
  - A 2-in. diameter pipe was selected for Segment 1 since:
    - It appeared to have the capacity to handle the Q_{DP} in the normal range of S (0.5–1.5 %) and V (<5 ft/s)
    - 2-in. diameter is a size often used for STEP mains
- Suitability?
  - The slope determined for Segment 1 was 1.5 %, which is okay since it is in the normal range of 0.5–1.5 %
  - The velocity determined for Segment 1 was 2.5 ft/s, which is fine since it is less than the recommended 5 ft/s

5.86

- Calculate the head loss due to friction (h_f) based on S and L using Eq. 5.7

$$h_f = S \times L \tag{5.7}$$
$$h_f = (0.015\ \text{ft/ft})(420\ \text{ft.}) = 6.3\ \text{ft.}$$

- TDH components for Segment 1

$$\text{TDH} = h_p + (h_e + h_{hv} + h_l) \tag{5.11}$$

* h_p = pressure head loss
  = 6.3 ft plus the sum of h_f for Segments 2–11
* h_e = elevation head loss
  = outlet elevation—segment elevation
  = 5960 ft—5900 ft = 60 ft.
* h_{hv} = discharge assembly losses
* h_l = lateral headloss due to friction

$\left.\begin{array}{l} h_{hv} \\ h_l \end{array}\right\}$ Typ. 5–10 ft.

5.87

Chapter 6

Treatment Using Septic Tanks

6-1. Scope

Septic tanks and similar treatment units have a long history of development and use in the United States and worldwide. They are simple, robust, and reliable unit operations that can achieve advanced primary treatment. One or more septic tanks are commonly included as a first unit operation in decentralized systems. This chapter describes the principles and processes that occur in an anaerobic septic tank and the design and implementation of septic tanks used for advanced primary treatment within a decentralized system.

6-2. Key Concepts

- Raw wastewaters often contain particulate solids (e.g., suspended solids), fats, oils and greases (FOG), and debris (e.g., rags, plastics, sticks). Solids separation and removal is normally a first unit operation in a wastewater treatment train and it can be accomplished using preliminary, primary or advanced primary treatment processes.
  - Preliminary treatment is used to physically remove FOG and debris from liquid wastewater. Grease interceptor tanks or coarse screening units are examples of unit operations that can be used to achieve preliminary treatment.
  - Primary treatment includes unit operations that are designed primarily for the removal of settleable solids. Primary treatment is often accomplished in settling basins or sedimentation tanks that have short hydraulic and solids retention times to limit anaerobic digestion since it can create solids settling problems.

R.L. Siegrist, *Decentralized Water Reclamation Engineering*,
DOI 10.1007/978-3-319-40472-1_6

- Advanced primary treatment involves removal of settleable and floatable solids along with some degree of anaerobic biodegradation. Advanced primary treatment can be accomplished with Imhoff tanks or septic tanks that have long hydraulic retention times (HRT) and very long solids retention times (SRT) to facilitate solids separation and anaerobic biological degradation processes.

■ Septic tanks provide a simple, robust, and reliable method for achieving advanced primary treatment of wastewaters generated in buildings that are commonly served by decentralized systems (e.g., houses, businesses, and institutional establishments).

■ The influent to a septic tank is typically untreated wastewater (e.g., combined wastewater or source separated graywater or blackwater). A septic tank is designed to provide quiescent conditions and attenuate the episodic flows that are generated from water using activities and events in buildings. A septic tank typically includes multiple compartments in a single tank or multiple tanks in series (e.g., with the first having 65–75 % of the total design volume). High length to width geometries (e.g., $\geq$2:1), shallow depths, and inlet and outlet baffles are used to minimize short-circuiting and solids washout from the tank.

■ Treatment in a septic tank occurs through physical, chemical and biological processes.
  - Settleable solids are removed by sedimentation and form a sludge layer at the bottom of a tank. Fats, oils, and greases are separated by flotation and form a scum layer at the surface of the liquid in the tank.
  - Chemical and biological reactions can transform the particulate and dissolved constituents during the relatively long HRTs. Design HRTs are typically 24–48 h but actual times can be longer if actual flows are less than design flows.
  - Solids retention times depend on the rate of solids accumulation and frequency of solids removal. For houses and other residential sources SRTs can be 3–10 years or more while for certain commercial and institutional sources (e.g., restaurant, highway rest area) SRTs might be as low as 1 year or less.
  - The supernatant between the sludge and scum layers flows out of the tank as septic tank effluent (STE).

■ In a typical septic tank, treatment processes can remove 30–50 % of the BOD_5 and 60–80 % of the total suspended solids (TSS). Organic nitrogen is converted to ammonium-nitrogen. However little removal of nutrients or pathogens occurs. As a result, STE still contains appreciable levels of many pollutants and pathogens.

- The sludge and scum that accumulates in a septic tank is often referred to as septage. Septage is a high strength sludge that requires periodic removal and proper management. Options for septage management typically include: (1) land treatment integrated with agriculture, (2) discharge to a local wastewater treatment plant or (3) discharge to a specially designed treatment facility. In the United States, septage is managed as a regulated waste under Federal regulations (40 CFR Part 503).
- Gases are evolved during anaerobic biodegradation processes in a septic tank.
  - Gases are primarily CH_4 (60–70 %) and CO_2 (30–40 %) but there can also be small amounts of H_2 and traces of H_2S, NH_3, H_2O and other gases.
  - Gases can pose hazards related to human entry into a tank through asphyxia caused by depletion of O_2 or acute toxicity caused by exposure to CH_4 or H_2S.
  - In most cases the gases produced are passively released to the atmosphere, but this can pose a concern since CH_4 and CO_2 are greenhouse gases. CH_4 recovery is possible with gas-tight tankage but this biogas may need conditioning before use as a fuel.
  - The potential for production of methane from a wastewater can be estimated based on the concentration of organics in the wastewater. For household wastewater alone, the net energy output from recovered CH_4 can be quite low compared to typical U.S. household demands (e.g., <5 %).
- Septic tanks are normally buried below ground outside but near the building being served. The tanks need to be watertight and must be able to handle physical loads without collapsing or cracking. Smaller tanks are often locally made of precast concrete or factory made out of manufactured plastic, often in tank sizes ranging from 500 to 2000 gal. Options for larger tanks include cast-in-place concrete tanks or manufactured tanks made of polyethylene or fiberglass. Coated steel tanks were used in the past but are not recommended any longer due to corrosion problems.
- Septic tanks should be located at a site such that there is equipment access from a roadway for installation and operation and maintenance (O&M). Bedding and backfilling of the excavation is important to support a tank. Ballast weights or other protection measures are also important at some sites to prevent flotation of an empty tank during installation or after septage removal (e.g., by high groundwater and buoyant forces).
- Appurtenances can be added to a typical septic tank to improve its operation and treatment functions. Plastic or precast concrete risers

extending from the tank top to the ground surface can be added to make tank inspection and septage removal easier. These risers should be outfitted with lids that are easily accessible but secured for safety reasons. Effluent screening units can be inserted near the outlet to prevent washout of larger solids released from the sludge and scum layers. Pumping vaults can be placed in an outlet compartment.

- Other treatment operations can be integrated with a septic tank. Aeration devices can be inserted near the outlet of a septic tank to add dissolved oxygen to the effluent and accomplish some BOD removal. An aerobic biofilter can be inserted in the flow regime so a portion of the septic tank effluent passes through it and is returned to the inlet end of the septic tank to achieve biological nitrogen removal.
- Operation and maintenance requirements are very limited for a basic septic tank. O&M needs can be greater if there are appurtenances (e.g., effluent screens or pumping units) or biogas recovery apparatus. One of the most important O&M requirements for a basic septic tank is concerned with the periodic removal and management of septage. During pumping of septage, the tank may be assessed with respect to structural soundness and watertightness, the flow conditions through the inlet and outlet baffles, and the degree of solids accumulations on the effluent screen (if used). If need be, corrective measures can be taken (e.g., clearing the inlet and outlet baffles or cleaning the effluent screen).
- The frequency of septage removal, normally accomplished by a pump-out truck, is dependent on the septic tank size, influent raw wastewater composition, and local temperatures.
  - For a given wastewater and temperature condition, a larger tank with more solids storage will require less frequent septage removal, due to a combination of more complete digestion and the larger storage volume. For a given wastewater and tank volume, a location with higher temperatures can require less frequent septage removal since anaerobic digestion occurs faster at elevated temperatures.
  - Removal from typical septic tanks serving residential dwellings is typically needed every 3–10 years. Septage removal from higher strength wastes (e.g., restaurants, rest areas, convenience stores) can be required much more frequently.
  - Solids accumulation can be measured periodically using manual methods. Automated sensors and monitoring devices are also available to measure solids accumulation levels and provide a real-time alert that septage removal is needed.

- Appliance and chemical use can have adverse effects on septic tank performance.
  - Garbage disposal use should be avoided due to the high amounts of TSS and FOG added to the wastewater, which increase the BOD and TSS concentrations and septage removal frequency.
  - Water softener use under normal conditions should not cause problems, but excessive use can increase water use and wastewater salinity. To avoid a high sodium adsorption ratio in the wastewater ($[Na^+]$ to $[Ca^{+2} + Mg^{+2}]$) discharged from a building, the regenerant solution from an ion exchange water softener (Ca^{+2} and Mg^{+2} rich) can be discharged into the building sewer.
  - Excessive disinfectant use (e.g., bleach) should be avoided as it can upset bioprocesses.
  - Pharmaceuticals and consumer products should not be disposed of via a toilet or another fixture or appliance. Constituents can include heavy metals and organic chemicals that can be of concern for discharge or reuse options but very difficult to remove or destroy in a septic tank or other downstream unit operation.
  - Use of special septic tank additives is of questionable value. Numerous products are on the market in the United States and include inorganic compounds, organic solvents, and enzyme mixtures. There is no clear evidence of any predictable positive impact on septic tank function and performance.

6-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 6 are presented in the Slides section.

6-4. Terminology

Terminology introduced and used in Chap. 6 is defined below.

Anaerobic—Refers to a biochemical state where microorganisms do not require oxygen and utilize organic matter or hydrogen as electron donors and inorganic (e.g., nitrate, sulfate) or organic matter as electron acceptors. Some anaerobic organisms may react negatively or even die if oxygen is present.

Appurtenance—Devices and equipment that are not essential to the basic function of a septic tank but can improve its function or enhance its operation in one way or another.

Biogas—Methane gas that evolves during anaerobic biological processes in treatment unit operations (e.g., a septic tank).

BOD_5—Oxygen demand exerted over 5 days due to biological degradation of organic matter plus potentially bio-oxidation of ammonia.

Chamber—A compartment within a single tank that is designed to allow supernatant movement out of it to a downstream compartment while retaining sludge and scum solids within it.

Compartmentalization—Providing the required effective liquid volume for septic tank treatment by using two or more separate chambers within one tank or multiple tanks in series through which wastewater must flow from the inlet to the outlet.

Effective liquid volume—The liquid volume provided by a treatment unit operation involving a tank, basin or compartment (e.g., septic tank, chlorine contact chamber) that yields a required hydraulic retention time. The effective liquid volume is equal to the liquid surface area multiplied by the liquid depth below the outlet from the tank, basin or compartment.

Effluent screen—A coarse screening device (e.g., ⅛ to ¼ in. openings) that is inserted in the flow path near the outlet from a tank, basin or compartment and is used to prevent larger solids from being discharged. Effluent screens are typically considered in the context of septic tanks and similar unit operations.

Flotation—The physical process by which solids that are less dense than a liquid are separated from the liquid (e.g., wastewater) by rising to the liquid surface due to buoyant forces.

Imhoff Tank—A tank that combines solids separation and anaerobic digestion to achieve advanced primary treatment of wastewater. An Imhoff Tank has a settling chamber that is physically separated from the chamber in which anaerobic digestion occurs.

Sedimentation—The physical process by which settleable solids are separated from liquid wastewater by gravity forces.

Septage—The sludge and scum that is separated and retained within a septic tank and requires periodic removal and proper management.

Septage management—Septage management encompasses the removal of septage from a septic tank (typically by pumping) followed by the proper management for its treatment and disposal or beneficial recovery. Options for septage management typically include: (1) land treatment integrated with agriculture, (2) discharge to a local wastewater treatment plant or (3) discharge to a specially designed treatment facility. In the United States, septage is managed as a regulated waste under Federal regulations (40 CFR Part 503).

Septic tank—A watertight tank with an inlet and outlet that combines solids separation and anaerobic digestion to achieve advanced primary treatment of wastewater. A septic tank has one or more compartments within which settling and flotation can occur and where the sludge and scum that is separated can undergo anaerobic digestion.

Septic tank effluent (STE)—The liquid that is discharged from a septic tank under gravity flow or by intermittent pumping.

Upflow anaerobic sludge blanket (UASB)—An advanced primary treatment unit that is designed with circuitous flow through a baffled tank so liquid contacts settled sludge. The tank can be sealed to enable collection of biogas.

Upset—A term that refers to a change in conditions that causes the function and performance of a treatment unit or system to deteriorate. Upset is often used in the context of biological treatment operations when changes to influent flow or composition adversely affect function and performance.

6-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 6 are listed below.

ASTM	American Society of Testing and Materials
BDL	Below detection limit
BOD	Biochemical oxygen demand
BOD_5	Biochemical oxygen demand exerted over 5 days
cBOD	Carbonaceous BOD
CDPHE	Colorado Department of Public Health and Environment
CFR	Code of Federal Regulations (United States)
CI	Confidence interval
C&I	Commercial and institutional
COD	Chemical oxygen demand
CSA	Canadian Standards Association
DOC	Dissolved organic carbon
DU	Dwelling unit
E	Estimated concentration
FOG	Fats, oils, greases
HRT	Hydraulic retention time
IAPMO	International Association of Plumbing and Mechanical Officials
L:W	Length to width ratio
N	Nitrogen
NPCA	National Precast Concrete Association
NR	Not reported
P	Phosphorus
PF	Peaking factor
QA/QC	Quality assurance/quality control
RL	Reporting limit

SRT	Solids retention time
ST	Septic tank
STE	Septic tank effluent
TOC	Total organic carbon
TS	Total solids
TSS	Total suspended solids
UASB	Upflow anaerobic sludge blanket reactor

C_I	Influent concentration
C_E	Effluent concentration
F	Conversion factor
f_b	Fraction of TSS and FOG separated as scum and sludge that are biodegraded
F_B	Fraction of TSS + FOG removed in the tank that remain as septage solids
F_S	Fraction of total tank volume occupied by solids
S_C	Solids concentration in septage
S_{in}	Influent concentration of TSS and FOG solids
S_M	Mass of septage generated
S_{out}	Effluent concentration of TSS and FOG solids
S_V	Volume of septage generated
Q_A	Average daily flow rate
Q_D	Design daily flow rate
Q_M	Methane produced per time
R_F	Septage removal frequency
V_{ST}	Total tank volume

6-6. Problems

6.1. Wastewaters contain a variety of materials based on the water-using activities and events in the building where the wastewater is generated. A septic tank is commonly used as a first unit operation to treat wastewater in a decentralized system. Briefly explain why.

6.2. Which of the following processes occur within a typical septic tank (check all that apply): (1) equalization of daily flow variations, (2) solids separation by sedimentation and flotation, (3) mechanical mixing of biomass with influent wastewaters, (4) aerobic biodegradation of dissolved organics, (5) anaerobic digestion of solids during long solids retention times?

6.3. How does the treatment efficiency for BOD_5 and TSS removal by a two-compartment septic tank compare to that of an anaerobic upflow sludge blanket reactor?

6.4. Why is the BOD removal efficiency lower in a septic tank compared to a primary sedimentation basin?

6.5. What gases are typically produced during septic tank treatment of wastewaters from residential sources? What gas has potential value as a source of energy?

6.6. Why is it important that septic tanks are watertight?

6.7. Steel tanks used to be used for septic tanks but they no longer are commonly used. Why not?

6.8. Septic tanks are equipped with a means of access to the contents. Why is this important and how is it facilitated during installation and setup?

6.9. Solids that accumulate in a septic tank are referred to as septage and require periodic removal. What affects the rate of accumulation and the frequency of removal?

6.10. What are the three methods that are most commonly used for treatment and ultimate disposition of the septage removed from septic tanks in the United States?

6.11. What option might be used to beneficially recover the organic matter and nutrients in septage?

6.12. A condominium complex in a small town in eastern Colorado has a maximum daily flow estimated to be 7000 gal/day based on a peaking factor of 2.5. Which of the following best describes the effluent quality that would be expected from a properly designed and operated multi-compartment septic tank with effluent screen?

1. $BOD_5 = 150$ mg/L $TSS = 60$ mg/L $NO_3 = 50$ mg/L Fecal coliforms $= 10^6$ org/100 mL
2. $BOD_5 = 150$ mg/L $TSS = 60$ mg/L $NH_4 = 50$ mg-N/L Fecal coliforms $= 10^6$ org/100 mL
3. $BOD_5 = 30$ mg/L $TSS = 30$ mg/L $NH_4 = 5$ mg-N/L Fecal coliforms $= <10^3$ org/100 mL

6.13. A septic tank unit operation is being designed for a remodeled restaurant located near Longmont, Colorado. The average daily flow is estimated to be 1500 gal/day with a peaking factor for the maximum daily flow of 2.0. With the information given below, determine the total liquid volume of the tankage and the approximate volume of each of two compartments.

Given information and assumed values: Raw wastewater $TSS = 350$ mg/L and $FOG = 150$ mg/L and there is 70 % removal in the septic tank. Design $HRT = 30$ h, $F_S = 0.5$, $F_B = 0.30$, $S_C = 0.67$ lb/gal.

6.14. A septic tank unit operation is being designed for a small apartment building near Lyons, Colorado. The average daily flow is estimated to be 2250 gal/day. What is the total volume of the septic tank needed (in gal) and the volumes you propose using for the first and second compartments in the tank? What is the most likely type of tankage to be

used, pre-cast concrete or fiberglass? How frequently will septage removal be required (in years)?
Given information and assumed values: Peaking factor for maximum daily flow = 2.5. Design HRT = 30 h. $F_S = 0.3$. Estimated septage generation rate = 500 gal/year.

6.15. A septic tank unit operation is being designed to handle the daily flow from a duplex residential unit located near Duluth, Minnesota. The owners of the duplex are interested in knowing if the potential energy they might obtain by recovery of the biogas produced in a septic tank could be used for heating the duplex instead of using natural gas. Estimate the energy content of the biogas produced and how much of the energy needed for winter heating might be obtained from the biogas.
Given information and assumed values: The average daily wastewater flow rate (Q_A) is estimated to be 976 gal/day with a chemical oxygen demand (COD) of 800 mg/L. The septic tank removal efficiency for COD is estimated at 70 %. The biogas generation rate, M = 3.2 ft^3 CH_4 per lb. COD removed based on literature data. Natural gas net heating value = 1000 BTU per ft^3 and methane net heating value = 910 BTU per ft^3. Natural gas used for heating during the 6-month winter heating season = 300 ft^3/day.

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Slide of Chapter 6

Decentralized Water Reclamation

Chapter 6: Treatment using Septic Tanks

Contents

6.1

6-1. Introduction

- Raw wastewaters normally contain particulate solids, fats, oils, grease, and debris (e.g., rags, plastics, sticks)
- Solids separation and removal is normally a first unit operation in a wastewater treatment train
  - Unit operations can be classified as shown in Fig. 6.1 and include:
    - "Preliminary treatment"—removal of debris, and fats, oils, and greases to protect downstream treatment operations, e.g.:
      - A grease interceptor or coarse screening unit
    - "Primary treatment"—removal of settleable solids, e.g.:
      - A settling basin or sedimentation tank
    - "Advanced primary treatment"—removal of settleable and floatable solids and some anaerobic biodegradation, e.g.:
      - An Imhoff tank or septic tank

6.2

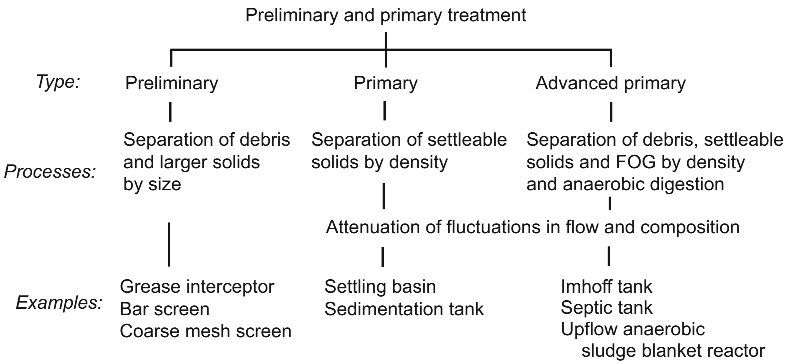


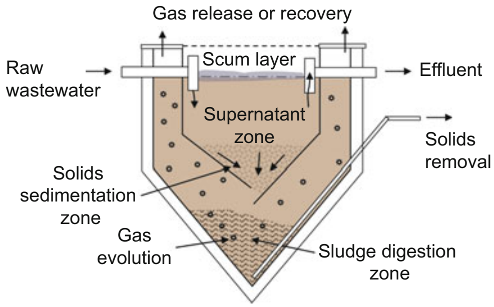
Fig. 6.1 Classification of preliminary and primary treatment unit operations

6.3

- Imhoff tanks for anaerobic primary treatment
  - Imhoff tank technology was developed in Germany and patented in 1906
    - Imhoff tanks involve combined sedimentation and anaerobic digestion as shown in Fig. 6.2
    - Imhoff tanks were used until the 1950s after which primary sedimentation units and separate digesters became common

6.4

Fig. 6.2 Schematic cross-section of a typical Imhoff tank used for advanced primary treatment of wastewater



- Septic tanks for anaerobic primary treatment
  - Septic tank technology was developed in France and a British patent was issued in 1895
    - Early versions of a septic tank involved a tank with an inlet and outlet that allowed for solids to separate from the wastewater liquid and anaerobic digestion to occur (Fig. 6.3)
    - In contrast to the decline in use of Imhoff tanks, septic tanks persisted and experienced expanded and widespread use
  - Today, septic tank operations can include modern manufactured components such as shown in Fig. 6.4
    - Other variations of septic tanks have been developed in an attempt to achieve solids separation along with enhanced anaerobic treatment plus biogas recovery (Fig. 6.5)
  - Chapter 6 is focused on modern septic tanks as used in decentralized systems

6.5

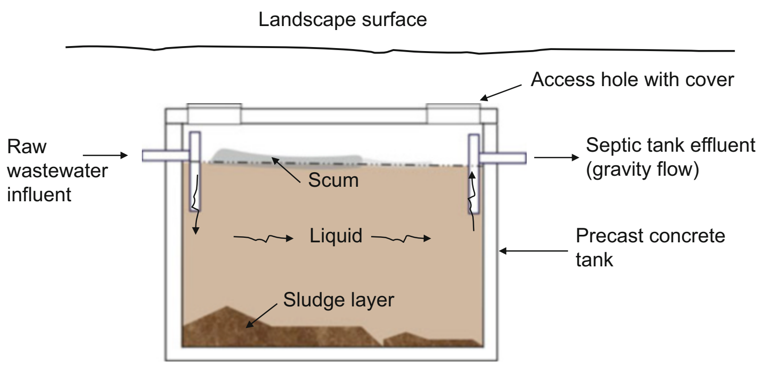


Fig. 6.3 Schematic of a single compartment septic tank as used in the United States during the 20th century

6.6

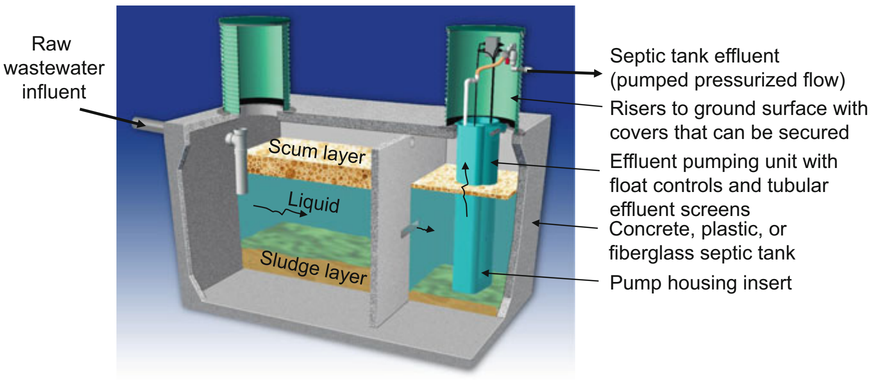


Fig. 6.4 Illustration of a modern two-compartment septic tank unit with effluent screen, pump and controls that was developed in the United States during the later 20th century (*Source*: Orenco Systems®, Inc.)

6.7

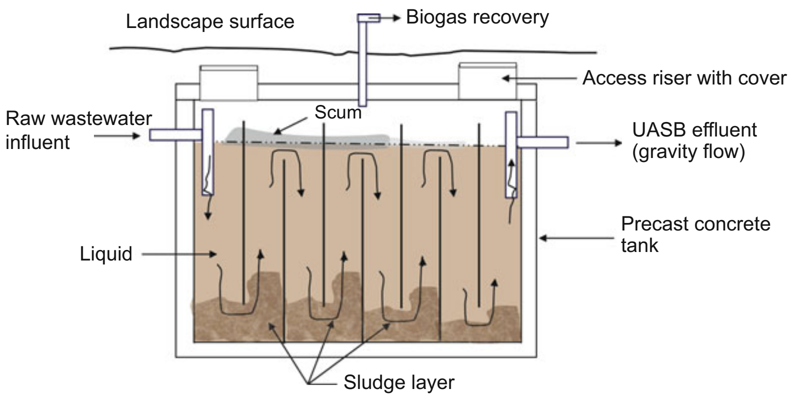


Fig. 6.5 Example of an upflow anaerobic sludge blanket (UASB) reactor. *Note*: UASBs have been most widely used for serving multiple buildings (e.g., neighborhoods of houses) in developing areas in the tropics since warmer climate conditions enhance anaerobic digestion and biogas generation

6.8

- Where are septic tanks used?
  - Septic tanks (or similar units) are almost always used in decentralized systems as a first unit operation to provide partial treatment and attenuation of raw wastewaters and enable subsequent treatment in other unit operations
  - Septic tanks are used in decentralized systems serving:
    - Individual houses, businesses, schools, churches, etc.
    - Clusters of individual sources
    - Small towns and communities
  - In some locations, particularly in developing regions of the world, a septic tank may be the only treatment unit
    - While not allowed in the United States, in some developing regions septic tank effluent is often released into a ditch, stream, lake, ocean or other surface water

6.9

6-2. Treatment Performance

- Septic tanks are normally designed to treat raw wastewaters and achieve advanced primary treatment
  - Debris, settleable solids, and fats, oils, and greases are separated from the liquid by sedimentation and flotation
  - Organic matter associated with the separated solids can be degraded by anaerobic biodegradation processes
    - BOD removal is affected by solids decomposition where acid-forming bacteria hydrolyze complex organics and convert them to volatile fatty acids (VFA)
    - VFA are dissolved and exert BOD in the septic tank effluent
    - As a result, BOD removal is somewhat lower than that of a typical primary sedimentation tank
  - There can also be a limited degree of transformation and removal of other pollutants and pathogens

6.10

- Treatment efficiency
  - Treatment efficiency as illustrated in Fig. 6.6 can be determined using Eq. 6.1
  - Septic tank treatment efficiencies that are achievable for constituents of potential concern are presented in Table 6.1

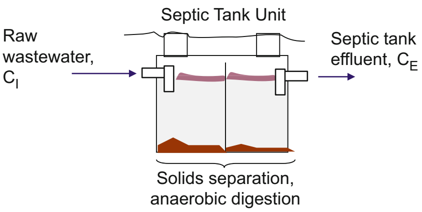


Fig. 6.6 Illustration of treatment efficiency achieved within a septic tank unit

$$R_E = \left(\frac{C_I - C_E}{C_I} \right) \times 100\% \qquad (6.1)$$

Where:

R_E = removal efficiency (%)

C_I = influent concentration (mg/L)

C_E = removal efficiency (%)

6.11

Table 6.1 Representative treatment efficiency achieved in a well-designed and operated septic tank unit

Constituent group	Removal efficiency (%)	Potential processes involved in treatment in a septic tank
Suspended solids (TSS)	60–80	Sedimentation and flotation
Fats, oils, greases (FOG)	60–80	Flotation
5-day Biochemical oxygen demand (BOD_5)	30–50	Removal by anaerobic biodegradation. Acid-forming bacteria hydrolyze complex organics and convert them to volatile fatty acids which are dissolved and exert BOD in STE
Nitrogen	Near 0	Ammonification of proteins, amino acids and other organic N compounds to NH_4^+ compounds
Phosphorus	<10	Potential for removal of P through sorption or precipitation
Pathogens	Negligible	Potential for limited die-off and inactivation but negligible impact on high concentrations normally found in STE
Trace organic chemicals	<20 %	Minor removal of some hydrophobic compounds through sorption to solids that are removed and potential for anaerobic biotransformation

6.12

- Septic tank effluent composition
  - Factors affecting treatment efficiency and effluent composition
    - Source of the wastewater influent to the septic tank, e.g.:
      - Individual house, combined residential source, source separated graywater, commercial or institutional
    - Septic tank unit design, e.g.:
      - Tank volume, compartments, baffles, effluent screens
    - Septic tank operating status, e.g.:
      - Actual flow versus design flow
      - Influent temperature and temperature within the tank
      - Solids accumulation and HRT
    - Occurrence of upset conditions, e.g.:
      - During periods of very low or high pH or elevated biotoxics such as high chlorine concentrations
  - STE composition data are presented in Tables 6.2–6.6

6.13

Table 6.2 Composition of septic tank effluent and raw wastewaters from residential dwellings in the United States (representative contemporary data)

Parameter	USEPA (2002)	Lowe et al. (2007)[a]	Conn (2008)[b]		Lowe et al. (2009)[c]	
	Range	Range	Median	Range[d]	Median	Range[d]
	Raw; STE	Raw; STE	STE	STE	Raw; STE	Raw; STE
Alkalinity (mg/L as $CaCO_3$)	NR	NR[e]; NR	383	298–503	260; 411	65–575; 172–862
TS (mg/L)	500–880; NR	NR; NR	693	623–787	1028; 623	252–3320; 290–3665
TSS (mg/L)	155–330; 50–100	18–2230; 22–276	48	35–150	232; 61	22–1690; 28–192
$cBOD_5$ (mg/L)	155–286; 140–200	30–1147; 38–861	252	133–324	420; 216	112–1101; 44–833
COD (mg/L)	500–660; NR	540–2404; 157–1931	NR	NR	849; 389	139–4584; 201–944
TOC (mg/L)	NR; 31–68	NR; NR	29.2	1.6–43.3	184; 105	35–738; 50–243
DOC (mg/L)	NR; NR	NR; NR	22.6	1.2–43.1	110; 66	29–679; 22–140

(continued)

6.14

Table 6.2 (continued)

Parameter	USEPA (2002)	Lowe et al. (2007)[a]	Conn (2008)[b]		Lowe et al. (2009)[c]	
	Range	Range	Median	Range[d]	Median	Range[d]
	Raw; STE	Raw; STE	STE	STE	Raw; STE	Raw; STE
Total N (as N) (mg-N/L)	26–75; 40–100	44–189; 26–124	106	42–107	60; 63	9–240; 27–119
TKN (as N) (mg-N/L)	NR; 19–53	43–124; 27–94	NR	NR	57; 60	16–248; 33–171
Ammonium-N (as N) (mg-N/L)	4–13; NR	9–154; 0–96	64.6	32.4–79.6	14; 53	2–94; 25–112
Nitrate N (as N) (mg-N/L)	<1; 0.01–0.16	0.05–1.1; 0–10.3	1.4	0.6–10.6	1.9 0.7	BDL[f]-9; BDL-7
Total P (as P) (mg-P/L)	6–12; 7.2–17	13–26; 3–40	12.3	8.2–25.7	10.4; 9.8	0.2–32; 0.2–33

[a]Data from a literature review.
[b]Data from averages of seven residential sites in CO.
[c]Data from 17 residential sites, all w/2 compartment septic tanks ranging from 1000 to 1500 gal and most <10-year old.
[d]Outliers were not removed and all data are included.
[e]*NR* not reported.
[f]*BDL* below detection limit.

6.15

Table 6.3 Composition of septic tank effluent from commercial and institutional (C&I) sources in the United States (after Conn 2008)

Parameter	Range of averages for the C&I sources included[a]	Restaurant	Convenience store	Retail	Elementary school	Elementary school	Church	Veterinary clinic
pH	6.5–8.2	6.9	6.6	6.5	6.9	7.1	6.7	8.2
Alkalinity (mg/L as $CaCO_3$)	343–906	906	506	343	527	419	429	393
TS (mg/L)	585–2865	2865	1110	962	1615	585	935	907
TSS (mg/L)	38.3–293	255	263	293	188	38.3	150	215
$cBOD_5$ (mg/L)	167–610	400	610	560	–[b]	167	550	255
TOC (mg/L)	31.7–231	92.9	231	76.8	64.7	31.7	105	65.1
DOC (mg/L)	31.3–171	41.0	171	71.1	43.0	31.3	93.0	64.8

(continued)

6.16

Table 6.3 (continued)

Parameter	Range of averages for the C&I sources included[a]	Restaurant	Convenience store	Retail	Elementary school	Elementary school	Church	Veterinary clinic
Total N (as N) (mg-N/L)	56–185	56.0	177	118	132	97.6	185.0	114
Ammonium-N (as N) (mg-N/L)	48–175	48.0	175	84.4	106	94.8	153	104
Nitrate N (as N) (mg-N/L)	0.3–7.2	7.2	2.0	5.3	1.5	0.3	3.1	1.9
Total P (as P) (mg-P/L)	13.4–25.0	15.0	24.2	16.9	25.0	13.4	24.5	10.9

[a]Average values shown are based on 2 to 3 samples of septic tank effluent collected at each source during 2003–2004.
[b] "–" = no data.

6.17

Table 6.4 Representative concentrations of microorganisms found in domestic wastewater and septic tank effluents in the United States (Lowe et al. 2007)

Microorganism		Most probable no. (MPN/100 mL)
Bacteria	Total coliform	10^7–10^{10}
	Fecal coliform	10^6–10^8
	Clostridium perfringens	10^3–10^5
	Enterococci	10^4–10^5
	Fecal streptococci	10^4–10^6
	Pseudomonas aeruginosa	10^3–10^4
	Shigella	10^0–10^3
	Salmonella	10^2–10^4
Virus	Enteric virus	10^3–10^4
	Coliphage	10^3–10^4
Protozoa	*Cryptosporidium parvum* oocysts	10^1–10^4
	Entamoeba histolytical cysts	10^{-1}–10^3
	Giardia lamblia cysts	10^3–10^4

6.18

Table 6.5 Concentrations of 10 trace organic compounds commonly observed in septic tank effluents from U.S. residential dwellings and commercial and institutional establishments (after Conn 2008)[a]

Organic compound	Use	Samples with detects (%)	Concentration range[b] (μg/L)
Caffeine	Stimulant	27/27 (100)	2.8–9400
Cholesterol	Animal sterol	27/27 (100)	2.1–920
Coprostanol	Animal sterol	27/27 (100)	2.4–14,000
Ethylenediaminetetraacetic acid	Metal-chelating agent	28/28 (100)	1.7–1300
Nitrilotriacetic acid	Metal-chelating agent	26/28 (93)	0.52–30.0
4-Nonylphenol	Surfactant metabolite	20/27 (74)	2.0–230
4-Nonylphenolethoxylate	Surfactant metabolite	19/27 (70)	1.4–72.8
4-Nonylphenolethoxycarboxylate	Surfactant metabolite	28/28 (100)	2.6–41.6
4-Methylphenol	Disinfectant	27/27 (100)	13.1–4700
Triclosan	Antimicrobial agent	17/27 (63)	0.85–74.6

[a]Results of analyses of 2 samples of septic tank effluents collected in Fall 2003 and Spring 2004 from each of 7 residential dwellings and 7 C&I establishments: 1 restaurant, 1 convenience store, 1 retail shop, 1 church, 1 veterinary clinic, 2 elementary schools. Analyses were made for 33 different organic chemicals and many were infrequently detected (<35 % frequency) or at very low levels (see SI section).
[b]Range in individual sample results for all 14 sources.

6.19

Table 6.6 Concentrations of major constituents in U.S. household graywater after treatment in a septic tank

Parameter	Unit	Siegrist and Boyle (1982)[a]		Brandes (1977)	Kristiansen and Skaarer (1979)[b]
		House[a]	House[b]		
pH	–	6.8–7.6	6.4–7.5	6.8	5.27–6.70
BOD_5	mg/L	216	139	149	107–160
COD	mg/L	502	409	366	307–370
TSS	mg/L	52	38	162	28–43
Kjeldahl N	mg-N/L	18.6	13.1	11.3	13–26.2
NH_4-N	mg-N/L	10.3	5.5	1.7	5.9–17.9
Total P	mg-P/L	3.2	5.5	1.4	–
Total coliform	Log #/L	7.96	8.96	8.38	–
Fecal coliform	Log #/L	7.09	7.31	7.15	5.27–6.70

[a]Graywater includes: kitchen sink, bathroom sinks, bath/shower, clothes washer, laundry sink.
[b]Range of average values at three systems. BOD is for BOD_7.

6.20

6-3. Principles and Processes

- Septic tank units involve tankage
  - One or more tanks or compartments within a tank
  - Watertight construction is used with an inlet and outlet
  - Tankage is covered, often buried, and thereby somewhat insulated in hot and cold climates
- Key processes involved in treatment in a septic tank
  - The key features of a septic tank are depicted in Fig. 6.7 and the treatment processes involved include:
    - Supernatant flow zone and attenuation of flow and composition variations
    - Physical solids separation and volume reduction
    - Chemical and biological transformations
    - Gas evolution

6.21

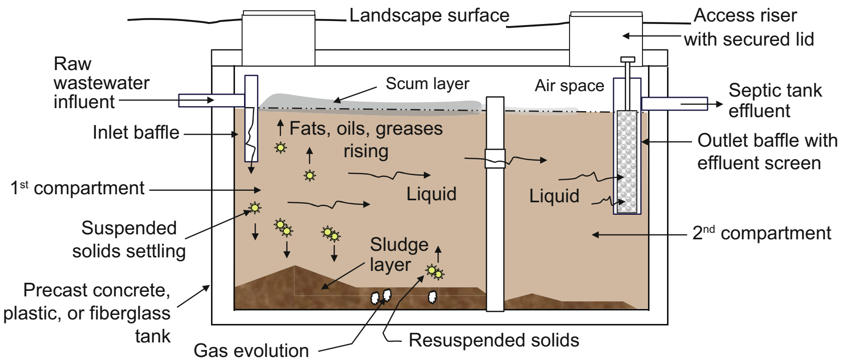


Fig. 6.7 Key features and processes involved in treatment within a basic septic tank unit. *Note*: some modern units may be outfitted with a pump and float or pressure transducer control equipment

6.22

- Septic tank supernatant flow zone
  - A supernatant zone occurs between the sludge layer and scum layer (Fig. 6.8)
  - This zone provides hydraulic retention time for solids separation and for treatment of the liquid
  - The supernatant liquid exits the tank as septic tank effluent

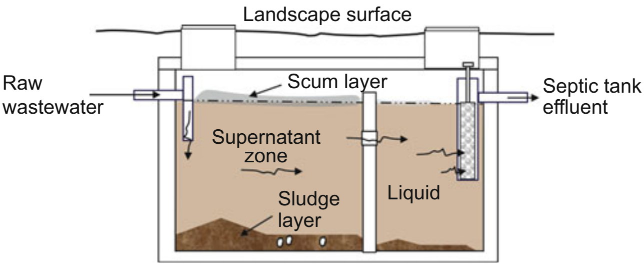


Fig. 6.8 Illustration of the supernatant flow zone in a septic tank

6.23

- Septic tank attenuation processes
  - Variations in raw wastewater flow and composition are caused by water-using and waste-generating events in a building or other source
  - Septic tanks can attenuate these variations and produce an effluent with more uniform flow and homogeneous composition (Fig. 6.9)

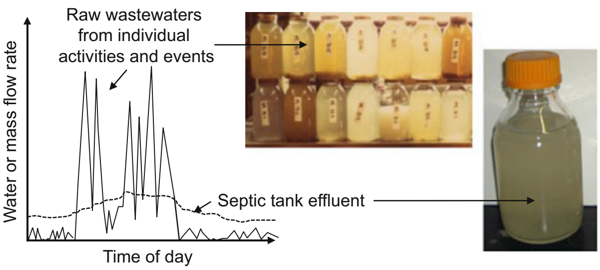


Fig. 6.9 Illustration of how a septic tank can produce an effluent with more uniform flow and homogeneous composition compared to the individual water-using activities and events

6.24

- Septic tank solids separation processes
  - Sedimentation includes Type I to IV gravity settling processes
    - Type I—Discrete particle settling according to Stokes' Law
    - Type II—Flocculent settling of colloidal particles in the supernatant
    - Type III—Hindered settling as solids approach the sludge layer
    - Type IV—Compression settling of the sludge layer in the bottom
  - Flotation involves density-based flotation processes
    - Fats, oils, and greases and other low density materials float
    - Solids that float form a scum layer at the water surface

6.25

- Septic tank chemical transformation processes
  - Chemical transformation processes can occur in the liquid phase or the solid phase
  - The primary chemical transformation process is ammonification
    - Conversion of NH_2-containing organic compounds into ammonium according to Eq. 6.2

$$R - NH_2 + H_2O + H^+ \Rightarrow R - OH + NH_4^+ \tag{6.2}$$

    - Ammonification is responsible for the fact that >85 % of the total N present in septic tank effluent is present as NH_4^+ (see Table 6.2)

6.26

- Septic tank biological transformation processes
  - Organic matter in the supernatant and in the solids that settle or float can undergo anaerobic decomposition
  - Anaerobic processes function in the absence of oxygen and thus require no aeration and associated power input
  - However, organic degradation in the absence of oxygen in less energetically favorable to the microorganisms
    - So the organic matter degradation is slower
    - Less cell mass is produced (e.g., $<20\,\%$ of aerobic systems)
  - Factors affecting anaerobic processes in a septic tank
    - Temperature: $>10\,^\circ C$; rates typically double for each $\Delta 10\,^\circ C$ above $15\,^\circ C$
    - pH: optimum is in the range of 6.5–7.5
    - NH_3: free ammonia at high levels can inhibit metabolism

6.27

- Anaerobic decomposition involves a consortium of microorganisms and three major biochemical processes
  - Hydrolysis
    - * Larger molecules are broken down into smaller organic molecules by extracellular enzymes so they can be transported into cells and metabolized
  - Acetogenesis and acid formation
    - * Fermentation of organics into organic acids, other low molecular weight compounds, H_2, and CO_2
  - Methanogenesis
    - * Fermentation of acetic acid to CH_4 and CO_2 (Eq. 6.3) and also reduction of CO_2 to CH_4 (Eq. 6.4)

$$CH_3COOH \xrightarrow{\text{Acetophilic bacteria}} CH_4 + CO_2 \tag{6.3}$$

$$4H_2 + CO_2 \xrightarrow{\text{Methanogens}} CH_4 + 2H_2O \tag{6.4}$$

6.28

- Septic tank gas evolution
  - Gases evolve during organic matter decomposition
    - Gases are primarily CH_4 (60–70 %) and CO_2 (30–40 %)
    - There can also be small amounts of H_2 and traces of H_2S, NH_3, H_2O and other gases
  - Gases can pose hazards re: entry into a tank
    - Depletion of oxygen or acute toxicity of CH_4 or H_2S
  - Gases can have potential value as a fuel
    - Methane gas that is produced can be recovered from a sealed gas-tight septic tank and used as a fuel
    - Potential for production of methane from a wastewater can be estimated

6.29

- Methane production
  - Methane potential is related to the concentration of organics
    - Measures of organic concentrations include:
      - 5-day biochemical oxygen demand (BOD_5)
      - Chemical oxygen demand (COD)
      - The COD is typically ~1.5–2 × BOD_5
  - Theoretical yield of CH_4 per lb. of COD removed
    - COD equivalent of 1 lb of $CH_4 = 4.0$ lb-O_2/lb-CH_4

$$CH_4 + 2O_2 \rightarrow CO_2 + 2H_2O \tag{6.5}$$

    - Conversion of 1 lb of COD is equivalent to 0.25 lb CH_4
      - 0.25 lb $CH_4 = 0.113$ kg $CH_4 = 7.06$ moles CH_4 at 16 g/mol
      - 7.06 moles at 22.4 L/mol-$CH_4 = 158$ L $= 5.6$ ft^3 CH_4 CH_4
  - For household wastewater alone, the net energy output can be quite low compared to household demands in the United States (e.g., <5 %)

6.30

- Retention times in a septic tank
  - Hydraulic retention time (HRT)
    - Length of time liquid remains in the tank
    - Important for attenuation of sporadic and variable influent flows
    - Important for solids separation by sedimentation and flotation
  - Solids retention time (SRT)
    - Length of time solids that separate are retained in the tank
    - Important for solids biotransformation and digestion
  - Hydraulic and solids retention times are interdependent
    - Solids accumulate in the sludge and scum layers
    - Accumulating solids occupy tank volume which reduces HRT
    - During continued operation, the SRT increases

6.31

- Illustration of the relationship between HRT and SRT during operation of a septic tank with periodic removal of accumulated solids is shown in Fig. 6.10

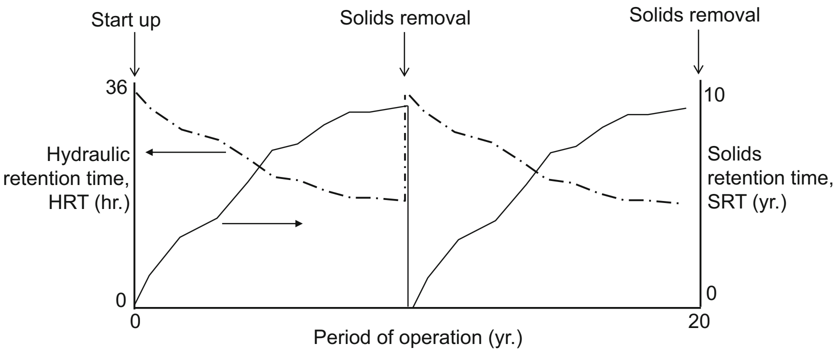


Fig. 6.10 Illustration of how HRT and SRT interact during normal operation of a septic tank

6.32

- Septage generation and removal
  - Septage is made up of suspended solids and fats, oils, and greases that are removed but not biodegraded (Fig. 6.11)

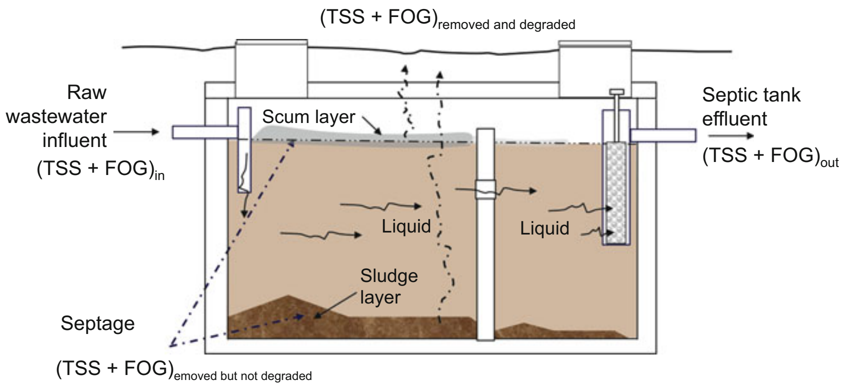


Fig. 6.11 Simplified mass balance on solids within a typical septic tank. *Note*: TSS = total suspended solids, FOG = fats, oils and greases

6.33

- Septage generation rates for residential sources is presented in Fig. 6.12

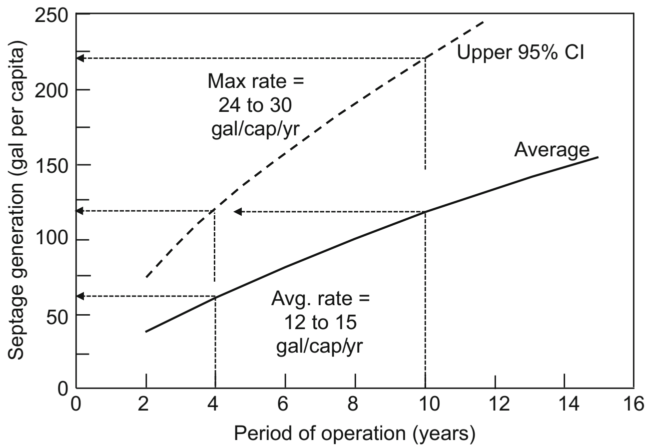


Fig. 6.12 Septage generation rates for septic tanks serving households in the United States (after Bounds 1994). *Note*: septage includes solids from both sludge and scum layers

6.34

- Septage removal
  - Removal frequency
    - Removal is often done when the sludge and scum volume occupies 30–50 % of the tank volume
    - Removal too often can adversely impact tank function by upsetting anaerobic processes and it can be costly
  - Removal frequency depends on the source and usage and climate
    - Residential sources (dwelling units (DUs))
      - For most DUs, every 5 years or more is adequate
      - For many DUs, removal could be each 10 years or more
    - Nonresidential sources (commercial, institutional, etc.)
      - Can generate more or less septage over time
      - Removal can be required every 1 year or less for some food service establishments with high TSS and FOG or facilities with primarily toilet usage
    - Climate can also impact septage generation and removal
      - Accumulation rates can be lower in warmer climates

6.35

- Septage composition
  - Septage is a high strength sludge (3–10 wt% solids)
  - The composition of septage depends on the composition of the wastewater treated in the septic tank(s)
    - Concentrations of solids, organic matter and nutrients in household septage appear in Table 6.7
    - Septage from nonresidential sources can be more concentrated than shown in Table 6.7 and have levels of heavy metals and other constituents of concern
    - Septage from households and nonresidential sources can also contain appreciable levels of trace organic compounds
- Septage management
  - Septage needs to be properly managed
    - In the United States, management is specified in Federal (40 CFR Part 503 2015) and state requirements (see Chap. 15)

6.36

Table 6.7 Concentrations of major constituents in septage removed from septic tanks serving households in the United States (USEPA 2002)

Parameter[a]	Units	Concentration	
		Average	Range
pH	–	–	1.5–12.6
BOD_5	mg/L	6480	440–78,600
Chemical oxygen demand	mg/L	31,900	1500–703,000
Suspended solids	mg/L	12,862	310–93,379
Kjeldahl nitrogen	mg-N/L	588	66–1060
Phosphorus	mg-P/L	210	20–760
Oil & grease	mg/L	5600	208–23,368

[a]Refer to Chap. 15 for additional composition data including trace organic compounds.

6.37

6-4. Design and Implementation

- Considerations for design and implementation (D&I) of a septic tank unit operation to achieve advanced primary treatment
  - Features of the source and site conditions
  - Total tank volume and compartmentalization
  - Tank geometry, inlet and outlet, tank access
  - Construction materials and watertightness
  - System installation at the site
  - Appurtenances and integral treatment operations
  - Biogas production
  - System operation and maintenance (O&M)
  - Appliance and chemical use

6.38

- D&I considerations—Features of the source and site
  - Land use and development attributes
    - Type and number of new versus existing wastewater sources and the current and planned wastewater infrastructure
    - Potential for clustering sources with shared septic tanks
    - Site land use and topography and construction space down-slope from the buildings and other sources
    - Separation distances available between the tank placement and buildings, property lines, wells, and surface waters
  - Subsurface characteristics
    - Principle characteristics that are important include depth to shallow bedrock and depth to ground water
    - Depth of freezing zone can also be important for septic tanks that have extended periods with little or no wastewater input

6.39

- D&I considerations—Tank volume
  - Effective tank volume
    - Effective tank volume equals the water surface area times the distance below the outlet invert to the tank bottom
    - Effective tank volume needs to provide:
      - A volume for flow to provide a desired HRT during operation
      - A volume for solids to accumulate (sludge and scum layers) for a desired SRT during operation
  - Design HRT and SRT
    - Typical values for HRT are in the range of 24–48 h
    - Typical values for SRT can be 3–10 years or more
    - Design HRT should account for recurring maximum daily flows
      - Also, the HRT available at startup is reduced with time as solids accumulate and occupy more volume in the tank

6.40

- The total effective tank volume can be calculated using Eq. 6.6

$$V_{ST} = HRT\left(\frac{Q_D}{1 - F_S}\right) \tag{6.6}$$

Where:
V_{ST} = total tank effective volume (water surface area x distance below outlet invert to tank bottom) (gal)
HRT = hydraulic retention time, typically set = 1–2 days
Q_D = design daily flow (gal/day) (e.g., $Q_A \times PF$)
F_S = fraction of total tank effective volume occupied by solids at the time of septage removal, typically set = 0.30–0.50 (–)

6.41

- An illustration of the effective tank volume calculated for different conditions is presented in Table 6.8
  - Note that the effective tank volume can be stipulated by regulations through sizing based on bedrooms or other factors

Table 6.8 Effective tank volume required for different sizing parameters using Eq. 6.6

Waste generating source	Q_A (gal/day)	Peaking factor	Effective tank volume for HRT = 24 h and F_S values (gal)		Effective tank volume for HRT = 48 h and F_S values (gal)	
			F_S = 0.30	F_S = 0.50	F_S = 0.30	F_S = 0.50
3-bedroom house	292	2.5	1040	1460	2080	2920
4, 8-unit apartment buildings	4878	2.0	13,940	19,510	27,880	39,020

Note: For a given septage generation rate, with F_S = 0.30 septage removal would be required more frequently than with F_S = 0.50.

6.42

- D&I considerations—Compartments
  - Dividing the total effective tank volume required into two compartments or into two tanks in series is recommended
    - Better solids separation occurs if there are two compartments
      - First compartment typically has 65–75 % of the total effective volume provided
      - In Colorado and most states, two compartments are required
    - Value of increased compartmentalization beyond two is not clear
      - Providing more than two compartments has not demonstrated improved performance in terms of STE quality
  - With multiple compartments in one tank or multiple tanks in series, the system needs to be vented to the atmosphere[a]
    - Venting is accomplished through the headspace in the tankage and associated piping and the drainage piping in the building or other source

[a]Unless gas-tight tankage is needed for biogas recovery.

6.43

- D&I considerations—Geometry
  - Elongated tanks are preferred
    - High length to width ratios (L:W) help prevent short-circuiting of flow from the inlet to outlet
    - L:W ratios should be $\geq$1.5:1 and $\geq$2:1 is preferred if possible
  - Shallower liquid depth and greater surface area are better
    - Settling efficiency is related to overflow rate or surface loading rate, which are a function of surface area
    - However, some liquid depth is required
      - Provides for sludge and scum accumulation
      - Ensures flow into and through the tank does not disrupt the sludge and scum layers
    - Minimum liquid depths?
      - About 36 in. below the outlet invert is recommended

6.44

- D&I considerations—Inlets and outlets
  - General features
    - A drop of 2–3 in. between the inlet and outlet is needed for gravity flow through the tank
    - A clear space of $\geq$9 in. above the invert of the outlet provides for scum accumulation and venting of the tankage
  - Baffling
    - Inlets and outlets are baffled to improve the flow regime
      - Plastic sanitary tees are most common
      - Curtain baffles have been used but are not recommended since scum can accumulate and block the inlet pipe
    - Inlet and outlet baffles using tees
      - Rising leg should be open, $\geq$6 in. above the liquid level
      - Dropping leg should extend into the supernatant flow zone but not more than 30–40 % of the liquid depth

6.45

- D&I considerations—Tank access
  - Access to the inside of a septic tank unit is needed for several reasons
    - To check sludge and scum levels and enable pumping septage
    - To enable inspection of the inlet and outlet baffles
    - For servicing appurtenances like effluent screens
  - Manways and ports are used over inlets and outlets
    - Manways are 18–24 in. diameter or square
    - Inspection ports are $\geq$8 in. in diameter
    - Manways and ports should rise to the ground surface (or near it) and be fitted with airtight, secured lids or caps
  - For compartmentalized tanks
    - Each compartment requires access

6.46

- D&I considerations—Tank construction
  - Pre-manufactured versus constructed in place
    - Tanks can be commercially purchased or built in place
    - Choice depends on tank volume and construction materials
  - Materials of construction
    - Concrete—most common for small tanks (e.g., $<$2000 gal)
    - Polyolefin (polyethylene or polypropylene)—light weight for larger tank shipping and handling
    - Fiberglass—light weight for larger tank shipping and handling
    - Coated steel—no longer widely used due to corrosion
  - Quality construction is critical to tank performance
    - Tank must handle soil loads without cracking or collapsing
    - Flexible joints for piping connections allow for settling
    - Watertight joints are needed to prevent leakage in or out

6.47

- D&I considerations—Watertightness
  - Watertightness is critical to tank performance and downstream unit operations
    - Inflow and infiltration can increase the flows for treatment
    - Exfiltration of wastewater in the tank can impact ground water
    - Leakage can cause structural failure by collapsing a tank
  - Tank joints and covers should be water tight
    - Tongue and groove joints and seals can be used
  - Testing for watertightness can be done as part of the QA/QC process during tank manufacturing and also at a job site
    - Hydrostatic or vacuum tests can be used
    - Procedures and criteria are published by ASTM, NPCA, IAPMO, and CSA

6.48

- D&I considerations—Installation at a site
  - Location at the site should have easy access from a roadway or driveway for tank installation and operation and maintenance
    - Within about 50 ft. provides ready access to pumper trucks
  - Bedding and backfilling is important to support a tank and the inlet and outlet piping
  - Ballast weights, an extended bottom slab, or other protection measures may be needed to prevent flotation
  - All materials must be resistant to corrosion
    - The environment within a septic tank is extremely corrosive to bare and poorly treated metal
    - All materials including tankage, piping, electrical conduits and panels must be corrosion resistant
  - Figures 6.13 and 6.14 illustrate a small and large installation

6.49

Fig. 6.13 Installation of two 2000 gal pre-cast concrete tanks in series to serve an apartment building: (**a**) is a view west during installation and (**b**) is a view east after installation. *Note*: the risers and secured lids at the ground surface following backfilling

6.50

Fig. 6.14 Photographs of a large installation of 20,000 gal fiberglass septic tanks to serve a national park visitor's center. *Note*: (**a**) the flotation ballast, (**b**) pea gravel bedding, (**c**) insulation and risers, and (**d**) backfill and secured riser lids. (Photographs courtesy of SCG Enterprises, Inc.)

6.51

- D&I considerations—Appurtenances
  - Effluent screening units (Fig. 6.15)
    - General features
      - Passive gravity flow through a coarse screen (e.g., 1/8- to 1/4-in. openings)
      - Effluent screening units can be temporarily removed and cleaned with water and the solids washed off are returned into the septic tank
      - A variety of screening devices are commercially available
    - Performance of effluent screening units
      - Highly variable depending on features of the screening unit and application
      - Screening devices can retain larger TSS and reduce carry-over of TSS and BOD in STE, particularly under overloading conditions
      - But, under normal operation they do not substantially lower average concentrations in typical STE

6.52

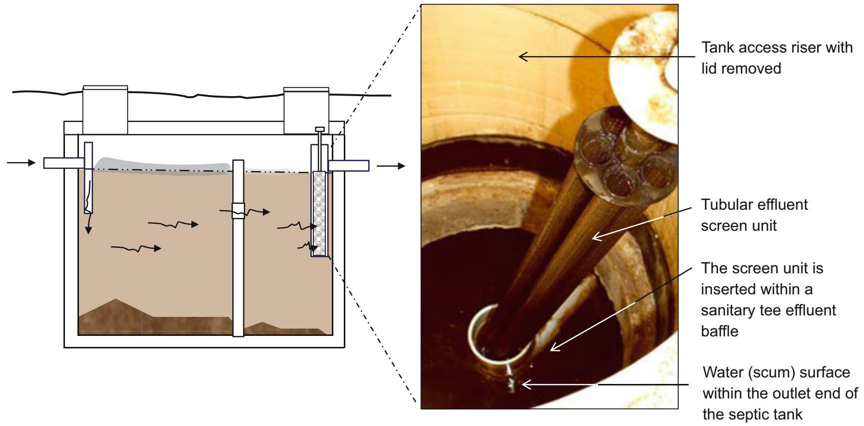
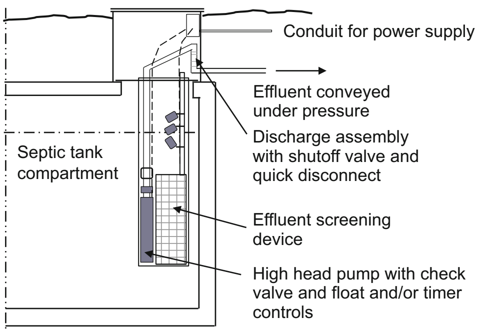


Fig. 6.15 Illustration of a septic tank effluent screening unit being lifted out of the effluent baffle tee to permit inspection and cleaning if needed

6.53

- Wastewater pumping units and controls (Fig. 6.16)
  - Pumping units can be in the tank or in a separate basin (often called a pump tank)
  - Pumps can be used to intermittently dose downstream unit operations
  - Modern pumps can have a long service life (e.g., 20 years)

Fig. 6.16 Illustration of a pumping unit in a cylindrical vault placed within the second compartment of a septic tank



6.54

- Sensors and monitoring devices
  - Pump controls
    - Pump on-off cycle data or elapsed time data can be used to produce flow rate information
    - This information can be compared to the design flow to verify the system is treating a flow at or below the design flow
    - This information also can be compared to indoor water meter readings to ascertain if there is infiltration into a tank or leakage out it
  - Level sensors
    - Level sensors can be used to determine if there is effluent leakage or clear water infiltration/inflow
    - Sensor data can also be used to determine solids accumulation and need for septage removal

6.55

- D&I considerations—Integral treatment operations
  - In-tank aeration devices (Fig. 6.17)
    - Placed in an insert or in a second compartment of a septic tank
    - Purpose is to add DO and provide some aerobic degradation of septic tank effluent prior to discharge to a next treatment unit operation
  - Integrated biofilter with recycle (Fig. 6.18)
    - Tank contents are treated in an aerobic biofilter and then a portion of the filtrate is recycled back into the septic tank near the inlet to the first compartment
    - Biological N removal can be achieved
      - NH_4^+ converted to NO_3^- in the aerobic biofilter
      - NO_3^- biological reduced to N_2 gas in the anaerobic septic tank

6.56

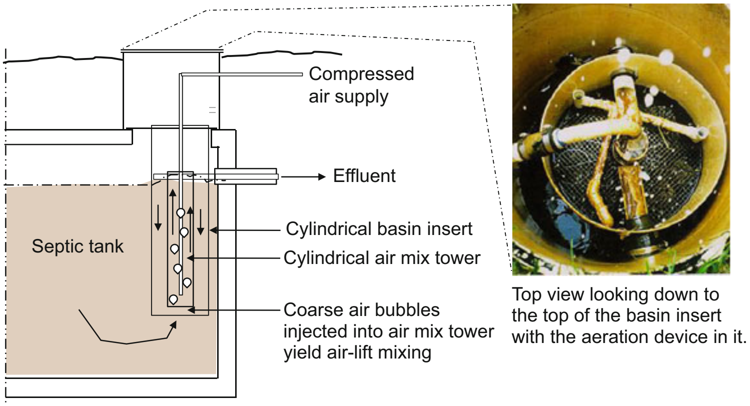


Fig. 6.17 Illustration of an aeration insert that consists of an air-lift pump that provides aeration in a septic tank and semi-continuously discharges small volumes of effluent

6.57

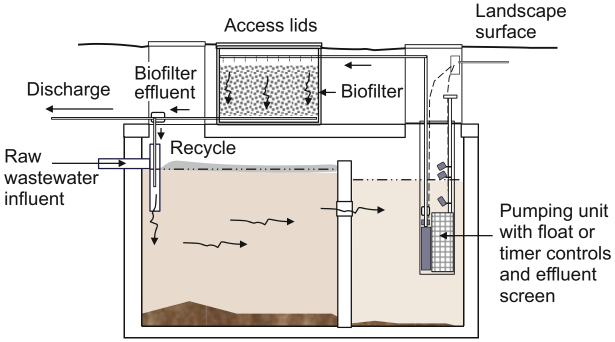


Fig. 6.18 Illustration of a septic tank with an integrated biofilter where there is partial recycle of the filter effluent back to the septic tank for biological N removal

6.58

- D&I considerations—Biogas production and recovery
  - The biogas production rate can be estimated using Eq. 6.7

$$Q_M = Q_A(C_I - C_E)M(F) \tag{6.7}$$

Where:
Q_M = methane produced per time (ft^3/day)
Q_A = average daily flow (gal/day)
C_I = influent concentration of total COD (mg/L)
C_E = effluent concentration of total COD (mg/L)
M = unit methane production (ft^3 CH_4 per lb. COD removed)
M = is typically less than the theoretical value (e.g., M = 3.2)
F = 8.34×10^{-6} = conversion factor for mg/L to lb/gal

  - Disposition of biogas
    - In most cases CH_4 is passively released to the atmosphere, but this can pose a concern as a green house gas emission
    - CH_4 recovery is possible with gas-tight tankage but biogas may need conditioning before use as a fuel

6.59

- D&I considerations—Operation and maintenance
  - A basic septic tank is a passive treatment unit
    - O&M requirements are very limited for a basic septic tank
    - O&M needs can be greater with appurtenances or integrated treatment operations (e.g., effluent screens, pumping units)
  - Typical O&M for a basic septic tank
    - Periodic inspection and removal of sludge and scum accumulation
      - Pump out and properly manage septage
    - During pumping the following observations can be made:
      - Observe and clean effluent screens
      - Observe inlet and outlet baffles
      - Assess structural soundness and watertightness

6.60

- Septage generation can be estimated based on a simplified mass balance
  - Fraction of solids removed that are not biodegraded is given by Eq. 6.8

$$F_B = \left(\frac{S_{in} - S_{out}}{S_{in}}\right)(1 - f_b) \tag{6.8}$$

$$F_B = (\text{Fractional removal})(1 - f_b)$$

$$F_B = (0.75)(1 - 0.80) = 0.15$$

Where:

S_{in} = influent concentration of TSS and FOG solids (mg/L)

S_{out} = effluent concentration of TSS and FOG solids in septic tank effluent (mg/L) (with % Removal = 75 % of S_I)

f_b = fraction of TSS and FOG separated as scum and sludge that are biodegraded during retention in the tank (during SRT) (–) (assume 80 %)

F_B = fraction of TSS + FOG removed in the tank that remain as septage solids (–); F_B = 0.15 for %Removal = 75 % and f_b = 80 %

6.61

- Rate of septage generation is given by Eqs. 6.9 and 6.10

$$S_M = (TSS + FOG)(Q_A)(F_B)F \tag{6.9}$$

$$S_V = \frac{S_M}{S_c} \tag{6.10}$$

Where:

S_M = mass of septage generated (lb/year)

TSS = influent total suspended solids concentration (mg/L)

FOG = influent fats, oils, and greases concentration (mg/L)

Q_A = average daily flow being treated (gal/day)

F_B = fraction of TSS + FOG that remain as septage solids (e.g., 0.15)

S_V = volume of septage generated (gal/year)

S_C = solids concentration in septage (lb/gal) (e.g., 0.25–0.83 lb/gal)

F = 0.003 = conversion factor for mg/L to lb/gal and days to years

6.62

- The frequency of removal for septage can be estimated using Eq. 6.11

$$R_F = \frac{F_S(V_{ST})}{S_V} \tag{6.11}$$

Where:
R_F = septage removal frequency (year)
V_{ST} = total septic tank effective volume (gal)
F_s = fraction of tank volume to be occupied by septage at time of septage removal, typically set = 0.30–0.50 (–)
S_V = volume of septage generation (gal/year)
calculated by Eq. 6.10 or literature data (e.g., Fig. 6.12)

6.63

- How frequently is septage removal required?
  * Removal frequency depends on the septage generation rate and the septic tank effective volume provided
  * Table 6.9 illustrates how wide the variability can be for a DU depending on the values of different parameters

Table 6.9 Septage removal frequency as affected by different sizing parameters and septage generation rates

Parameter	Units	Removal frequency (R_F) based on generation rate, septic tank volume, and F_S			
		Average		Upper limit 95 % CI	
		$F_S = 0.3$	$F_S = 0.5$	$F_S = 0.3$	$F_S = 0.5$
Septage generation rates	gal/cap/year	14		28	
Removal frequency for different tank effective volumes:					
$V_{ST} = 1000$ gal	Year	5.3	8.9	2.7	4.5
$V_{ST} = 1250$ gal	Year	6.7	11.2	3.3	5.6
$V_{ST} = 1500$ gal	Year	8.0	13.4	4.0	6.7

Note: Calculations were made using Eq. 6.11 assuming an average DU with a conservative occupancy of 4 persons and septage removal at $F_S = 0.3$ or 0.5.

6.64

- Septage treatment and disposal/reuse
  - Management programs vary by states and municipalities
    - Tracking or manifest systems help prevent illegal dumping
    - Procedures help ensure proper treatment and disposal
  - Fate of septage that is pumped from septic tanks (see Chap. 15)
    - Land application
      - Most common
      - Requires careful site selection and application methods
    - Treatment at a municipal wastewater treatment plant
      - Also common
      - Different approaches can be used to feed septage into a plant to avoid upsets
    - Treatment in a special septage treatment plant
      - Employed when other options are not available or where biosolids recovery is desired for a larger scale application

6.65

- D&I considerations—Appliance and chemical use
  - Garbage disposal use
    - Garbage disposals grind up solid food waste and discharge it into the kitchen sink wastewater stream
      - Garbage disposal use should be avoided due to the TSS and FOG added to the wastewater that increases BOD and TSS concentrations and septage removal frequency
      - Biodegradable food waste can be composted
  - Water softener use
    - Water softeners typically use ion exchange resins to remove hardness ions (Ca^{+2} and Mg^{+2}) from the water supply and resin regeneration is done periodically with a Na^{+} solution
    - Normal operation should not cause problems, but excessive use can increase water use and wastewater flow and salinity

6.66

- To avoid a high sodium adsorption ratio in the wastewater (ratio of $[Na^+]$ to $[Ca^{+2} + Mg^{+2}]$), the regenerant solution (rich in Ca^{+2} and Mg^{+2}) can be discharged into the building sewer

- Chemical use
  - Excessive disinfectant use (e.g., bleach) should be avoided as it can upset bioprocesses
  - Pharmaceuticals and consumer products should not be disposed of via a toilet or other fixture or appliance
    - Constituents can include heavy metals and organic chemicals that can inhibit or upset treatment processes
    - Constituents can be of concern for discharge or reuse options but very difficult to remove or destroy
  - Use of special septic tank additives is of questionable value
    - Numerous products are on the market in the United States
    - Inorganic compounds, organic solvents, enzyme mixtures
    - No clear evidence of any predictable positive impact

6.67

6-5. Summary

- Septic tanks are commonly designed as a first unit operation in a decentralized wastewater system
- Wastewater treatment occurs primarily by physical solids separation and anaerobic digestion
  - Advanced primary treatment can be achieved passively along with attenuation of event-based wastewater generation
  - Typical HRTs are 1–2 days and SRTs are 3–10 years
  - Appurtenances and integral components can be employed to improve operation and treatment efficiency
- Removal and proper handling of septage is needed
  - Removal can be every 3–10 years +/− for houses but much more often for higher strength sources

6.68

6-6. Example Problems

- 6EP-1. Septic tank design for an apartment building
  - Given information
    - 8-unit apartment building in Golden, Colorado with normal water-using fixtures and appliances
    - Occupancy is 2.5 persons per dwelling unit
    - Peaking factor for recurring maximum daily flow rate = 2.0
    - Design HRT for the septic tank(s) = 36 h
  - Determine
    - The average and maximum daily flow rates
    - The total effective volume and size of each compartment for a 2-compartment septic tank and a feasible tank option
    - The likely concentrations of BOD_5, TSS, total N and total P in the septic tank effluent

6.69

- Solution
  - Calculate the average daily flow (Q_A) and recurring maximum daily flow (Q_P)

$$Q_A = 69.2 + 37.2N_P = 69.2 + 37.2(2.5) = 162\,\text{gal/day} \tag{3.2}$$

$$Q_A = \left(\frac{Q_A}{DU}\right)(N_{DU}) = (162\,\text{gal/d})(8\,\text{DU}) = 1298\,\text{gal/day} \tag{3.5}$$

$$Q_P = PF(Q_A) = 2(1298\,\text{gal/d}) = 2595\,\text{gal/day} \tag{3.8}$$

6.70

- Calculations of total tank volume required
  - Given that HRT = 36 h and select $F_S = 0.30$

$$V_{ST} = HRT\left(\frac{Q_D}{1 - F_S}\right) \tag{6.6}$$

$$V_{ST} = 1.5\ \text{days}\left(\frac{2595\ \text{gal/day}}{1 - 0.30}\right) = 5560\ \text{gal}$$

6.71

- Determine compartment sizing
  - Assume 2/3 of V_{ST} in a first compartment and 1/3 in a second
    - Compartment 1 = 0.667 × 5560 gal = 3708 gal
    - Compartment 2 = 0.333 × 5560 gal = 1852 gal
  - Based on volumes required, use a pre-manufactured tank
    - An example tank would be a Xerxes 6000-gal fiberglass dual compartment tank (Fig. 6EP.1)

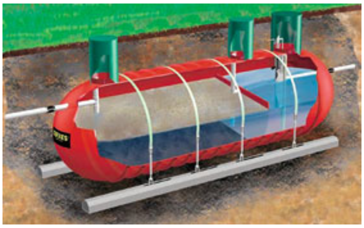


Fig. 6EP.1 Illustration of a Xerxes 6000-gal fiberglass dual compartment tank. *Source*: http://www.xerxes.com/assets/documents/library/Onsite-wastewater-brochure.pdf

6.72

- Estimate the concentrations of BOD_5, TSS, total N and total P in the septic tank effluent
  - Since the source is from residential dwelling units the average concentrations can be approximated by those reported in the literature (Table 6EP.1)

Table 6EP.1 Representative concentrations of major constituents in STE from residential sources[a]

Parameter / Source	BOD_5 (mg/L)	TSS (mg/L)	Total N (mg-N/L)	Total P (mg-P/L)
Single homes	156 (38–861)	58 (22–276)	55 (26–124)	10 (3–39)
Clusters of ≤8 dwelling units	184 (63–229)	62 (27–99)	46 (30–75)	6.9 (5–10)

[a] *Source*: Literature review and data analysis completed by Lowe et al. 2007. Median values and ranges are given.

6.73

- 6EP-2. Estimated frequency of septage removal
  - Given information
    - Condominium building in Pueblo, Colorado with 6 dwelling units
    - Estimated occupancy = 2.5 persons per DU
    - Estimated $Q_A = 973$ gal/day with PF = 2.5
    - Anticipated influent concentrations: TSS = 200 mg/L and FOG = 15 mg/L
    - Select HRT = 30 h, PF = 2.5, $F_S = 0.3$, $F_B = 0.15$, $S_C = 0.58$ lb/gal
  - Determine
    - Total septic tank effective volume required (gal)
    - Septage generation rate (gal/year and gal/year per capita)
    - Frequency of septage removal required (year)

6.74

- Solution
    - Calculate the total effective volume of the septic tank

$$V_{ST} = HRT\left(\frac{Q_D}{1 - F_S}\right) \tag{6.6}$$

$$V_{ST} = 1.25\left(\frac{973 \times 2.5}{1 - 0.3}\right) = 4344 \ \text{gal}$$

6.75

- Calculate the septage generation rate

$$S_M = (TSS + FOG)(Q_A)(F_B)(F) \tag{6.9}$$

$$S_M = \left(200 + 15 \ \frac{mg}{L}\right)\left(973 \frac{gal}{day}\right)(0.15)(0.003) = 94.1 \ \frac{lb}{year}$$

$$S_V = \frac{S_M}{S_c} = \frac{94.1 \ lb/year}{0.58 \ lb/gal} = 162 \ \frac{gal}{year} \tag{6.10}$$

6.76

- Determine the septage generation rate per person
  - * Calculated generation rate for the condominium
    - – Calculated rate = 162 gal/year
    - – Occupancy = 15 persons (6 DU × 2.5 persons per DU)
    - – Generation rate per person = 10.9 gal/year per capita
  - * Literature data on septage generation rate per person
    - – For example, see septage generation graph presented in Fig. 6.12
    - – The average rate = 12–15 gal/year per capita
  - * The two estimates of generation rates differ—why?
    - – Different conditions could be occurring at the condominium building in Pueblo versus the literature data, e.g.:

      Influent TSS and FOG could be lower or temperatures could be higher at the Pueblo site

6.77

- Calculate the septage removal frequency

$$R_F = \frac{F_S(V_{ST})}{S_V} \tag{6.11}$$

$$R_F = \frac{0.3(4344)}{162}$$

$$R_F = 8.0 \text{ year.}$$

6.78

- 6EP-3. Estimate the potential biogas generation
  - Given information
    - Home in Brighton, Colorado with 4 occupants
    - $Q_A = 218$ gal/day
    - Untreated wastewater COD = 1000 mg/L
    - Septic tank effluent COD = 300 mg/L
    - Select M = 3.2 ft^3 CH_4 per lb. COD removed based on literature data
  - Determine
    - Estimated methane generation rate (ft^3 CH_4 per day)
    - Energy content of the methane produced (BTU/day)
    - How the biogas fuel compares to household natural gas use

6.79

- Solution
  - Estimate of the volume of biogas generated

$$Q_M = Q_A(C_I - C_E)M(F) \tag{6.7}$$

$$Q_M = \left(218\ \frac{\text{gal}}{\text{day}}\right)\left(1000 - 300\ \frac{\text{mg}}{\text{L}}\right)\left(3.2\frac{\text{ft}^3\text{CH}_4}{\text{lb COD}}\right)\left(8.34 \times 10^{-6}\right)$$

$$Q_M = 4.1\,\text{ft}^3\ \ \text{CH}_4\ \text{per day}$$

6.80

- Estimate of the energy content of the biogas generated
  - Methane net heating value = 910 BTU per ft^3

$$Q_{MM} = 4.1\,ft^3 \ \ CH_4 \ \ per\,day$$
$$yields \ \ 3730 \ \ BTU \ \ per\,day$$

- Comparison of biogas methane generation to natural gas use
  - Average natural gas use in a U.S. home during 2009
    - Average use = 193 ft^3 per day
    - Which equals 198,211 BTU per day
  - Biogas generated from a household septic tank based on the calculations made
    - Amounts to 4.1 ft^3 per day
    - Or about 2 % of the average natural gas usage within a household in the United States

6.81

Chapter 7

Treatment Using Aerobic Bioreactors

7-1. Scope

Aerobic biological processes have a long history of use for treatment of wastewaters in cities and urban areas of the United States and abroad. Application of aerobic treatment in decentralized systems evolved in an attempt to produce a higher quality effluent than that typically produced by a septic tank or other anaerobic unit operation. Aerobic treatment of wastewater is capable of producing an advanced secondary effluent that can enable surface discharge and reuse options. Chapter 7 describes the principles and processes involved in aerobic biological treatment and the design and implementation of aerobic treatment units using bioreactors to achieve organic matter removal and nitrification. Biological treatment for enhanced nutrient removal is covered in Chap. 13.

7-2. Key Concepts

- Aerobic biological treatment exploits microorganisms that can use constituents in wastewater as a source of carbon and nutrients and also energy for growth. During growth the soluble and colloidal organic matter present is converted to CO_2 or cell mass and the biomass solids produced can be separated from the liquid by a clarification or filtration process. During clarification nonbiodegradable suspended solids can become entrained in the flocs that form and also be removed in the clarification process.
- Aerobic treatment methods for wastewater are generally based on a 100^{+}-yr old process known as “activated sludge”. In this process

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"activated microbial biomass" involves a consortium of microorganisms that are contacted with wastewater under aerobic conditions to support microbial growth for a certain period of time.

- To accomplish aerobic treatment using an activated sludge process there must be a flow regime where the wastewater to be treated is contacted with microorganisms (referred to as biomass) during flow through an aeration zone (i.e., a tank, basin, channel, etc.). The biomass becomes acclimated and activated and the biomass solids retention time (SRT) in the system is much longer than the hydraulic retention time (HRT).
- The activated biomass can be suspended in wastewater that is being aerated or it can be attached to a surface that is aerated. The contents of an aeration zone that contains activated biomass and wastewater that is being aerated are referred to as "mixed liquor". The concentration of mixed liquor volatile suspended solids (MLVSS) is often used as a measure of the concentration of the activated biomass.
- Aerobic treatment can be used to achieve different rates of removal of different constituents including organic matter and nutrients. Aerobic treatment methods can be classified to include those that are designed to achieve:
  - High rate organic matter removal.
  - Intermediate rate organic matter removal with no nitrification of ammonia.
  - Low rate organic matter removal plus nitrification of ammonia.
  - Low rate removal of total nitrogen and phosphorus.

■ Aerobic treatment methods most commonly applied in decentralized systems are used to treat primary effluent (e.g., primary settling tank or septic tank effluent) and achieve advanced secondary treatment with low rate removal of organic matter and nitrification of ammonia. As used in decentralized systems, aerobic treatment methods are typically implemented in two distinct ways:

- Suspended-growth, extended aeration units including flow-through tanks, basins, or channel reactors or sequencing batch reactors.
- Attached-growth flow-through units including submerged media reactors or rotating biological contactors or non-submerged trickling filters.
- In either of these approaches,
  - Aerobic treatment can be accomplished using a single stage or multiple stages (e.g., a stage is a single unit of aeration w/ clarification).
  - Biological treatment systems can also be laid out with sequential zones of aeration and no aeration to create zones of anoxic or anaerobic conditions.

- In the context of Chap. 7, aerobic treatment methods are implemented in aerobic treatment units (ATUs) that involve a bioreactor (suspended or attached growth, flow-through or batch flow). Design and implementation of can be accomplished in two ways.
  - For larger design flows and certain types of ATUs (e.g., extended aeration bioreactors, aerated lagoons, oxidation ditches, trickling filters) site-specific engineering, construction, and startup practices are often used.
  - For smaller design flows and common decentralized applications, general engineering is used to support application of a packaged aerobic treatment unit. Packaged ATUs typically include extended aeration and solids separation processes that are commercially available for design flows up to 2500 gal/day or more.

- Aerobic treatment of wastewater can involve complicated processes and components that require careful consideration during design and implementation. Design and implementation considerations include: the source of wastewater to be treated; the bioreactor configurations and specific design details (bioreactor volume, biomass solids separation and retention, solids production and wasting, oxygen requirements and supply method, energy needs); system installation and startup at a site; system operation and maintenance requirements; and appurtenances and integral treatment operations.
  - Example design parameters associated with suspended growth systems such as implemented in an extended aeration ATU include: (1) the solids retention time (SRT) and net solids production (Y_N), (2) the target MLVSS level in the aeration zone, (3) the food to microorganism ratio (F:M ratio = lb of BOD or COD per day applied to the lb of MLVSS in the aeration zone), (4) the method of activated biomass retention and recycle ratio (if applicable), (5) the method of aeration and oxygen requirements, and (6) energy requirements and supply.
  - Example design parameters associated with attached growth systems such as implemented in a trickling filter ATU include: (1) the type and depth of packing medium used in the filter, (2) the hydraulic loading rate to the surface area of the filter, (3) the organic loading rate to the volume of the filter, (4) the recirculation ratio used, (5) the net solids production and method of removal and management, and (6) energy requirements and supply.

- The influent to an aerobic treatment unit is commonly the effluent from an upstream primary settling tank or septic tank. In some designs, the influent can be raw untreated wastewater, but in that case there is normally a first compartment of the ATU that can function like a primary settling tank or septic tank. In either case, flow equalization is achieved and this is very important to proper function and performance of an ATU.

Day-to-day continuity of influent flow and reasonable consistency of composition is also important to biological process functions and this can be difficult to ensure in individual homes and small businesses. Hence, ATUs tend to function and perform better when used for clusters of houses or businesses and multi-source developments.

- There are different approaches to achieving aerobic conditions in an ATU, but they all attempt to achieve mixing or flow conditions that enable contact of wastewater with activated biomass where there is sufficient dissolved oxygen (DO >2 mg/L) and conditions conducive to aerobic biological treatment.
  - Suspended growth ATUs can achieve mixing and aerobic conditions by using mechanical aerators or compressed air diffusers.
  - Attached growth ATUs can achieve contact and aerobic conditions by using (1) plastic media that is continuously or intermittently submerged within a compartment, tank, or basin that is aerated (e.g., by compressed air diffusers) or (2) rock or plastic media that is passively aerated during intermittent and unsaturated flow through a bed of the media.
- For aerobic biological treatment, activated biomass must be retained in the ATU for a period of time.
  - In suspended growth ATUs, biomass is suspended in the aeration zone and then separated during clarification in a settling compartment or tank. The separated biomass is returned to the aeration tank or zone.
  - In attached growth ATUs, biofilms attach and grow on stones, plastic honeycombs or other surfaces that the wastewater flows over. The attached biomass is thus retained in system for a long period until it sloughs off. The biomass that sloughs off the media is subsequently separated from the wastewater by clarification in a settling compartment or tank.
- Treatment in an ATU occurs through biological processes supported by physical and chemical processes. Sizing of the bioreactor is typically based on time and loading parameters.
  - In a suspended growth ATU, system sizing is determined by extended aeration where HRTs are typically 8–36 h and SRTs can be much longer (e.g., 30–180 days).
  - In an attached growth ATU, sizing is based on hydraulic loading rates (HLRs) to the filter surface area (e.g., 50–500 gal/day per ft^2 of surface area) and organic loading rates (OLRs) to the filter volume (e.g., 10–100 lb-BOD/day per ft^2 for BOD removal and 5–15 lb-BOD/day per ft^2 for nitrification and NH_4^+ removal).
- An ATU can produce a high quality, secondary effluent if it is correctly designed and properly operated and maintained. Effluent concentrations of <20 mg/L BOD_5 and TSS are achievable along with near 100 %

conversion of ammonium nitrogen to nitrate nitrogen and 15–25 % removal of total nitrogen. Some removal of total phosphorus (e.g., 10–20 %) and pathogenic microorganisms (90–99 %) can also occur. Aerobic treatment can also be designed for enhanced removal of nitrogen and phosphorus as described in Chap. 13.

- Solids are produced during aerobic treatment of wastewater and some solids accumulate over time. The solids generated depend on the influent wastewater being treated and the conditions of growth (e.g., SRT and the temperature).
  - In a suspended growth ATU, a poorer quality wastewater being treated with a shorter SRT under colder temperatures generates a greater mass of solids. For example, for a SRT of 30 days, the net solids production is estimated to range from 0.34 lb VSS per lb BOD removed during treatment of primary effluent at 30 °C up to 1.18 lb VSS per lb BOD removed during treatment of raw wastewater at 10 °C. Net solids generation rates can be very low in some system designs (e.g., 0.1 lb VSS per lb BOD removed during treatment of primary effluent at 20 °C with a SRT of 180 days). Low solids production rates can be of great benefit to decentralized applications where operation and maintenance services are only provided monthly, quarterly or even less often.
  - In an attached growth ATU, biomass is attached to a support surface and sloughing may occur intermittently (low rate processes) or continuously (high rate processes). Net solids generation rates can be very low if the biomass solids are retained in an attached form for a long SRT.

- Excess solids produced during aerobic biological treatment of wastewater need to be removed periodically and properly managed.
  - Removal depends on the rate of generation and process function and performance. Removal is a process control measure that can be used to achieve a desired performance and effluent quality.
    - In an ATU that includes suspended growth in an extended aeration bioreactor, if excess solids build up they can lead to a MLVSS level that is too high and a F:M ratio that is too low, both of which can result in solids that settle poorly. Solids that settle poorly can wash out in the effluent and cause high (and potentially noncompliant) concentrations of BOD_5 and TSS in the effluent. Too high a solids loading in a secondary clarifier can also cause solids wash out.
  - Removal is required periodically based on the system design features and operating conditions and removal frequencies can range from daily up to monthly or longer.
    - For low-rate suspended growth and attached growth ATUs (e.g., extended aeration designs typical of aerobic treatments used in

decentralized systems) removal might only be needed every 3–6 months.
    - Removal of excess solids from a suspended growth ATU can occur in one of two ways: (1) by pumping out a portion of the mixed liquor in the aeration zone (common for small systems) or (2) by diverting a portion of the return flow conveying solids from a clarifier back to the aeration zone.
    - Removal of excess solids from an attached growth ATU typically occurs by wasting from the bottom of a secondary clarifier used for solids separation.
  - Excess solids that are removed from an ATU are often referred to as waste activated sludge.
  - In the United States, Federal and State regulations govern proper management of the waste activated sludge (and septage removed from a septic tank if used for primary treatment). Management approaches generally include the following options: (1) land treatment integrated with agriculture, (2) discharge to a local wastewater treatment plant, or (3) discharge to a specially designed treatment facility. Chapter 15 covers the management of waste activated sludge and other waste solids and residuals.
- Due to the complexities involved in aerobic biological treatment including the need for electrical and mechanical components (e.g., aerators, pumps, valves), proper function and performance of an ATU depends on provision of routine and reliable operation and maintenance. This includes timely removal and proper management of the excess solids that are generated.

7-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 7 are presented in the Slides section.

7-4. Terminology

Terminology introduced and used in Chap. 7 is defined below.

Activated sludge—A biological process where microorganisms are grown under aerobic conditions using organic matter in the influent wastewater as a source of food and energy.

Aeration zone—A term that describes the physical system within which active aeration occurs. Examples include an aeration chamber or compartment in a larger tank, a stand-alone tank of basin devoted to aeration, and so forth.

Aerobic—Refers to a biochemical state where microorganisms require oxygen to survive and function by using oxygen as an electron acceptor.

Aerobic treatment unit (ATU)—Refers to a physical system of compartments, tanks or basins used to establish an aerobic bioreactor and the supporting components and appurtenances used to achieve aerobic biological treatment of wastewater. Aerobic treatment unit (ATU) may also be used to refer to a small-scale packaged plant used for aerobic biological treatment of wastewater.

Anaerobic—Refers to a biochemical state where microorganisms do not require oxygen and utilize organic matter or hydrogen as electron donors and inorganic (e.g., nitrate, sulfate) or organic matter as electron acceptors. Some anaerobic organisms may react negatively or even die if oxygen is present.

Anammox—The name of a biological process that involves the simultaneous oxidation of ammonia nitrogen combined with denitrification of nitrite nitrogen.

Attached growth—Refers to a aerobic biological process where the microorganisms involved in treatment are attached to, and grow on, physical surfaces such as rocks or plastic honeycombs.

Autotrophic—Refers to a group of microorganisms that use an inorganic material as an electron donor (e.g., elemental sulfur) and acceptor (e.g., nitrate nitrogen).

Biomass—Biological material derived from living microorganisms involved in biological wastewater treatment.

Biosolids—Biosolids refers to treated sewage sludge that is made suitable for beneficial use through incorporation into soil and agriculture. In the United States the U.S. Environmental Protection Agency sets pollutant and pathogen requirements for biosolids relative to use for land application and surface disposal in Federal regulation 40 CFR Part 503, which sets standards for the use or disposal of sewage sludge.

Continuously stirred tank reactor (CSTR)—A type of reactor that is constantly mixed during a chemical or biochemical reaction.

Denitrification—Denitrification involves the reduction of nitrate to N_2O and N_2 gas under anoxic conditions (DO < 0.5 mg/L) by heterotrophic bacteria (different anaerobic and facultative bacteria) that utilize organic matter as a source of energy and organic carbon. Denitrification can also be carried out under anoxic conditions by autotrophic bacteria (*Thiobacillus denitrificans* and *Thiomicrospira denitrificans*) that can use sulfur as an electron donor and NO_3^- as an electron acceptor.

Extended aeration—A term that can be used to (1) refer in general to wastewater being aerated over a long period of time or (2) to a suspended growth flow-through aerobic treatment system that has a long solids retention time.

Floc—Refers to the clustering of microorganisms and biomass solids that develops during flocculant settling in a secondary clarifier of an aerobic treatment unit operation.

Food to microorganism ratio (F/M)—A design parameter that is defined as the substrate entering an aeration zone compared to the concentration of microorganisms in the aeration zone. Units are typically given as lb of BOD or COD per day per lb of MLVSS (which is equivalent to inverse days).

Heterotrophic—Heterotrophic bacteria (different anaerobic and facultative bacteria) utilize organic matter as a source of energy and organic carbon.

High rate process—A term that refers to an aerobic biological treatment process with respect to how fast it achieves biodegradation of organic matter in wastewater. A high rate process is one that only requires a relatively short period of aeration of the wastewater to achieve a secondary quality effluent.

Hydraulic retention time (HRT)—(1) In the context of confined treatment operations the hydraulic retention time is a design parameter that describes how long liquid remains in a specified chamber, basin, or tank. HRT is defined as the volume of the chamber, basin, or tank (e.g., gal) divided by the flow rate passing it through it (gal/day). (2) In the context of land-based treatment operations, hydraulic retention time refers to the length of time that water remains within a volume. Within a soil treatment unit this means between the location of the soil infiltrative surface and some depth of soil below it or distance in groundwater away from the point of recharge.

Kinetics—A term that refers to the process concerned with measuring and studying the rates of reactions.

Low rate process—A term that refers to an aerobic biological treatment process with respect to how fast it achieves biodegradation of organic matter in wastewater. A low rate process is one that typically has an extended period of aeration of the wastewater in order to achieve a secondary quality effluent.

Mean cell residence time (MCRT)—A design parameter that describes the average length of time that a microorganism remains in the aeration zone of an aerobic treatment unit.

Mixed liquor—A term used to refer to the contents of the aeration zone (compartment, tank, basin) within an activated sludge biological treatment system.

Mixed liquor volatile suspended solids (MLVSS)—A measure of the biomass cells in the mixed liquor within an aeration zone (i.e., a compartment, tank, or basin).

Nitrification—Nitrification involves the conversion of ammonia nitrogen to nitrite and nitrate nitrogen under aerobic conditions by autotrophic bacteria that utilize O_2 as an electron acceptor and CO_2 as a carbon source.

Overflow rate—The rate at which liquid in a secondary clarifier flows over the weirs within it to exit the aerobic treatment system.

Oxidation ditch—A type of aerobic treatment system that involves wastewater being aerated as it flows around an oval channel.

Plug flow—A flow regime where the velocity of the fluid is assumed to be constant across any cross-section of the tank, basin or other unit perpendicular to the axis of the inlet to the outlet flow path.

Rotating biological contactor (RBC)—A type of aerobic treatment system that consists of a tank through which wastewater flows and a cylindrical unit containing closely spaced disks that are supported on a rotating shaft just above the surface of the wastewater in the tank. Microorganisms grow on the disks and contact substrate as the disks are rotating through the wastewater and receive aeration through passive means via exposure to the atmosphere as they rotate above the liquid level.

Sequencing batch reactor (SBR)—A type of aerobic treatment system that consists of a single tank within which multiple steps occur in sequence. The steps typically include: fill, aerate, settle, decant and idle (waste biosolids).

Settleability—A term that refers to the tendency of a biomass suspended in wastewater to settle and compact by gravity under quiescent conditions.

Sloughing—Refers to the process by which biomass attached to support surfaces in an attached growth system such as a trickling filter (e.g., rocks or plastic honeycombs) separates from the surfaces and is carried out of the system.

Sludge—Sludge can have different meanings depending on the context. In general it refers to a liquid-solid mixture (mostly water with 1–10 % by wt. solids). As applied to wastewater it refers to the solids and associated water that are separated during the treatment of wastewater. This definition can include domestic septage and waste activated sludge.

Sludge age—A term that is sometimes used as a measure of how long biomass solids remain in an aerobic treatment system. Sludge age is defined as the total mass of MLVSS that are in the aeration tank divided by mass of VSS influent to the system.

Sludge volume index (SVI)—A crude measure of the settleability of mixed liquor solids resulting from aerobic biological treatment. It is defined as the volume of one gram of settled solids in one liter of solids containing wastewater (e.g., mixed liquor) after 30 min of quiescent settling.

Solids—(2) Scum and sludge that is separated from raw wastewater during treatment using solids separation methods (e.g., a septic tank or screening device). (3) Solid (and excess solids) that result from biological growth in a bioreactor used for aerobic biological treatment of wastewater (biomass) plus nonvolatile suspended solids that get entrained within flocs of biomass.

Solids retention time (SRT)—A term that describes the length of time that solids produced during biological wastewater treatment are retained in the aeration zone. SRT is equal to the total mass of mixed liquor volatile suspended solids in the aeration zone divided by the mass of volatile suspended solids wasted from the system.

Substrate—A term that describes the organic matter that microorganisms involved in biological treatment use as a source of organic matter and organic carbon.

Suspended growth—Refers to an aerobic biological process where the microorganisms involved in treatment are suspended in the liquid wastewater.

Volumetric loading rate—A design parameter used to size the aeration basin of an aerobic bioreactor that is equal to the mass of organic substrate (BOD or COD) per day per ft^3 of aeration zone.

Waste activated sludge (WAS)—Refers to the excess biomass that is produced during aerobic biological treatment and requires periodic removal from a bioreactor to maintain a desired solids retention time. Waste activated sludge can also include organic and mineral matter that becomes associated with the biological flocs that are separated from the liquid (either by clarification or filtration). The excess solids are removed by pumping out a portion of the aeration compartment, tank or basin or by diverting a portion of the return flow from a clarifier as a sludge with a low solids content (e.g., 1 % by wt. or less).

Waste biological solids—Refers to excess biomass that is removed from aerobic biological treatment operations including waste activated sludge from aerobic treatment units and membrane bioreactors and also excess biomass that sloughs off of media in recirculating porous media biofilters.

Yield (Y_N)—The net production of solids during aerobic biological treatment that is typically based on the characteristics of the wastewater being treated, the solids retention time, and the temperature.

7-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 7 are listed below.

AGB	Attached growth bioreactor
ATU	Aerobic treatment unit
BNR	Biological nutrient removal
BOD	Biochemical oxygen demand
COD	Chemical oxygen demand
CSTR	Continuously stirred tank reactor
EBPR	Enhanced biological phosphorus removal
F/M	Food to microorganism ratio

HRT	Hydraulic retention time
MCRT	Mean cell residence time
MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids
N	Nitrogen
OLR	Organic loading rate
OM	Organic matter
OR	Overflow rate
P	Phosphorus
RAS	Return activated sludge
RBC	Rotating biological contactor
R_E	Removal efficiency
S	Substrate, concentration of limiting substrate
SBR	Sequencing batch reactor
SGB	Suspended growth bioreactor
SRT	Solids retention time
SVI	Sludge volume index
T	Temperature
TSS	Total suspended solids
VSS	Volatile suspended solids
WAS	Waste activated sludge

A_c	Surface area of the clarifier
A_F	Trickling filter surface area
C_I	Influent concentration
C_E	Effluent concentration
D_F	Depth of the filter medium
f	Ratio of BOD_5 to ultimate BOD
F	Conversion factor
K	Constant
N_E	Effluent TKN concentration
N_I	Influent TKN concentration
OLR	Organic loading rate
O_2R	Total oxygen requirement
OR_D	Overflow rate for design
P_X	Waste activated sludge solids that needs to be wasted
Q	Daily flow rate
Q_D	Design daily flow rate
Q_E	Daily effluent flow rate
Q_W	Daily flow rate of waste solids
R	Recycle ratio
r_d	Rate of endogenous decay
r_g	Rate of bacterial growth
r_g'	Net rate of bacterial growth
r_{su}	Rate of substrate utilization

S_E	Effluent concentration of substrate (BOD or COD)
SF	Safety factor for nonideal conditions in clarifiers
S_I	Influent concentration of substrate (BOD or COD)
SLR	Solids loading rate
V_A	Volume of the aeration zone (or aeration tank or basin)
V_F	Volume of the trickling filter
V_i	Estimated settling velocity of the solids interface
V_M	Volume of the membrane zone (or tank or basin)
V_{max}	Maximum settling velocity of the interface
dX/dt	Rate of change of cells in the bioreactor
X	Concentration of VSS (or TSS)
X_A	Concentration of VSS (or TSS) in the aeration tank volume
X_R	Concentration of VSS (or TSS) in the recycle line back to the aeration tank
X_W	Concentration of VSS (or TSS) in the waste activated sludge
X_E	Concentration of VSS (or TSS) (or cells) in the effluent
X_I	Influent concentration of VSS (or TSS)
X_R	Concentration of VSS (or TSS) in the recycle line
Y	Maximum yield coefficient
Y_{Net}	Net solids production
Y_{obs}	Observed yield coefficient
k	μ_m/Y
k_d	Endogenous decay coefficient
K_S	Substrate concentration at 50 % of μ_m
μ	Specific growth rate
μ_m	Maximum specific growth rate
k_T	Reaction rate constant at temperature, T
k_{20}	Reaction rate constant at 20 °C
θ	Temperature activity coefficient

7-6. Problems

7.1. One common approach to aerobic biological treatment utilizes suspended growth processes, which are based on the concept of activated sludge. For aerobic treatment units that depend on suspended growth processes, which of the following statements are true: (1) the bioreactor consists of a tank or basin that is aerated to maintain DO and provide mixing, (2) the hydraulic retention time is longer than the solids retention time, (3) the mixed liquor volatile suspended solids concentration is used as a measure of biomass, (4) some biomass needs to be routinely wasted to maintain a desirable solids retention time, (5) the biological processes can be adversely impacted by certain chemicals.

7.2. Aerobic treatment units are normally designed to produce a high quality effluent. Which of the following best describes the effluent quality that would be expected from a properly functioning aerobic treatment unit?
   1. $BOD_5 = 150$ mg/L $TSS = 60$ mg/L $NO_3 = 50$ mg/L Fecal coliforms $= 10^6$ org/100 mL
   2. $BOD_5 = 150$ mg/L $TSS = 60$ mg/L $NH_4 = 50$ mg-N/L Fecal coliforms $= 10^6$ org/100 mL
   3. $BOD_5 = 30$ mg/L $TSS = 30$ mg/L $NH_4 = 5$ mg-N/L Fecal coliforms $\leq 10^3$ org/100 mL

7.3. During aerobic biological treatment, biomass solids that are produced in an aeration zone are retained in the system for an extended period of time. For a conventional suspended growth process, how is this normally accomplished?

7.4. Settleability of biomass solids produced in an aerobic biological treatment process is sometimes assessed with a simple lab test that yields a value known as the sludge volume index (SVI). How is the SVI value interpreted with respect to settleability?

7.5. In an aerobic treatment unit that uses a suspended growth biological process, what happens to the effluent quality if the biomass solids do not settle well?

7.6. Aerobic treatment units that utilize attached growth biological processes tend to be more resistant to upsets when the flow rate or composition of the influent wastewater changes. Why?

7.7. The magnitude of the net solids production, Y_N, during aerobic biological treatment depends primarily on what three key factors?

7.8. Can an aerobic biological treatment unit be designed and operated so that biomass solids never have to be wasted from the treatment system? Briefly explain your answer.

7.9. An extended aeration aerobic treatment unit is being designed to handle an average daily flow of 10,000 gal/day generated by a resort development in northern Montana where the wastewater temperature averages about 10 °C. Calculate the volume of the aeration tank (in gal), the hydraulic retention time (in days), the F/M ratio (lb/day BOD_5 per lb MLVSS) and the rate of wasting excess solids from the aeration tank contents (gal/day).

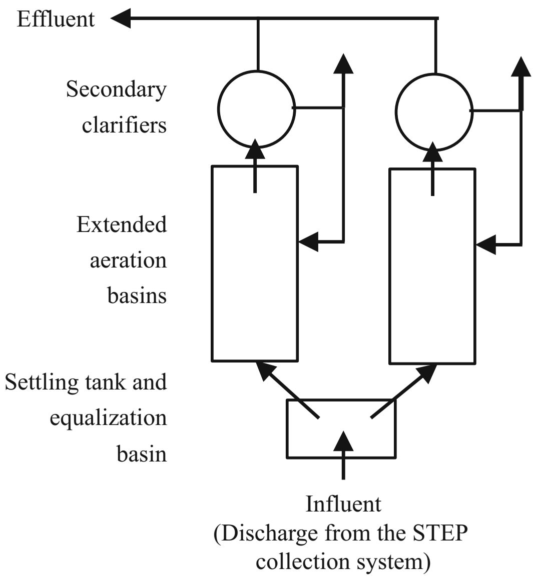
   Given information and assumed values: Peaking factor for maximum daily flow = 2.0. The influent to the aerobic treatment unit is septic tank effluent that has an expected BOD_5 of 200 mg/L. The aeration tank MLVSS = 4000 mg/L and the solids retention time = 20 days. Solids will be wasted from the aeration tank.

7.10. An extended aeration treatment unit was installed at an apartment building outside Lyons, Colorado in 2010. The ATU had been operating properly and performing well for over three years, producing an effluent with $BOD_5 \leq 20$ mg/L, $NH_4^+ < 4$ mg/L, and DO > 4 mg/L. Then the flood of September 2013 hit and there were power outages lasting for nearly two weeks. Compared to the period prior to the flood, during the period with no power how would the effluent quality from the ATU likely change with respect to the following parameters?

1. DO (mg/L) in the effluent: no change______ increase______ decrease______
2. BOD_5 (mg/L) in the effluent: no change______ increase______ decrease______
3. NO_3^- (mg-N/L) in the effluent: no change______ increase______ decrease______

7.11. For the Mines Park housing development located on the Colorado School of Mines campus in Golden, Colorado you are tasked with preliminary design of an aerobic treatment unit. The ATU should be capable of producing a high quality effluent (BOD_5, COD, and TSS ≤ 20 mg/L) that would be suitable for disinfection and reuse for lawn and garden irrigation around Mines Park. Based on the information given, answer the following design questions. (1) What is the volume (gal) of each of the two aeration tanks needed to handle the design daily flow and what geometry would you propose (length and width in ft)? (2) Given the volume calculated what is the HRT (h) and is it within the range of typical values for an extended aeration process? (3) What is the rate of solids production (lb-VSS/day) and the rate of wasting (gal/day) required to maintain the target MLVSS? (4) Determine the oxygen requirements assuming oxidation of organic matter and nitrification (in lb-O_2/day). (5) Determine the area of the secondary clarifier required and check the overflow rate and solids loading rate to be sure they are within recommended limits.

Given information and assumed values: The average daily flow, $Q_A = 28{,}425$ gal/day. A STEP collection system will convey septic tank effluent (STE) to the treatment site. The STE quality expected: COD $= 220$ mg/L, $BOD_5 = 160$ mg/L, TSS $= 80$ mg/L, TKN $= 60$ mg/L. The aerobic treatment unit will employ an extended aeration suspended growth process with solids recycle and wasting from the clarifier. The design flow (Q_D) equals the maximum recurring daily flow (PF $= 2$). The average wastewater temperature $= 15\,^{\circ}$C and the target solids retention time $= 15$ days. The target MLVSS in the aeration tank, $X_A = 3200$ mg/L and in the recycle line, $X_R = 8000$ mg/L. Two aeration units will be provided in parallel (design flow will be split 50:50) and each will have a water depth $= 10$ ft and L:W ratio $=$ approx. 3:1.



7.12. For the development presented in Problem 7.11, complete preliminary sizing of a low-rate trickling filter. Determine the surface area required and the volume and depth of the filter bed.

Given information and assumed values: The development has the same features as described in Problem 7.11. For the biofilter assume there are two filters in parallel with a hydraulic loading rate of 25 gal/day/ft^2 and an organic loading rate of 5 lb-BOD$_5$/day per 1000 ft^3.

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Slides of Chapter 7

Decentralized Water Reclamation

Chapter 7: Treatment using Aerobic Bioreactors

Contents

7.1

7-1. Introduction

- Aerobic biological treatment of wastewaters
  - Primary treated wastewaters normally contain dissolved and colloidal organic matter and fine particulates that are not removed during primary treatment
  - Aerobic biological processes can provide further treatment of primary effluents such as septic tank effluent
  - Aerobic biological processes can be used to achieve:
    - Advanced secondary treatment
      - Removal of soluble and colloidal organic matter (BOD)
      - Removal of colloidal and fine particulates (TSS)
      - Nitrification of ammonia (NH_4^+)
    - Nutrient removal
      - Aerobic treatment can also be used for enhanced removal of nitrogen and phosphorus as described in Chap. 13

7.2

- Treatment using aerobic bioreactors
  - Aerobic treatment exploits microorganisms (biomass) that can use constituents in wastewater as a source of carbon and nutrients and also energy for growth
  - Aerobic treatment requires a flow regime to contact biomass with wastewater while maintaining proper aeration status
    - This can be accomplished in bioreactors which can be classified based on biomass and reactor type (Fig. 7.1)
    - Aerobic treatment of wastewater in a bioreactor is generally based on a 100^+-year old process known as "activated sludge"
  - Chapter 7 is focused on aerobic treatment in bioreactors such as included in Fig. 7.1
    - There are other treatment operations that rely on aerobic biological processes that are covered in other chapters

7.3

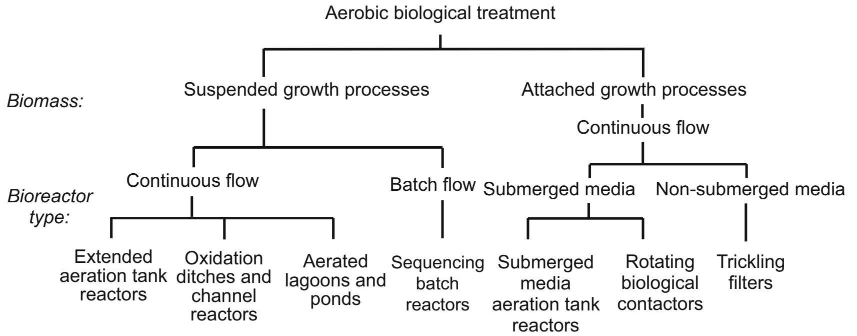


Fig. 7.1 Classification of aerobic biological treatment methods based on how the biomass is contacted with the wastewater within different types of bioreactors

Note: in addition to the methods shown in Fig. 7.1 there are other treatment methods that rely on aerobic biological processes that are covered in other chapters, including porous media biofilters (Chap. 8), membrane bioreactors (Chap. 9), constructed wetlands (Chap. 10), soil-based treatment operations (Chaps. 11 and 12), nutrient reduction methods (Chap. 13), and waste solids and residuals management methods (Chap. 15)

7.4

- Activated sludge processes
  - "Activated microbial biomass" involves a consortium of bacteria and other microorganisms that are contacted with wastewater and aerated to support microbial growth
    - In an aerated tank the microbial biomass and wastewater are referred to as "mixed liquor" (Fig. 7.2)
      - * Aeration tank is used throughout Chap. 7 to denote an aerated zone, compartment, basin, channel, etc.
    - Mixed liquor volatile suspended solids (MLVSS) is often used as a measure of biomass concentration

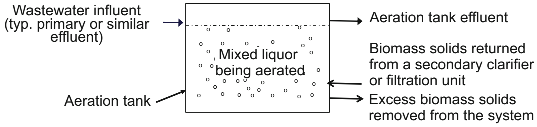


Fig. 7.2 Illustration of mixed liquor being aerated in an aeration tank

7.5

- Several terms are used to describe how long biomass solids remain in the biological treatment system
  - Sludge age

$$\text{Sludge age (days)} = \frac{\text{Total lb of MLVSS in aeration tank}}{\text{lb/day of VSS influent to the system}}$$

  - Solids Retention Time (SRT)

$$\text{SRT (days)} = \frac{\text{Total lb of MLVSS in aeration tank}}{\text{lb/day of VSS wasted from the system}}$$

  - Mean Cell Residence Time (MCRT)

$$\text{MCRT (days)} = \frac{\text{Total lb of MLVSS in aeration tank}}{\text{lb/day of VSS wasted and in clarifier effluent}}$$

  - For practical purposes, SRT $\approx$ MCRT

7.6

- Activated sludge processes for secondary treatment in a bioreactor can be classified based on:
  - The rate of organic matter removal from the liquid wastewater
  - If nitrification occurs during aeration ($NH_4^+ \rightarrow NO_3^-$)
- Classification based on organic matter removal and nitrification—examples of two different options
  - High rate processes that are non-nitrifying: These processes have a high treatment capacity (e.g., gal/day per ft^3 of reactor volume) for organic matter removal but do not achieve nitrification
  - Low rate processes that are nitrifying: These processes have a lower treatment capacity for organic matter removal but they do achieve nitrification
    - In low rate processes wastewater is typically aerated over an extended period of time
    - Low rate processes that are nitrifying are often used in decentralized systems

7.7

- Basic features of aerobic treatment units
  - For decentralized systems, aerobic biological treatment is often implemented using aerobic treatment units (ATUs)
  - ATUs are often implemented as a packaged unit such as illustrated in Figs. 7.3, 7.4 and 7.5

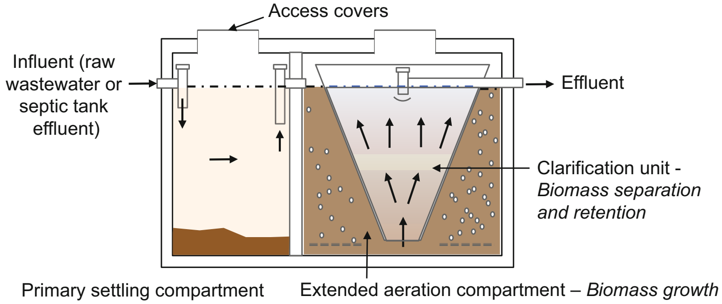
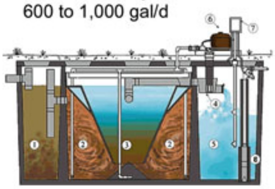


Fig. 7.3 Illustration of an ATU with an integrated primary settling unit, suspended growth bioreactor, and secondary clarifier

7.8

- Examples of commercially available small ATUs
  - Examples of ATUs that utilize suspended growth bioprocesses are shown in Fig. 7.4

Hoot Aerobic systems
600 to 1,000 gal/d



Source: www.hootsystems.com

Norweco Singular
500 to 1,500 gal/d

Source: www.norweco.com/wiki/ATS.htm

Delta Whitewater
400 to 1,500 gal/d

Source: www.deltaenvironmental.com/df_series.asp

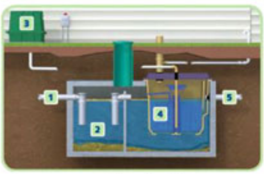
Fig. 7.4 Examples of three commercially available ATUs that utilize submerged growth bioprocesses.

Note: many of these ATU manufacturers also provide larger package plant ATUs that handle flows up to 100,000 gal/day or more. Refer to the company websites for more information. *Source:* www.hootsystems.com. *Source:* www.norweco.com/wiki/ATS.htm. *Source*: www.deltaenvironmental.com/df_series.asp

7.9

- Examples of ATUs that utilize attached growth bioprocesses are shown in Fig. 7.5

Bio-Microbics MicroFAST
500 to 9,000 gal/d



Source: www.biomicrobics.com

MicroSeptec Enviroserver
600 to 2,500 gal/d

Source: www.microseptec.com

Multi-Flo
500 to 1,500 gal/d

Source: www.Consolidatedtreatment.com/products/multi-flo-wastewater-treatment-system.aspx

Fig. 7.5 Examples of three commercially available ATUs that utilize attached growth bioprocesses.

Note: many of these ATU manufacturers also provide larger package plant ATUs that handle flows up to 100,000 gal/day or more. Refer to the company websites for more information. *Source:* www.biomicrobics.com *Source:* www.microseptec.com. *Source*: www.consolidatedtreatment.com/products/multi-flo-wastewater-treatment-system.aspx

7.10

- Where is aerobic treatment used?
  - Where secondary treatment is warranted, for example:
    - For treatment of wastewaters with high organic matter content such as those from restaurants and other commercial sources
    - To enable or enhance soil-based treatment operations
      - * Enable higher rate subsurface infiltration systems
      - * Reduce the separation distance to shallow groundwater or setback distance to a property line
    - To enable surface discharge and reuse of effluent which often requires secondary or higher quality effluent
  - Aerobic treatment is used in decentralized systems serving:
    - Residential units and developments
    - Commercial and institutional buildings and developments
    - Mixed-use developments, small towns and urban areas

7.11

7-2. Treatment Performance

- Aerobic treatment units are normally used to achieve advanced secondary treatment
  - BOD and TSS removal
    - Biodegradable organics (dissolved and colloidal) are converted to cell mass and CO_2 which can be separated from the effluent
    - Reduced inorganics which can exert BOD can be converted to oxidized forms (e.g., NH_4^+ to NO_3^-)
    - TSS in the form of colloidal and fine particulates are biodegraded and/or removed by flocculant settling
  - Nitrification of ammonia
  - During normal secondary treatment there can also be some degree of removal of nutrients and pathogens
    - Aerobic treatment units can be specifically designed and operated to achieve enhanced removal of N and P (Chap. 13)

7.12

- Treatment efficiency
  - Treatment efficiency is illustrated in Fig. 7.6 and can be determined using Eq. 7.1
    - Application of Eq. 7.1 can be limited if the composition of the influent (C_I) to the ATU is unknown or has to be assumed

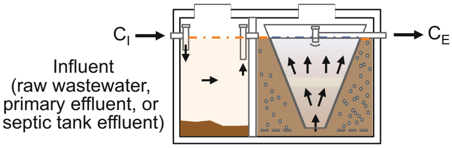


Fig. 7.6 Illustration of treatment efficiency achieved within an aerobic treatment unit

$$R_E = \left(\frac{C_I - C_E}{C_I}\right) \times 100\% \qquad (7.1)$$

Where:
R_E = removal efficiency (%)
C_I = influent concentration (mg/L)
C_E = effluent concentration (mg/L)

7.13

- Aerobic treatment efficiencies for constituents of potential concern are presented in Table 7.1

Table 7.1 Representative treatment efficiency achieved within a well-designed and operated aerobic unit

Constituent group	Effluent concentration (mg/L) or removal (%)	Potential processes involved in treatment
BOD_5	$<$20 mg/L	Dissolved and colloidal organics are converted to cell mass and CO_2 which can be separated from the effluent
TSS	$<$20 mg/L	TSS in the form of colloidal and fine particulates are biodegraded and/or removed by flocculant settling
Nitrogen	Up to 100 % NH_4^+ 15–25 % total N	Nitrification of NH_4+ compounds to NO_3^- but only 15–25 % removal of total N unless the process is designed for total N removal
Phosphorus	10–20 % total P	Incorporation of P into cell mass and potential for sorption
Pathogens	90–99 %	Potential for limited die-off and inactivation but negligible impact on high concentrations normally found in STE
Trace organic chemicals	0 to $>$90 %	Near zero removal for some compounds but up to 90 % or more removal of compounds that are susceptible to aerobic biodegradation

7.14

- Aerobic treatment unit effluent composition
  - Factors affecting treatment efficiency and effluent composition
    - Source and consistency of the influent to an ATU
    - ATU design and operating conditions
      - Bioreactor type and sizing
      - Actual flow and organic load versus design
      - Clarifier sizing and solids separation efficiency
      - Temperatures and effects on bioprocesses
    - Degree of routine operation and maintenance that is provided
    - ATU treatment efficiency can decline during upsets, e.g.:
      - During periods of very low or high flow, low or high pH, or elevated biotoxics (e.g., quaternary ammonium salts or high chlorine)

7.15

7-3. Principles and Processes

- Biological treatment depends on a healthy biosystem
  - A "healthy" biological treatment system involves a consortium of microorganisms that help achieve treatment efficiency and settleability and retention of biomass
  - The nature of the consortium depends on conditions in the biological system and the length of time the microorganisms spend in the system (Fig. 7.7)
  - Bacteria are important for aerobic biological treatment
    - Bacteria and other organisms grow in the biological system
    - Growth normally occurs in phases depending on food supply as shown in Fig. 7.8

7.16

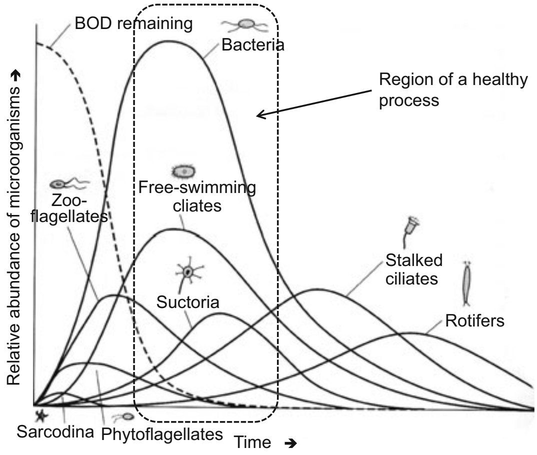
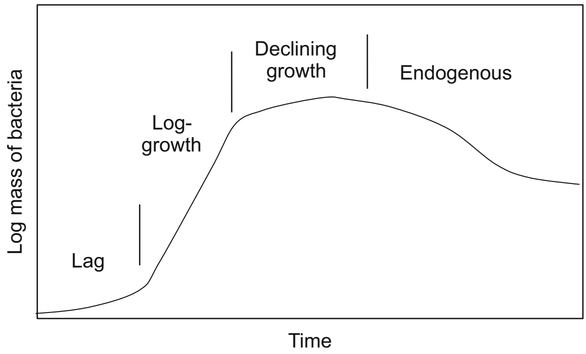


Fig. 7.7 Relative abundance of microorganisms within a healthy activated sludge during aerobic biological treatment (after Fig. 7.5 in Crites and Tchobanoglous 1998)

7.17



Lag – Bacteria require time to acclimate

Log-growth – Excess food is available and rate of metabolism and growth is only controlled by processing rate

Declining growth – Limitations in food supply reduce rate

Endogenous – Bacteria metabolize their own protoplasm

Fig. 7.8 Illustration of bacterial growth as a function of time and food supply

7.18

- Biological treatment depends on bacterial metabolism
  - Metabolism requires a source of energy and carbon
  - Most biological processes used for waste treatment are driven by heterotrophic bacteria (Table 7.2)
    - Energy is derived from the transfer of electrons from a donor to an acceptor and carbon is derived from organic matter
    - Heterotrophic modes of metabolism are summarized in Table 7.3

Table 7.2 Classification of biological processes based on sources of energy and carbon

Classification	Nutrition group	Source of energy	Source of carbon
Autotroph	Photoautotrophic Chemoautotrophic	Derive energy from sunlight Derive energy from oxidation of inorganic substances	CO_2
Heterotroph	Photoheterotrophic Chemoheterotrophic	Derive energy from sunlight Derive energy from oxidation of organic matter	Organic matter

7.19

Table 7.3 Features of heterotrophic modes of metabolism involved in aerobic treatment processes

Mode of metabolism	Electron donor	Electron acceptor	End products	Relative energy potential
Aerobic respiration	Organic matter NH_4^+ Fe^{+2} S^{-2}	O_2 O_2 O_2 O_2	CO_2, H_2O NO_2^-, NO_3^-, H_2O Fe^{+3} SO_4^{-2}	Higher
Anaerobic respiration	Organic matter Organic matter H_2 H_2	NO_3^- SO_4^{-2} SO_4^{-2} CO_2	N_2, CO_2, H_2O S^{2-}, CO_2, H_2O S^{-2}, H_2O CH_4, H_2O	
Fermentation	Organic matter	Organic matter	Organic compounds, CO_2, CH_4	Lower

7.20

- Equation 7.2 is a simplified reaction (not balanced) for aerobic biodegradation of organic matter (substrate) in wastewater

$$\text{Organic substrate} + \text{oxygen} \Rightarrow \text{energy} + \text{cells} + \text{other products}$$
$$C_{10}H_{19}O_3N + O_2 \Rightarrow C_5H_7O_2NP_{0.08} + CO_2 + H_2O + NH_3 \quad (7.2)$$

  - The level of substrate (S) available affects the reaction
    - Assuming substrate is represented by BOD_5
    - With low BOD_5 levels
      - To generate energy, a greater proportion of BOD_5 is used to generate CO_2 and fewer cells are produced
    - With high BOD_5 levels
      - Since there is adequate BOD_5 to produce energy, more cells are produced

7.21

- Cell growth and substrate utilization
  - Bacterial cell growth and wastewater substrate used in aerobic bioreactors are often represented by lumped parameters
    - For the concentration of cells in the aeration tank:
      - Volatile suspended solids (VSS) are typically used
        - For the contents of an aeration tank this is often referred to as "mixed liquor VSS" or MLVSS
      - MLVSS is a fraction of the mixed liquor suspended solids (MLSS)
        - For domestic wastewater in the United States, MLVSS is typically about 65–80 % of the MLSS
        - For some wastewaters, MLVSS can be different (e.g., in Bangkok MLVSS can be only 45–55 % of the MLSS)
    - For the concentration of substrate in the aeration tank:
      - Biochemical oxygen demand (BOD)
      - Chemical oxygen demand (COD)

7.22

- Growth reactions are important to treatment
  - Primary growth reactions target dissolved organic matter
    - Organic matter is degraded to provide C for cell growth and oxidized to CO_2 and H_2O to provide energy
  - Growth reactions can also achieve removal of nutrients
    - Transformation and removal of nitrogen
      - Nitrification

        NH_4^+ is converted to NO_3^-
        Organic-N can be converted to NH_4^+ and then to NO_3^-
      - Denitrification

        NO_3^- is converted to N_2O and N_2 gas
    - Removal of phosphorus
      - P can be incorporated into new cells
      - Excess P is stored in cells under certain conditions

7.23

- During aerobic treatment, growth is promoted
  - Dissolved oxygen (DO) is provided to support bacterial growth
  - Growth involves use of substrate in the wastewater
    - Substrate is typically viewed as a constituent of concern (e.g., BOD or COD)
    - Organic substrate is biodegraded to yield carbon
    - Energy is derived by transferring electrons, e.g.:
      - From organic matter to O_2
      - From NH_4^+, Fe^{2+} or S^{2-} to O_2
  - Rates of growth and substrate utilization are related
    - Higher growth rate equals a higher substrate utilization rate
    - Substrate utilization equates to treatment (e.g., BOD or COD removal)

7.24

- Kinetics of cell growth and substrate utilization
  - Some substrate (organic matter) is converted to new cells and some is oxidized to inorganic and organic end products
  - The kinetics of growth and utilization are complicated, but underpin aerobic treatment process function and performance
  - The rate of cell growth and substrate utilization are related
    - The rate of growth needs to account for (1) energy required for cell maintenance and (2) cell death and loss by predation
      - Lump these factors as "endogenous decay"
    - Net cell growth occurs and some biomass needs to be wasted periodically to sustain proper bioreactor function and performance
    - Temperature effects on rates can be substantial
      - Within a certain range, higher temperatures yield higher rates
      - But very low or very high temperatures can inhibit metabolism

Note: In depth coverage of biological treatment kinetics can be found in reference texts such as Grady et al. (2011) and Tchobanoglous et al. (2014).

7.25

- Summary of kinetics of growth and substrate utilization
  - In a batch or continuously mixed culture of bacteria, with no food supply limitations, growth can be defined by Eq. 7.3
  - If substrate or nutrients are limiting, the specific growth rate can be defined by Eq. 7.4 (Fig. 7.9)

$$r_g = \mu X \tag{7.3}$$

$$\mu = \mu_m \left(\frac{S}{K_S + S} \right) \tag{7.4}$$

Where:

r_g = rate of bacterial growth (mgVSS/L per day)

μ = specific growth rate (day^{-1})

μ_m = maximum specific growth rate (day^{-1})

X = concentration of bacteria (mgVSS/L)

S = concentration of growth limiting substrate (mgBOD or COD/L)

K_S = substrate concentration at 50 % of μ_m (mg/L)

7.26

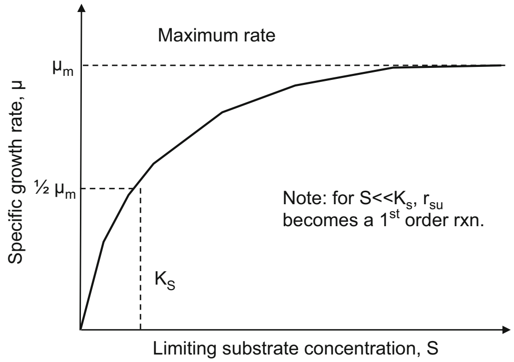


Fig. 7.9 Illustration of specific growth rate as a function of substrate concentration

7.27

- Combining Eqs. 7.3 and 7.4 yields the rate of growth given by Eq. 7.5

$$r_g = \left[\mu_m\left(\frac{S}{K_s + S}\right)\right]X \tag{7.5}$$

Where:
r_g = rate of bacterial growth (mgVSS/L per day)
μ_m = maximum specific growth rate (day^{-1})
X = concentration of bacteria (mgVSS/L)
S = concentration of growth limiting substrate (mg/L BOD or COD)
K_S = substrate concentration at 50 % of μ_m (mg/L)

7.28

- Growth and substrate utilization
  - A portion of substrate (organic matter) is converted to new cells and a portion is oxidized to inorganic and organic end products
  - Rate of growth and substrate utilization are related as given by Eq. 7.6

$$r_g = -Y r_{su} \tag{7.6}$$

Where:
r_g = rate of bacterial growth (mgVSS/L per day)
Y = maximum yield coefficient (mgVSS/mg BOD or COD)
= mass of cells formed to mass of substrate consumed during a fixed period of log growth
r_{su} = rate of substrate utilization (mg/L BOD or COD used per day)

Note: the "–" sign denotes substrate is being removed from the system.

7.29

- Combining Eqs. 7.5 and 7.6 yields the rate of substrate utilization given by Eq. 7.7

$$r_{su} = -\frac{1}{Y}\left[\mu_m\left(\frac{S}{K_s + S}\right)X\right] \tag{7.7}$$

$$r_{su} = -k\left(\frac{S}{K_S + S}\right)X$$

Where:
r_{su} = rate of substrate utilization (mg/L BOD or COD used per day)
μ_m = maximum specific growth rate (day^{-1})
X = concentration of bacteria (mgVSS/L)
S = concentration of growth limiting substrate (mg/L BOD or COD)
K_S = substrate concentration at 50 % of μ_m (mg/L)
Y = maximum yield coefficient (mgVSS/mg BOD or COD)
k = μ_m/Y (day^{-1})

7.30

- Accounting for endogenous decay
  - * The rate of growth needs to account for:
    - – Energy required for cell maintenance
    - – Cell death and loss by predation
  - * Lump these factors as "endogenous decay" (Eq. 7.8)

$$\text{Cells} + \text{oxygen} \xrightarrow{\text{bacteria}} \text{simple prod.} + \text{stable org. mtr.} + \text{energy} \quad (7.8)$$

$$C_5H_7O_2NP_{0.08} + 5O_2 \xrightarrow{\text{bacteria}} 5CO_2 + 2H_2O + NH_3 + \text{stable org. mtr.} + \text{energy}$$

  - * Rate of endogenous decay is given by Eq. 7.9

$$r_d = -k_d\ X \quad (7.9)$$

Where:
r_d = rate of endogenous decay (mgVSS/L per day)
k_d = endogenous decay rate (day^{-1})
X = concentration of cells (mgVSS/L)

7.31

- Combining Eq. 7.9 with Eqs. 7.5 and 7.6 yields the net rate of growth given by Eqs. 7.10 and 7.11

$$r_g' = r_g - r_d = \left[\mu_m\left(\frac{S}{K_s + S}\right)\right]X - k_dX \quad (7.10)$$

$$r_g' = -Y\,r_{su} - k_dX \quad (7.11)$$

Where:
r_g' = net rate of bacterial growth (mgVSS/L per day)
μ_m = maximum specific growth rate (day^{-1})
X = concentration of bacteria (mgVSS/L)
S = concentration of growth limiting substrate (mg/L BOD or COD)
K_S = substrate concentration at 50 % of μ_m (mg/L)
k_d = endogenous decay rate (day^{-1})
Y = maximum yield coefficient (mgVSS/mg BOD or COD)
r_{su} = rate of substrate utilization (mg/L BOD or COD used per day)

7.32

- Cell yield
  - Net cell growth occurs as substrate is utilized
  - An observed cell yield coefficient, Y_{obs} can be defined by Eq. 7.12

$$Y_{obs} = \frac{r_g{}'}{r_{su}} \tag{7.12}$$

Where:
Y_{obs} = observed yield coefficient (mgVSS/mg BOD or COD removed)
$r_g{}'$ = net rate of bacterial growth (mgVSS/L per day)
r_{su} = rate of substrate utilization (mg/L BOD or COD per day)

7.33

- Temperature affects the rates of processes involved in biological treatment
  - Very low or very high temperatures can inhibit metabolism involved in certain biological processes
  - Within a favorable range of temperatures, the temperature effects on reaction rates are often expressed by Eq. 7.13

$$k_T = k_{20}\theta^{(T-20)} \tag{7.13}$$

Where:
k_T = reaction rate constant at temperature, T °C
k_{20} = reaction rate constant at 20 °C
T = temperature (°C)
θ = temperature activity coefficient (–)
= Values of θ vary from about 1.02 to 1.09 with 1.04 being typical

7.34

- Aerobic bioreactor configurations
  - Bioreactors utilized to achieve aerobic biological treatment have features that affect design and operation
    - Biomass can be suspended or be attached to surfaces
    - Reactor can be completely mixed or it can have a plug flow regime
    - Biomass solids can be recycled within the system or not recycled
    - Treatment can target removal of organic substrates and/or nutrients
      - Design for high rate organic matter (OM) removal alone
      - Design for intermediate rate OM removal but no nitrification
      - Design for low rate OM removal and nitrification
    - The bioreactor system can have a single stage or multiple stages
      - A stage is a single unit of aeration w/ clarification
    - Systems can also be laid out with sequential zones of aeration and no aeration to create zones of anoxic or anaerobic conditions
      - Sequencing zones is commonly used for nutrient removal

7.35

- Examples of different bioreactor configurations are illustrated in Figs. 7.10 and 7.11

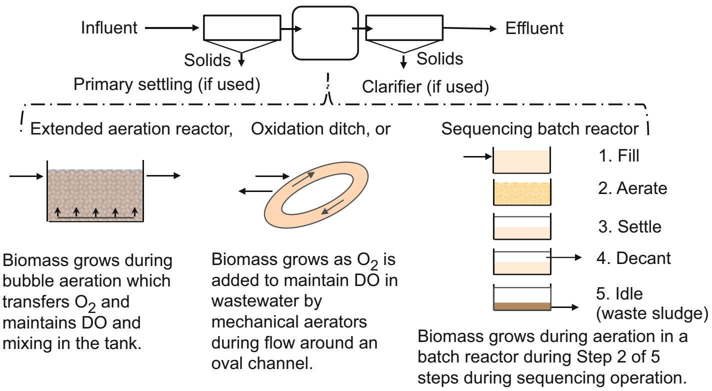


Fig. 7.10 Simplified schematic of unit operations that implement suspended growth aerobic biological treatment

7.36

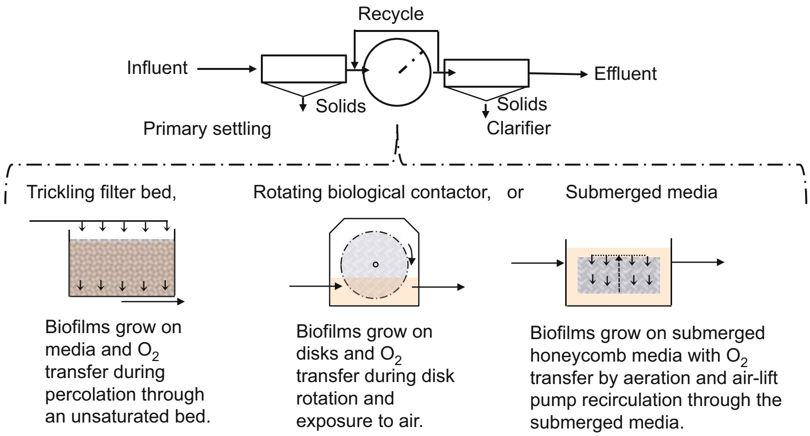


Fig. 7.11 Simplified schematic of unit operations that implement attached growth aerobic biological treatment

7.37

- Decoupling hydraulic and solid retention times
  - Water flows through a bioreactor but biomass is retained
    - This is fundamental to an activated sludge process
  - Retention of biomass solids can be achieved in several ways
    - Through settling of biomass solids in a clarifier following the aeration tank in a suspended growth, flow-through bioreactor (Fig. 7.12)
    - Through settling of biomass solids during a clarification step in a suspended growth, sequencing batch reactor
    - Through attachment of biomass to physical structures placed in the aeration tank such as stones or plastic honeycombs in an attached growth bioreactor
    - Through use of a membrane through which biomass solids cannot pass (Membrane Bioreactors are described in Chap. 9)

7.38

- Figure 7.12 illustrates a basic activated sludge process where aeration tank biomass solids are separated in a secondary clarifier
  - Biomass solids are returned to the aeration tank as Return Activated Sludge
  - Excess biomass solids are removed as waste activated sludge by (a) pumping from the aeration tank or (b) diverting a portion of the underflow of the secondary clarifier

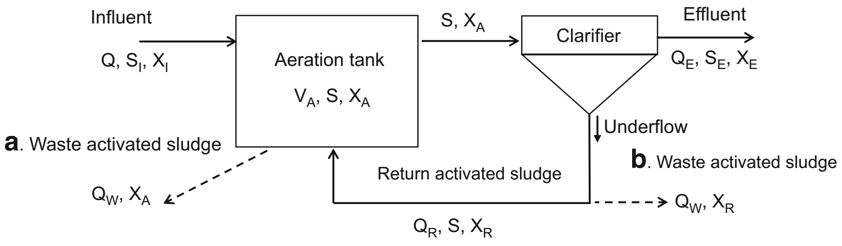


Fig. 7.12 Simplified illustration of a common suspended growth activated sludge system with solids separation by clarification and solids wasting from (**a**) the aeration tank or (**b**) from the recycle line from the secondary clarifier

7.39

- For the flow regime shown in Fig. 7.12b, the hydraulic retention time (HRT) and the solids retention time (SRT) are given by Eqs. 7.14 and 7.15

$$\mathrm{HRT} = \frac{V_A}{Q_D} \qquad (7.14) \qquad\qquad \mathrm{SRT} = \frac{V_A X_A}{Q_W X_W + Q_E X_E} \approx \frac{V_A X_A}{Q_W X_W} \qquad (7.15)$$

Where:
HRT = hydraulic retention time (day)
SRT = solids retention time (day)
V_A = aeration tank volume (gal)
Q_D = design daily flow rate (gal/day) (e.g., $Q_A \times$ PF)
Q_W = daily flow rate of waste solids (gal/day)
Q_E = daily effluent flow rate (gal/day)
X_A = concentration of VSS in the aeration tank (mg-VSS/L)
X_W = concentration of VSS in the waste activated sludge (mg-VSS/L)
X_E = concentration of VSS in the effluent (mg-VSS/L) (assumed negligible compared to X_A or X_W

7.40

- Mass balances on cells and substrate in the bioreactor
  - Mass balances are used to formulate expressions for cell growth and substrate utilization (e.g., Eq. 7.16 based on Fig. 7.12b)

Accumulation = inflow—outflow + net growth

$$\frac{dX}{dt}V_A = QX_I - (Q_W X_R + Q_E X_E) + V_A r_g' \tag{7.16}$$

Where:
dX/dt = rate of change of cells in the bioreactor (mg-VSS/L per day)
V_A = volume of aeration unit (gal)
Q = influent flow rate (gal/day)
Q_W = wasting flow rate (gal/day)
Q_E = effluent flow rate (gal/day)
X_I = influent concentration of cells (mg-VSS/L)
X_R = concentration of cells in the recycle line (mg-VSS/L)
X_E = concentration of cells in the effluent (mg-VSS/L)
r_g' = net rate of growth of cells (mg-VSS/L per day)

7.41

- Solids generation, retention and wasting
  - Cells are generated as substrate is utilized for growth
  - The net generation rate depends on the rate of cell growth, the SRT and the endogenous decay rate
    - The rates are temperature dependent, generally increasing with increasing temperature
  - In an activated sludge process, there is a need for wasting of excess cells and other organic and inorganic solids that become associated with them during clarification
    - Assuming cells in the effluent are much lower than cells in the aeration tank, at steady-state the net rate of cell mass growth equals the rate of cell mass solids being wasted (Eq. 7.17)
    - The net rate of growth for a given MLVSS in an aeration tank is equal to the inverse of the SRT (Eq. 7.18)

7.42

$$r_g{}'V_A \approx Q_W X_W \Rightarrow \frac{r_g{}'V_A}{X_A} \approx \frac{Q_W X_W}{X_A} \tag{7.17}$$

$$\frac{r_g{}'}{X_A} \approx \frac{Q_W X_W}{X_A V_A} \approx \frac{1}{SRT} \tag{7.18}$$

Where:

$r_g{}'$ = net rate of bacterial growth (mgVSS/L per day)
V_A = aeration tank volume (gal)
Q_W = daily flow rate of waste solids (gal/day)
X_A = concentration of cells in the aeration tank (mgVSS/L)
X_W = concentration of cells in the wasted solids (mgVSS/L)
SRT = solids retention time (day)

7.43

- Wasting of activated sludge solids
  - In a suspended growth process, solids can be wasted by:
    - Removal directly from the aeration tank (Fig. 7.12a)
    - Removal from the recycle line that returns solids from a secondary clarifier to the aeration tank (Fig. 7.12b)
  - In an attached growth process, solids can be wasted by:
    - Sloughing of solids from media supporting attached growth followed by clarification
  - Important to solids separation by clarification, the conditions under which cell growth occurs can influence the character of the activated sludge with respect to its settleability
    - Settleability is critical to achieving activated sludge separation from liquid wastewater, which is often accomplished using a clarification process (Fig. 7.12)
  - Activated sludge solids that are wasted need to be properly treated for disposal or beneficial recovery (see Chap. 15)

7.44

- Biological nutrient removal
  - While more complicated, aerobic treatment units can be designed for nutrient removal as summarized in Table 7.4
    - Biological nutrient removal is covered in Chap. 13

Table 7.4 Features of biological processes for nutrient removal

Nutrient	Process	Description	Basic requirements
N	Nitrification	Conversion of NH_4^+ to NO_2^- and NO_3^-	Commonly occurs during aerobic biological treatment when the SRT is long and adequate DO is maintained
	Denitrification	Conversion of NO_3^- to N_2	Sequentially occurs after nitrification but requires anoxic conditions and presence of a suitable organic substrate
P	Enhanced removal	Microbes are stimulated to take up excess P and cells containing excess P are then removed	More difficult to implement and control; Requires an anaerobic zone followed by an aerobic zone so microbes use P for cell maintenance, synthesis and energy transport, but also store P for future use

7.45

7-4. Design and Implementation

- Considerations for design and implementation (D&I) of aerobic bioreactors to achieve secondary treatment and nitrification
  - Source of wastewater to be treated
  - Bioreactor configurations and specific D&I options
    - Design of suspended growth bioreactors (SGB)
      - Design parameters
      - Bioreactor volume, biomass solids separation and retention, solids production and wasting, oxygen and energy needs
    - Design of attached growth bioreactors (AGB)
      - Design parameters
      - Bioreactor volume, other considerations
  - System installation and startup at the site
  - System operation and maintenance (O&M) requirements
  - Appurtenances and integral treatment operations

7.46

- D&I considerations—Wastewater source
  - Aerobic treatment can be used for wastewaters of widely different characteristics
    - Wastewaters from residential and nonresidential buildings and developments, especially where BOD concentrations are high
    - Buildings and developments should have relatively continuous and consistent day-to-day wastewater flow and composition
  - Pretreatment requirements
    - Primary treatment can be beneficial
    - Flow equalization can be important and can be accomplished in:
      - A septic tank or first compartment of the bioreactor system
      - An equalization tank upstream of the bioreactor
    - Consideration needs to be given to water quality issues
      - Toxic substances can inhibit or disrupt bioprocesses (e.g., quaternary ammonium salts, zinc)
      - Alkalinity additions may be needed to support nitrification

7.47

- D&I considerations—Bioreactor configurations
  - There are numerous bioreactor configurations that have different biomass and flow regime characteristics to achieve similar or different treatment goals
  - For example, for many decentralized applications the following configurations are common

Smaller flows (e.g., 500 gal/d)

Larger flows (e.g., 50,000 gal/d)

- Flow-through completely mixed, suspended or attached growth bioreactors with extended aeration
  - Extended aeration (suspended growth)
  - Submerged media or trickling filters (attached growth)
- Flow-through pseudo plug flow or batch systems, with suspended growth and sequential zones of aeration and no aeration to help achieve nutrient removal
  - Oxidation ditch (suspended growth)
  - Sequencing batch reactor (suspended growth)

7.48

- Examples of bioreactor configurations that are often used in decentralized applications are summarized in Tables 7.5 and 7.6

Table 7.5 Features of aerobic treatment operations involving suspended growth processes[a]

Unit operation	Reactor type	Features
Extended aeration	Continuous flow-through tank	A tank or basin receives wastewater (typ. primary effluent) and is aerated for an extended period. Low organic loadings and long SRTs enable endogenous respiration and degradation of biomass solids which can minimize waste sludge production
Oxidation ditch	Continuous flow-through channel	An oval-shaped channel receives wastewater (typ. untreated) and mechanical aerators are used to achieve extended aeration of wastewater as it flows in the channel around the oval at around 1 ft/s
Sequencing batch reactor	Batch fill-and-draw tank	A reactor is filled with wastewater (typ. untreated) and then aerated, settled, decanted, and left idle before repeating the cycle. Secondary clarification is not needed. Use of two or more reactors can enable continuous operation

[a]*Note*: this table does not include all variations of aerobic biological treatment operations but only those that are most likely to be applicable to decentralized applications.

7.49

Table 7.6 Features of aerobic treatment operations involving attached growth processes[a]

Name	Reactor type	Features
Trickling filter	Flow-through packed bed of media	Wastewater is intermittently sprayed onto the top of a bed of crushed rock, slag, or gravel (2–4 in. diam.) and is aerated and treated as it percolates downward through the bed (typ. 6–9 ft depth). Beds containing plastic modules with high surface areas can also be used to enable taller beds and higher loading rates
Rotating biological contactor	Rotating disks in a flow-through tank	Disks are mounted on a central shaft and submerged (40–80 %) in a tank filled with wastewater. The disks are rotated slowly and biofilms grow on the disks and obtain oxygen as portions of the disks are exposed to air. Supplemental aeration of the wastewater can be used
Submerged media	Honeycomb in a flow-through tank	A honeycomb module is submerged within an aeration basin and biofilms grow on the media surfaces. Aerated wastewater is recirculated through the honeycomb often using an air-lift pumping unit

[a]*Note*: this table does not include all variations of aerobic biological treatment operations but only those that are most likely to be applicable to decentralized systems.

7.50

- Design and implementation options for bioreactor configurations
  - Pre-manufactured, off-the-shelf commercial units
    - Selected based on source features (flow and composition)
    - For larger residential systems and non-residential sources, some engineering may be required to select the correct unit (often referred to as "package plants")
  - Engineered and site-specific construction
    - For larger flows (e.g., large commercial or institutional sources, mixed-use developments, etc.)
    - Mix of pre-manufactured components plus site-fabricated tankage and equipment etc.
- In the following pages several considerations specific to common suspended and attached growth processes are given to illustrate how design and implementation can be accomplished

7.51

- D&I considerations for a SGB—Design parameters
  - Design of a suspended growth bioreactor
    - Design includes determining the aeration tank volume, clarifier sizing, solids generation and sludge wasting, DO requirements, and so forth
    - Design parameters are based on biological treatment processes and principles, but specific values are often based on experience with field operations
  - Tables 7.7 and 7.8 list key design parameters for two configurations often used for decentralized applications
    - Extended aeration bioreactors
    - Sequencing batch bioreactors
  - The following sections illustrate how the parameters shown in Table 7.7 can be used for design of an extended aeration unit such as illustrated in Fig. 7.13

7.52

Table 7.7 Design parameters for ATUs using suspended growth extended aeration (after USEPA 2002)

Parameter	Extended aeration
Pretreatment (if needed)	Septic tank or equivalent
Mixed liquor suspended solids (mg/L TSS)	2000–6000
F/M loading (lb-BOD/day to lb-MLVSS)	0.05–0.15
Hydraulic retention time (h)	24–120[a]
Solids retention time (days)	20–40
Mixing power input (hp per 1000 ft^3 of tank volume)	0.2–3.0
DO level (mg/L)	$>$2.0
Clarifier overflow rate (gal/day per ft^2 of surface area	200–400 (800 peak)
Clarifier solids loading (lb/day per ft^2 of surface area	30 (50 peak)
Bioreactor solids generated (lb-TSS/lb-BOD removed)	0.6–0.9
Frequency of solids removal (months)	3–6

[a]HRTs are normally not used for design and the high end of the range shown here is not typical o operating conditions. Typical values are in the range of 8–36 h.

7.53

Table 7.8 Design parameters for ATUs using suspended growth sequencing batch reactors (after USEPA 2002)

Parameter	Sequencing batch reactor
Pretreatment (if needed)	Septic tank or equivalent
Mixed liquor suspended solids (mg/L TSS)	2000–6500
F/M loading (lb-BOD/day to mg/L MLVSS)	0.04–0.20
Hydraulic retention time (h)	9–30
Total cycle times (h)	4–12
Solids retention time (days)	20–40
Decanter overflow rate (gal/min/ft^2)	$<$100
Frequency of biosolids removal (months)	As needed

7.54

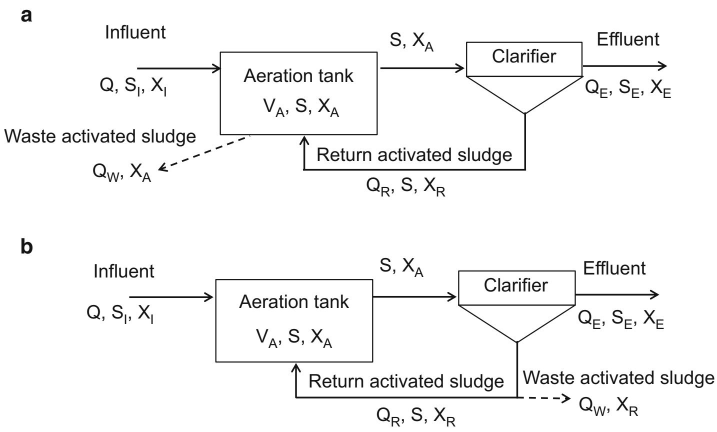


Fig. 7.13 Examples of a common suspended growth activated sludge process with solids separation by clarification and solids wasting from (**a**) the aeration tank or (**b**) from a recycle line

7.55

- D&I considerations for a SGB—Solids production
  - Net solids production (Y_N) is a design and operation parameter that takes into account the wastewater characteristics, the chosen SRT, and temperature
    - Example values of Y_N are provided in Table 7.9
  - In general, Y_N is higher with a lower quality influent and a shorter SRT under lower temperature conditions, e.g.:
    - Lower quality influent (e.g., without primary treatment compared to an influent after primary treatment)
      - $Y_N \approx 1.4\times$ to $1.8\times$ higher
    - Shorter solids retention time (e.g., 5 vs. 30 days)
      - $Y_N \approx 1.4\times$ to $1.6\times$ higher
    - Lower temperatures (e.g. 10 °C vs. 30 °C)
      - $Y_N \approx 1.2\times$ to $1.4\times$ higher

7.56

Table 7.9 Net solids production, Y_N (lb-VSS/lb-BOD removed)[a], in an aerobic treatment system as affected by wastewater characteristics, solids retention time and temperature

Solids retention time (day)	With primary treatment[b]			Without primary treatment[c]		
	10 °C	20 °C	30 °C	10 °C	20 °C	30 °C
1	0.87	0.72	0.61	1.18	1.05	0.94
5	0.71	0.62	0.52	1.03	0.92	0.85
15	0.55	0.48	0.42	0.83	0.76	0.71
30	0.44	0.39	0.34	0.70	0.65	0.60
90[d]	0.3	0.25	0.2	–	–	–

[a]Values in Table 7.9 are approximate since they were scaled off of Fig. 8.7 in Tchobanoglous et al. (2014)
[b]COD/BOD = 1.9–2.2; TSS/BOD = 0.5–0.7; Primary treatment at 60 % removal of TSS, with primary effluent inert TSS = 30 %.
[c]COD/BOD = 1.9–2.2; TSS/BOD = 0.9–1.1; Primary treatment inert TSS = 50 %.
[d]The Y_N values for the SRT of 90 days are very approximate since they are from the extreme of the data presented in Fig. 8.7.

7.57

- The net solids production that needs to be wasted can be calculated using Eq. 7.19
  - Example solids production values are given in Table 7.10

$$P_X = Y_N[Q(S_I - S_E)]F \tag{7.19}$$

Where:
P_X = waste activated sludge solids that needs to be wasted (lb/day)
Y_N = net solids production (lb-VSS/lb-BOD removed) as a function of SRT (see Table 7.9)
Q = average daily flow rate (gal/day) (*Note:* this could be the design daily flow rate or the actual daily flow rate being processed)
S_I = influent BOD concentration (mg/L)
S_E = effluent BOD concentration (mg/L)
F = 8.34×10^{-6} = conversion factor for mg/L to lb/gal

7.58

Table 7.10 Calculated net solids produced (lb-VSS/mon) during removal of 100 mg/L of BOD_5 from a daily flow rate of 1000 gal/day[a]

Solids retention time (day)	With primary treatment			Without primary treatment		
	10 °C	20 °C	30 °C	10 °C	20 °C	30 °C
5	17.8	15.5	13.0	25.8	23.0	21.3
15	13.8	12.0	10.5	20.8	19.0	17.8
30	11.0	9.8	8.5	17.5	16.3	15.0

[a]Net solids produced are calculated based on Eq. 7.19 using Y_N values from Table 7.9.

7.59

- D&I considerations for a SGB—Aeration tank sizing
  - The volume of the aeration tank can be determined based on a chosen design SRT using Eq. 7.20

$$V_A = \frac{(SRT)(Q_D)(S_I)(Y_N)}{X_A} \tag{7.20}$$

Where:
V_A = volume of the aeration tank (gal)
SRT = design solids retention time (days)
Q_D = design average daily flow rate (gal/day)
S_I = influent BOD or COD substrate concentration (mg/L)
Y_N = net yield coefficient (lb-VSS produced per lb-BOD or COD removed) as a function of SRT (–)
X_A = average mixed liquor volatile suspended solids (MLVSS) (mg/L)

Note: MLVSS ≈ a fraction of mixed liquor total suspended solids

7.60

- With V_A determined, the HRT is given by Eqs. 7.14 or 7.21

$$HRT = \frac{V_A}{Q_D} \tag{7.14}$$

$$HRT = \frac{(SRT)(S_I)(Y_N)}{(X_A)} \tag{7.21}$$

Where:
HRT = hydraulic retention time (day)
V_A = aeration tank volume (gal)
Q_D = design daily flow rate (gal/day)
SRT = solids retention time (day)
S_I = influent BOD or COD (mg/L)
Y_N = net solids production rate (lb-VSS/lb-BOD or lb-COD removed)
X_A = average MLVSS concentration (mg-VSS/L)

7.61

- D&I considerations for a SGB—F/M ratio
  - The food to microorganism ratio (F/M) has been used as a design and operational parameter for activated sludge systems and is calculated using Eq. 7.22

$$F/M = \frac{Q_D S_I}{X_A V_A} \tag{7.22}$$

Where:
F/M = food to microorganism ratio (lb-BOD or -COD/day per lb-MLVSS)
V_A = aeration tank volume (gal)
Q_D = design daily flow (gal/day) (e.g., $Q_A \times PF$)
S_I = influent substrate concentration—BOD or COD (mg/L)
X_A = average mixed liquor volatile suspended solids (MLVSS) (mg/L)

7.62

- D&I considerations—Operating parameter interactions
  - Parameters are inter-related and can't be set independently
    - Equations 7.21 and 7.23 reveal the relationships for SRT, Y_N, MLVSS, HRT and F/M

$$\mathrm{HRT} = \frac{(\mathrm{SRT})(\mathrm{S_I})(\mathrm{Y_N})}{(\mathrm{X_A})} \tag{7.21}$$

$$\mathrm{F/M} = \frac{(\mathrm{Q_D})(\mathrm{S_I})}{(\mathrm{X_A})(\mathrm{V_A})} = \frac{(\mathrm{Q_D})(\mathrm{S_I})}{(\mathrm{X_A})(\mathrm{HRT})(\mathrm{Q_D})} = \frac{(\mathrm{S_I})}{(\mathrm{X_A})(\mathrm{HRT})} \tag{7.23}$$

7.63

- Table 7.11 shows calculation results using Eqs. 7.21 and 7.23 for treatment of primary effluent at 20 °C and reveals:
  - As the SRT increases, Y_N decreases (and vice versa)
  - For the same SRT, as the X_A increases the HRT decreases (and vice versa) and F/M stays the same

Table 7.11 Example parameter values for extended aeration under varied conditions

Extended aeration system treating $S_I = 250$ mg/L BOD	Target X_A in aeration tank (mgVSS/L)	Selected SRT (day)	Estimated Y_N[a] (lb-VSS/lb-BOD)	Resulting HRT (day)	Resulting F/M lb-BOD/day per lb-VSS
Condition A	4000	15	0.48	0.45	0.139
Condition B	6000	15	0.48	0.30	0.139
Condition C	4000	30	0.39	0.73	0.085
Condition D	6000	30	0.39	0.49	0.085
Condition E	4000	180	0.10[b]	1.125	0.056

Note: *SI* influent BOD, X_A mixed liquor volatile suspended solids, *SRT* solids retention time, Y_N net solids production, *HRT* hydraulic retention time, *F/M* food-to-microorganism ratio.
[a]Estimated Y_N values are based on data presented in Table 7.9 for aerobic treatment of primary effluent at 20 °C
[b]This Y_N is roughly approximated based on the data presented in Table 7.9 and shown in Fig. 8.7 of Tchobanoglous et al. (2014).

7.64

- D&I considerations for a SGB—Solids recycling
  - Activated sludge solids must be retained in the aeration tank
    - A mass balance on the aeration tank with X_I assumed negligible is given by Eq. 7.24, which can be rearranged as Eq. 7.25

$$Q_R(X_R) = (Q + Q_R)X_A \tag{7.24}$$

$$R \approx \frac{Q_R}{Q} \approx \frac{X_A}{X_R - X_A} \tag{7.25}$$

Where:
Q = influent daily flow rate (gal/day)
Q_R = recycle flow rate from a clarifier back to the aeration tank (gal/day)
X_I = influent VSS or TSS; $X_I \ll X_A$
X_R = VSS or TSS in the recycle line back to the aeration tank (mg/L)
X_A = VSS or TSS in the aeration tank (mg/L)
R = recycle ratio (–) assuming influent VSS is ≪ aeration tank VSS
= 0.5 to 1.5 (typ.) for extended aeration

7.65

- D&I consideration for a SGB—Solids wasting
  - Wasting of the activated sludge solids produced is required to maintain a target SRT and MLVSS level, and a healthy F/M ratio
    - A definition schematic for wasting from a suspended growth activated sludge process is shown in Fig. 7.13
    - As shown in Fig. 7.13, wasting can occur in two ways
      - By removing mixed liquor from the aeration tank (Fig. 7.13a) (common for small systems)
      - By removing sludge from the return activated sludge line from a secondary clarifier (Fig. 7.13b)
    - The daily waste sludge flow rate can be computed for either wasting option using Eqs. 7.26, 7.27 and 7.28

7.66

- Wasting from the aeration tank
  - SRT is defined by Eq. 7.15

$$SRT = \frac{V_A X_A}{Q_W X_W + Q_E X_E} \tag{7.15}$$

  - Equation 7.15 can be rearranged and with $X_W = X_A$ and $X_E \approx 0$, Q_W is given by Eq. 7.26

$$Q_W = \frac{V_A}{SRT} \tag{7.26}$$

Where,
SRT = solids retention time (days)
Q_W = flow rate for solids wasting (gal/day)
Q_E = effluent flow rate (gal/day)
V_A = volume of the aeration tank (gal)
X_A = concentration of MLVSS in the aeration tank (mg VSS/L)
X_W = concentration of MLVSS in the waste solids flow (mg VSS/L)
X_E = concentration of MLVSS in the effluent (mg VSS/L)

7.67

- Wasting from the return line from a clarifier
  - A mass balance around the aeration tank in Fig. 7.13b where S_I is negligible is given by Eq. 7.24
  - Equation 7.24 can be rearranged as Eq. 7.27 and substituting Eq. 7.27 into Eq. 7.15 and rearranging with $X_R = X_W$ yields Eq. 7.28

$$X_A = \left(\frac{R}{1+R}\right) X_R \tag{7.27}$$

$$Q_W = \frac{\left(\frac{R}{1+R}\right) V_A}{SRT} \tag{7.28}$$

Where,
X_A = concentration of VSS in the aeration tank (mg/L)
R = recycle ratio, Q_R/Q, (–) (e.g., 0.5–1.5)
X_R = concentration of VSS in the return line (mg/L)
Q_W = influent flow rate (gal/day)
SRT = solids retention time (day)
V_A = volume of the aeration tank (gal)

7.68

- How frequently must solids be wasted?
  - This is an important question in the context of aerobic treatment used in a decentralized setting
    - If you don't waste excess solids, the SRT and MLVSS increase and the F/M ratio can decline
    - As a result, solids settleability deteriorates and clarifiers can be overloaded so treatment performance declines
  - In larger high-rate systems, wasting might be needed daily
  - In smaller low-rate systems, including packaged ATUs, wasting is normally done intermittently, e.g., every 3–6 months, by pumping a portion of the mixed liquor out of the aeration tank
    - This will result in the MLVSS varying around an average value over the duration of an average SRT
  - It is noted that wasting needs can be estimated but they are often determined by operational monitoring and control (e.g., MLVSS level, solids character and settleabilty, effluent quality)

7.69

- Example of intermittent solids wasting from a low-rate extended aeration treatment unit
  - To treat $Q = 1000$ gal/day with a BOD_5 of 150 mg/L to yield an effluent $BOD_5 = 30$ mg/L assuming:
    - Target SRT = 180 day, average MLVSS is 3000 mg-VSS/L
    - Primary treatment and the temperature = 20°C with $Y_N = 0.10$ lb-VSS/lb-BOD_5 removed
  - According to Eq. 7.20, the aeration tank volume, $V_A = 900$ gal
  - For daily wasting to maintain the SRT and MLVSS relatively constant, Eq. 7.26 yields $Q_W = 5$ gal/day
  - According to Eq. 7.19, removal of 120 mg/L of BOD_5 produces 18 lb of VSS during 180 days of operation
  - This 18 lb could raise the concentration of VSS in the 900-gal aeration tank by 2400 mg-VSS/L during 180 days of operation

7.70

- If solids are wasted from the aeration tank every 180 days (same as the target SRT), about 57 % of the aeration tank volume would need to be removed to keep the MLVSS in a range around the average of 3000 mg-VSS/L
  - * The MLVSS would range from about 4200 down to 1800 mg-VSS/L) as shown in Fig. 7.14

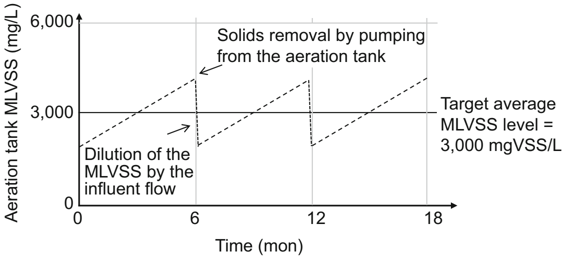


Fig. 7.14 Illustration of MLVSS varying around an average value of 3000 mg/L and SRT of 180 days due to intermittent wasting of solids from the aeration tank only every 180 days.

Note: the aeration tank volume is 900 gal and to remove the 18 lb of excess solids about 514 gal (57 % of V^A) of mixed liquor at 4200 mg-VSS/L is pumped out every 180 days.

7.71

- D&I considerations for a SGB—Solids separation
  - Solids produced in the aeration tank are separated from the liquid, normally by settling in a chamber or secondary clarifier
    - Under quiescent conditions, cells in a state of endogenous decay can form polymers that aid flocculation and settling under gravity
    - Nonvolatile suspended solids can get caught up in the flocs that form and can be removed by settling
    - With continued settling the flocs undergo compaction
  - Settleability (including compactibility) are affected by the SRT and F/M ratio
    - At low SRTs, activated sludge solids can be populated by filamentous organisms that inhibit flocculation and lead to poor settleability
    - At very long SRTs with low F/M, flocculation can be inhibited and pinpoint flocs can form that do not settle very well

7.72

- Sludge settleability as assessed using the sludge volume index
  - ○ The sludge volume index (SVI) is a simple test to assess solids settleability but not theoretically based
    - * The SVI test procedure is simple and easily done
      - – After 30 min. the volume of settled solids in a 1-L conical cylinder can be used to calculate the SVI (Eq. 7.29)

$$\mathrm{SVI} = \frac{\text{settled solids volume } (\mathrm{mL/L})(1000\,\mathrm{mg/g})}{\text{suspended solids } (\mathrm{mg/L})} = \frac{\mathrm{mL}}{\mathrm{g}} \quad (7.29)$$

  - ○ Activated sludge with SVI values in the range of 50–150 mL/g is typically considered to have good settling characteristics
  - ○ Sludge settleability varies as a function of operating conditions such as the SRT or F/M
    - * The effects on settleability can be reflected in the SVI as shown in Fig. 7.15

7.73

  - ○ If it assumed that conditions in a clarifier are the same as in the lab SVI test, Eq. 7.30 can be used to estimate X_R (for X_R <10,000 mg/L)

$$X_R \approx \frac{(10^3\,\mathrm{mL/L})(10^3\,\mathrm{mg/g})}{\mathrm{SVI}\ \ \mathrm{mL/g}} = \frac{\mathrm{mg}}{\mathrm{L}} \quad (7.30)$$

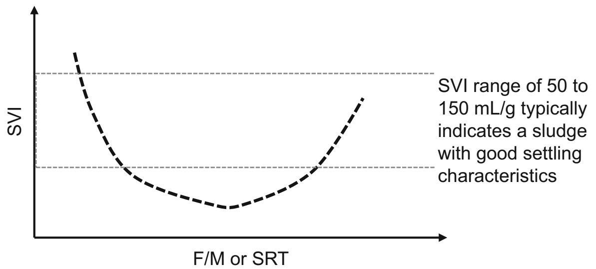


Fig. 7.15 Illustration of sludge settleabilty as measured by the SVI revealing the effects of F/M or SRT

7.74

- Sludge settleability assessed by zone settling rates
  - Assessing sludge settleability using zone settling rates is more technically sound than the SVI but this requires more complicated testing
    - * If the zone settling rate (V_i) is measured during a lab test it can be used to determine the overflow rate for design purposes (Eqs. 7.31 and 7.32)

$$OR_D = \frac{(V_i)(179.5)}{SF} \tag{7.31}$$

$$V_i = V_{max}\exp\left(\left(-K \times 10^{-6}\right)X_A\right) \tag{7.32}$$

Where:
OR_D = design surface overflow rate (gal/ft^2/day)
V_i = estimated settling velocity of the solids interface (ft/h)
179.5 = conversion factor from ft/h to gal/ft^2-day [(24 h/day)(748 gal/ft^3)]
SF = safety factor for nonideal conditions in clarifiers (typ. 1.75–2.5)
V_{max} = maximum settling velocity of the interface (typ. 23 ft/h)
K = constant (e.g., around 600 L/mg)
X_A = average mixed liquor suspended solids (mg/L)

7.75

- Design of secondary clarifiers
  - Clarifiers need to be designed to achieve suitable surface overflow rates (OR) and solids loading rates (SLR) to avoid carryover of solids into the effluent
  - A secondary clarifier is illustrated in Fig. 7.16

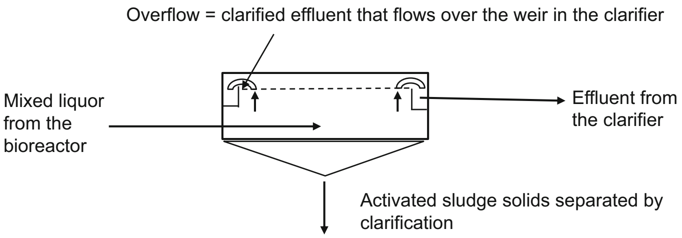


Fig. 7.16 Cross-section of a secondary clarifier used for activated sludge solids separation and return

7.76

- The OR and SLR are calculated as shown in Eqs. 7.33 and 7.34

$$A_c = \frac{Q_D}{OR_D} \tag{7.33}$$

$$SLR = \frac{(1+R)(Q_D)(X_A)(F)}{A_c} \tag{7.34}$$

Where:
A_c = surface area of the clarifier (ft^2)
Q_D = design flow rate into the clarifier (gal/day)
R = recycle ratio for activated sludge return (–)
OR_D = design surface overflow rate (gal/day/ft^2) (Eq. 7.31)
SLR = solids loading rate (lb-TSS/ft^2/h)
X_A = average mixed liquor suspended solids (mg/L)
$F = 0.35 \times 10^{-6}$ = conversion factor for mg/L to lb/gal and hour to days

7.77

- Experience-based values for OR and SLR
  * Typical OR (gal/day/ft^2) to clarify effluents from different aerobic unit designs are presented in Table 7.13
  * If SLR values get too high, solids will wash out of the clarifier and the effluent quality will deteriorate

Table 7.13 Experienced-based values for clarifier OR and SLR values

Secondary clarifiers receiving effluents	Typical depth (ft)	OR (gal/ft^2/day)		SLR (lb-TSS/ft^2/h)	
		Average	Peak	Average	Peak
Extended aeration unit effluent	12–20	200–400	600–800	0.2–1.0	1.4
Trickling filter effluent	10–18	400–600	1000–1200	0.6–1.0	1.6
Secondary effluent	10–18	400–800	1000–1200	0.8–1.2	2.0
Nitrified effluent	10–18	400–600	800–1000	0.6–1.0	1.6

Source: adapted from Table 7.14, Crites and Tchobanoglous 1998.

7.78

- D&I considerations for SGB—Oxygen requirements
  - For advanced secondary treatment, oxygen is required for organic matter degradation and nitrification as given by Eq. 7.35

$$O_2R = \left[\frac{Q(S_I - S_E)}{f} - 1.42(Q_W X_W) + 4.57Q(N_I - N_E)\right]F \quad (7.35)$$

Where:
O_2R = total oxygen requirement (lb-O_2/day)
Q = average daily flow rate (gal/day)
S_I = influent BOD_5 concentration (mg/L)
S_E = effluent BOD_5 concentration (mg/L)
f = ratio of BOD_5 to ultimate BOD (–) (e.g., 0.70)
Q_W = waste flow rate (gal/day)
X_W = concentration of solids in the waste activated sludge (mgVSS/L)
N_I = influent TKN concentration (mg-N/L)
N_E = effluent TKN concentration (mg-N/L)
$F = 8.34 \times 10^{-6}$ = conversion factor for mg/L to lb/gal

7.79

- Experience-based oxygen requirements for several activated sludge processes are highlighted in Table 7.14

Table 7.14 Typical oxygen requirements to support activated sludge biological treatment (Crites and Tchobanoglous 1998)

Activated sludge process	SRT (day)	Oxygen requirements
High rate (non-nitrifying)	0.75–2	0.6–0.8 lb-O_2/lb-COD
Conventional (non-nitrifying)	3–8	0.7–0.9 lb-O_2/lb-COD
Low rate (nitrifying)	>15	0.8–1.1 lb-O_2/lb-COD plus 4.6–4.7 lb-O_2/lb-nitrate formed

7.80

- Methods of adding DO to wastewater
  - Mechanical aeration devices that mix air into wastewater
    - O_2 transfer into the wastewater depends on the aerator type and size
      - Transfer rates can range from 1.5 to 2.5 lb-O_2 per hp-h.
  - Diffusers which inject air into wastewater being aerated
    - O_2 transfer depends on the diffuser type (fine bubble vs. coarse bubble) and activated sludge process
      - Oxygen transfer rates typically can range from 4 to 16 % of the O_2 injected
      - High purity oxygen can be used instead of air to increase the DO in the wastewater
  - Both methods require power for motors, pumps, mixers, and/or compressors

7.81

- D&I considerations for a SGB—Energy requirements
  - Aerobic biological treatment operations normally require power
  - One major area of power consumption is for mixing
    - Mixing needs to keep the biomass suspended in an aeration tank, basin, channel, etc.
    - Mixing needs to bring the wastewater into contact with attached growth biomass
  - Mixing can be achieved during aeration by mechanical aerators or diffusers
  - Example requirements for mixing are illustrated in Table 7.15

Table 7.15 Input requirements for mixing during activated sludge biological treatment

Mixing technology	Input required for mixing per 1000 ft^3 of aeration tank
Mechanical aerators	0.75–1.5 hp
Diffusers	20–30 ft^3/min for diffusers along a sidewall
	10–15 ft^3/min for diffusers in a grid on the bottom

7.82

- D&I considerations for an AGB—Design parameters
  - Design of an ATU that utilizes an attached growth bioreactor
    - Design includes determining the bioreactor volume, biomass solids retention and return, sludge wasting, and so forth
    - Design parameters are based on biological treatment processes and principles, but specific values are often based on experience with field operations
  - Tables 7.16 and 7.17 list key design parameters for two configurations often used for decentralized applications
    - Trickling filters
    - Rotating biological contactors
  - The following sections illustrate how the design parameters can be used for design of a trickling filter

7.83

Table 7.16 Design parameters for ATUs using attached growth trickling filters and rotating biological contactors (after USEPA 2002)

Parameter	Trickling filter	Rotating biological contactor
Pretreatment (if needed)	Septic tank or equiv.	Septic tank or equiv.
Surface hydraulic loading (gal/day/ft^2)	10–25	N/A
Organic loading (lb-BOD/day per ft^2 of filter surface area (Trickling Filter) or 1000 ft^2 of disk surface area (RBC))	5–20 (3 to 10 to nitrify)	2.5 (6.4 to nitrify)
Clarifier overflow rate (gal/day/ft^2)	600–800–average Q	600–800–average Q
	1000–1200—peak Q	1000–1200—peak Q
Clarifier solids loading rate (lb-TSS/day per ft^2)	0.8 to 1.2—average Q	0.8 to 1.2—average Q
	2.0—peak Q	2.0—peak Q
Recirculation	Optional	Optional
Bioreactor solids generated (lb-TSS per lb-BOD removed)	0.6–1.1	0.6–1.1

7.84

Table 7.17 Design parameters for ATUs using attached growth trickling filters (Crites and Tchobanoglous 1998)

Parameter	Rock or slag		Plastic
	Low rate	High rate	High rate
Filter medium diam. (in.)	1–5	1–5	24 × 24 × 48
Void space (%)	40–55	40–55	92–97
Specific surface (ft^2/ft^3)	12–30	12–30	24–60
Specific weight (lb/ft^3)	50–90	50–90	2–6
Hydraulic loading rate ($gal/day/ft^2$)	23–92	230–918	344–2066
Organic loading rate (lb-BOD_5/day per 1000 ft^3):			
Organic matter removal	5–25	30–80	50–200
Nitrification	5–10	5–15	10–25
Depth (ft)	6–8	6–8	10–40
Recirculation ratio (–)	0	1–2	1–2
BOD_5 removal efficiency	80–90	65–90	65–90
Sloughing of solids	Intermittent	Continuous	Continuous
Filter flies	Many	Few	Few or none

Source: Table 7.15 in Crites and Tchobanoglous (1998).

7.85

- D&I considerations for an AGB—Bioreactor sizing
  - A variety of attached growth system designs are used including trickling filters and rotating biological contactors
  - Sizing of the bioreactor for attached growth systems is more empirical than sizing of suspended growth aeration designs
    - For example, for low-rate trickling filters using rock media (Fig. 7.17) typical design parameters are:
      - Media diameter = 1 to 5 in.
      - Depth of filter medium (D_F) = 6 to 8 ft
      - Hydraulic loading rate (HLR) = 23 to 92 $gal/day/ft^2$
      - Recirculation ratio = 0 to 1
      - Organic loading rate (OLR) = 5 to 25 lb-BOD_5/day per 1000 ft^3 of filter volume for BOD removal and 5 to 10 lb-BOD_5/day per 1000 ft^3 for nitrification

7.86

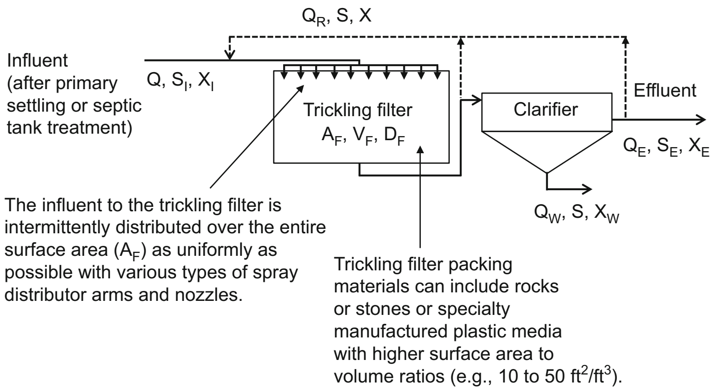


Fig. 7.17 Illustration of a trickling filter used for aerobic biological treatment

7.87

- Simplified sizing equations for a trickling filter are given in Eqs. 7.36, 7.37 and 7.38

$$A_F = \frac{Q_D}{HLR} \tag{7.36}$$

$$V_F = \frac{(Q_D)(BOD_5)(F)}{OLR} \tag{7.37}$$

$$D_F = \frac{V_F}{A_F} \tag{7.38}$$

Where:
A_F = trickling filter surface area (ft^2)
V_F = volume of the trickling filter (ft^3)
D_F = depth of the filter medium (ft)
Q_D = design daily flow (gal/day)
BOD_5 = influent concentration of BOD_5 (mg/L)
HLR = hydraulic loading rate applied to the filter surface area (gal/day/ft^2)
OLR = organic loading rate applied to the filter volume (lb-BOD_5/day per ft^3)
F = 8.34×10^{-6} = conversion factor for mg/L to lb/gal

7.88

- D&I considerations for an AGB—Other considerations
  - Other D&I considerations are similar to those that apply to aerobic treatment using suspended growth processes, including:
    - Biomass generation and removal
    - Solids wasting and management
    - Oxygen and energy requirements
  - However, there are differences in requirements and parameter values
    - These are reflected in the respective design parameters shown in the tables for attached growth bioreactors (Tables 7.16 and 7.17) compared to suspended growth bioreactors (Tables 7.7 and 7.8)

7.89

- D&I considerations—Installation and startup
  - Proper installation is critical to achieving the expected performance of aerobic biological treatment systems
    - This is similar to, but even more important than, that for septic tanks (refer to Chap. 6), including:
      - The location of the aerobic treatment unit must enable access for construction equipment and service vehicles
      - Tankage and piping needs to be watertight and structurally sound
      - Materials of construction need to be corrosion resistant
      - Power is required for pumps, aerators, controls, etc. so power sources need to be robust and reliable
  - Proper startup is also critical to performance
    - Seeding of the aeration tank with activated sludge from another biological treatment unit can be helpful

7.90

- D&I considerations—Appurtenances and integral treatment processes
  - Basic appurtenances can be important, if not critical, to successful aerobic biological treatment, including:
    - Air compressors, tubing, and diffusers
    - Mixers, pumps and controls
    - Sensors, data recorders, alarms
  - There can also be integral treatment processes, for example:
    - ATUs can have an integral primary sedimentation chamber to which raw wastewater flows to avoid the need for a separate settling basin or septic tank prior to the bioreactor
    - Some commercial ATUs also have integral disinfection unit operations, which can enable surface discharge and reuse of effluent

7.91

- D&I considerations—Operation & maintenance
  - Proper O&M is critical to achieving performance capabilities
  - Inspections should be done frequently (e.g., monthly)
    - Conditions important to treatment should be observed
      - Power and controls need to be online and functional
      - Pumps and valves need to be properly functioning
      - Aeration equipment needs to be operational
      - Distributors and nozzles for attached growth systems such as trickling filters need to be clear and functioning
      - DO levels must be adequate (typ. >2 mg/L)
      - Mixed liquor should be settleable (e.g., SV $=$ 50–150 mg/L)
  - Effluent quality monitoring may be needed for process control and compliance
  - Excess solids should be removed and wasted as needed

7.92

- Certain problems leading to O&M needs can occur when aerobic treatment units are used in decentralized applications
  - Suspended growth processes
    - Problems with maintaining adequate aeration and mixing and good settling characteristics for the activated sludge solids
  - Attached growth processes
    - Problems with pumping and distribution of wastewater over the media that biomass is attached to without causing sloughing
- In general, aerobic treatment operations serving larger flows (e.g., clusters of houses or businesses) tend to perform better than those serving individual houses because:
  - The influent flow and composition tends to be relatively continuous and consistent over time
  - Based on size and economies of scale, the necessary routine O&M needs can be assured

7.93

7-5. Summary

- Aerobic biological treatment processes are involved in many unit operations used in decentralized systems
- Aerobic treatment units can be designed where the biomass is suspended or attached to surfaces in different types of bioreactors
  - Extended aeration systems with long SRTs are most often used to minimize excess solids production and enable infrequent wasting of solids
- Aerobic treatment units can produce advanced secondary effluents that are very low in BOD_5, TSS, and NH_4^+
- Aerobic treatment units require a reasonably continuous and consistent influent, conditions favorable to aerobic bioprocesses, and reliable O&M to ensure performance is sustainably achieved

7.94

7-6. Example Problems

- 7EP-1. Sizing an aeration tank and estimating the solids wasting rate
  - Given information
    - Extended aeration is being used to treat the wastewater from a residential and commercial complex
    - Influent characteristics: Design $Q_D = 52{,}840$ gal/day; primary effluent $BOD_5 = 250$ mg/L and TSS = 50 mg/L; average temperature = 20 °C
    - Effluent requirements: BOD_5 and TSS <20 mg/L
    - Aeration tank design parameters are chosen to be SRT = 15 days with MLVSS = 4000 mg/L
  - Determine
    - The volume of the aeration tank (gal)
    - The hydraulic retention time (HRT) (days)
    - The rate of wasting excess solids from the aeration tank (gal/day)

- Solution
  - Determine the volume of the aeration unit based on the given design flow rate and operating conditions using Eq. 7.19
    - For primary effluent with a temperature = 20 °C, $Y_N = 0.39$ lb VSS produced per lb BOD removed (see Table 7.9)

$$V_A = \frac{(SRT)(Q_D)(S_I)(Y_N)}{X_A} \tag{7.20}$$

$$V_A = \frac{(15\,\text{days})(52{,}840\,\text{gal/day})(250\,\text{mg/L})(0.39\,\text{lb-VSS/lb-BOD})}{4000\,\text{mg-VSS/L}}$$

$$V_A = 19{,}320\,\text{gal}$$

Note: you could provide the required V_A in two 9660 gal tanks operated in parallel.

- Determine the hydraulic retention time using Eq. 7.14

$$\mathrm{HRT} = \frac{V_A}{Q_D} \tag{7.14}$$

$$\mathrm{HRT} = \frac{19{,}320\,\mathrm{gal}}{52{,}840\,\mathrm{gal/day}} = 0.36\,\mathrm{days} = 8.8\,\mathrm{h}.$$

- Determine the rate of solids wasting from the aeration tank using Eq. 7.26

$$Q_W = \frac{V_A}{\mathrm{SRT}} \tag{7.26}$$

$$Q_W = \frac{19{,}320\ \mathrm{gal}}{15\,\mathrm{days}} = 1288\frac{\mathrm{gal}}{\mathrm{day}}$$

7.97

- You can also estimate the solids wasting rate using Eq. 7.18
  * Equation 7.18 uses the net rate of cell growth and the target SRT and ignores the contribution due to TSS

$$\frac{r'_g}{X_A} \approx \frac{Q_W X_W}{X_A V_A} \approx \frac{1}{\mathrm{SRT}} \tag{7.18}$$

$$r'_g \approx \frac{X_A}{\mathrm{SRT}} \approx \frac{4000\,\mathrm{mg/L}}{15\ \mathrm{days}} = 267\,\mathrm{mg/L/day}$$

$$Q_W = \frac{r'_g V_A}{X_W} = \frac{(267\,\mathrm{mg/L/d})(19{,}320\ \mathrm{gal})}{4000\ \mathrm{mg/L}}$$

$$Q_W = 1289\,\mathrm{gal/day}$$

7.98

- 7EP-2. Design of an extended aeration unit
  - Given information
    - Alpine Meadows is a condominium complex (32 units) that needs a new decentralized wastewater system and is considering an extended aeration unit
    - Based on monitoring over the past 2 years, $Q_D = 5000$ gal/day and after septic tank treatment, the $BOD_5 = 150$ mg/L and TSS = 100 mg/L
    - The average daily temperature is about 20 °C
  - Determine
    - Choose reasonable values for the SRT and X_A and determine the size of the aeration tank (gal) to handle the design flow
    - Check the F/M ratio and the HRT to see if they are reasonable for an extended aeration process

7.99

- Solution
  - Determine the volume of the aeration tank based on the given design flow rate and operating conditions using Eq. 7.20
    * Assuming a SRT = 15 days with average $X_A = 3000$ mg-VSS/L, for primary effluent with a temperature = 20 °C, $Y_N = 0.48$ lb-VSS per lb-BOD removed (see Table 7.9)

$$V_A = \frac{(SRT)(Q_D)(S_I)(Y_N)}{X_A} \tag{7.20}$$

$$V_A = \frac{(15\,\text{days})(5000\,\text{gal/day})(150\,\text{mg/L})(0.48\,\text{lb-VSS/lb-BOD})}{3000\,\text{mg-VSS/L}}$$

$$V_A = 1800\,\text{gal}$$

7.100

- Check the F/M ratio and HRT

$$F/M = \frac{Q_D S_I}{X_A V_A} \tag{7.22}$$

$$F/M = \frac{(5000\,\text{gal}/\text{day})(150\,\text{mg BOD}/\text{L})}{(3000\,\text{mg VSS}/\text{L})(1800\,\text{gal})}$$

$$F/M = 0.139\ \text{day}^{-1}$$

$$HRT = \frac{V_A}{Q_D} \tag{7.14}$$

$$HRT = \frac{1800\ \text{gal}}{(5000\ \text{gal}/\text{d})} = 0.36\ \text{days} = 8.6\ \text{h}.$$

  - * The F/M ratio is high, but within the range of experienced-based values used for extended aeration (e.g., 0.05–0.15 day^{-1}).
  - * The HRT is relatively short but still in the range of experienced-based values for extended aeration (e.g., 8–36 h).

7.101

- 7EP-3. Design of an aeration basin
  - Given information
    - Western Terrace (WT) is a small commercial development outside Denver, CO that is planning to upgrade an existing wastewater facility and considering conversion to an aerobic treatment system
    - There is an existing basin that is 10 ft wide by 30 ft long with a total depth of 10 ft that could be used as an extended aeration basin
    - Based on monitoring over the past 2 years, $Q_D = 20{,}000$ gal/day and after primary treatment, the $BOD_5 = 200$ mg/L
  - Determine
    - Determine if the HRT the existing basin could provide is sufficient for an extended aeration process with reasonable values for a SRT and X_A

7.102

- Solution
  - Volume of the basin available for use as aeration tank

$$V_A = L \times W \times H$$
$$V_A = 30\,\text{ft.} \times 10\,\text{ft.} \times 7\,\text{ft.} = 2100\,\text{ft}^3$$
$$V_A = \left(7.48\,\text{gal}/\text{ft}^3\right)\left(2100\,\text{ft}^3\right) = 15{,}708\,\text{gal}$$

Note: H = 7 ft. to allow 3 ft. as freeboard

  - Maximum hydraulic retention time provided by the basin

$$\text{HRT} = \frac{V_A}{Q_D} = \frac{15{,}708\ \text{gal}}{(20{,}000\,\text{gal}/\text{day})} \tag{7.14}$$
$$\text{HRT} = 0.78\ \text{days} = 18.8\ \text{h.}$$

  - HRT that could be used is within the range of experienced-based values for extended aeration (e.g., 8–36 h.)

7.103

- * The SRT can be estimated by trial and error using Eq. 7.21

$$\text{HRT} = \frac{(\text{SRT})(S_I)(Y_N)}{(X_A)} \tag{7.21}$$

Calculate HRT for a trial SRT $= 15\,\text{days}$ with $Y_N = 0.48\,\text{lb-VSS}/\text{lb-BOD}$

and $X_A = 2000\,\text{mg-VSS} = \text{L}$

$$\text{HRT} = \frac{(15\,\text{days})(150\,\text{mg}/\text{L})(0.48\,\text{lb-VSS}/\text{lb-BOD})}{(2000\,\text{mg-VSS}/\text{L})}$$

$$\text{HRT} = 0.54\,\text{d} = 13\,\text{h.}$$

  - So, choosing a SRT of 15 days and a target X_A of 2000 mg-VSS/L results in a required HRT of 13 h, which is lower than the maximum HRT available for the existing basin.

7.104

- 7EP-4. Design of an extended aeration bioreactor to remove organic matter and ammonia nitrogen
  - Given information
    - Influent to the aerobic unit will be primary effluent (Table 7EP.1) with a design flow $Q_D = 7500$ gal/day
    - Design parameters include: SRT = 90 days, $X_A = 3000$ mg-VSS/L, and $Y_N = 0.2$ lb-VSS per lb-BOD

Table 7EP.1 Composition of the influent to be treated in an extended aeration bioreactor

BOD_5 (mg/L)	COD (mg/L)	TSS (mg/L)	Org-N (mg-N/L)	NH_4-N (mg-N/L)	P (mg-P/L)	Alkalinity (mg/L)	Temp. (°C)
160	250	100	18	22	5	350	20

  - Determine
    - Design an extended aeration unit (w/ solids recycle) to produce a secondary effluent with nitrification

7.105

- Solution
  - Calculate the design loadings to the aeration tank
    - * Example calculation using BOD_5 from Table 7EP.1 with other calculated values presented in Table 7EP.2

$$\mathrm{BOD_5} = (160\,\mathrm{mg/L})(7500\,\mathrm{gal/day})(3.785\,\mathrm{L/gal})\left(1 \times 10^{-6}\,\mathrm{kg/mg}\right)$$

$$\mathrm{BOD_5} = 4.5\,\mathrm{kg/day} = 9.92\ \mathrm{lb/day}$$

Table 7EP.2 Design loadings in an influent to be treated in an extended aeration bioreactor

Stream	Units	BOD_5	COD	TSS	Org-N	NH_4-N	P	Alkalinity
Influent	mg/L	160	250	100	18	22	5	350
	kg/day	4.5	7.1	2.8	0.51	0.62	0.14	
	lb/day	9.9	15.6	6.2	1.1	1.4	0.3	
Effluent	mg/L	20	20			<1	1	

7.106

- Determine the volume of the aeration tank
  - Sizing for a SRT = 90 days

$$V_A = \frac{(SRT)(Q_D)(S_I)(Y_N)}{X_A} \tag{7.20}$$

$$V_A = \frac{(90\,day)(7500\,gal/day)(160\,mg/L)(0.2\,lb\text{-}VSS/lb\text{-}BOD)}{(3000\,mg{-}VSS/L)} = 7200\,gal$$

- Check the F/M using $BOD_5 = 160$ mg/L

$$F/M = \frac{(Q_D)(S_I)}{X_A V_A} \tag{7.22}$$

$$F/M = \frac{(7500\,gal/day)(160\,mg/L)}{(3000\,mg\text{-}VSS/L)(7200\,gal)} = 0.06\,day^{-1}$$

  - The F/M ratio of 0.06 is okay as it is within the range of experienced-based values used for extended aeration (e.g., 0.05–0.15).

7.107

- Determine the hydraulic retention time

$$HRT = \frac{V_A}{Q_D} = \frac{7200\,gal}{7500\,gal/day} = 0.96\,day = 23\,h. \tag{7.14}$$

  - HRT is within the range of experienced-based values for extended aeration (e.g., 8–36 h)
- Determine the recycle ratio
  - Assume the recycle line concentration = 8000 mg-VSS/L and the target $X_A = 3000$ mg-VSS/L

$$R = \frac{X_A}{X_R - X_A} = \frac{3000}{8000 - 3000} = 0.6 \tag{7.25}$$

    - The R ratio of 0.6 is okay as it is within the range of experienced-based values used for extended aeration (e.g., 0.5–1.5).

7.108

- Determine the solids production
  - Solids production can be estimated using Eq. 7.19

$$P_X = Y_N[Q(S_I - S_E)]F \tag{7.19}$$

$$P_X = 0.2\,\text{lb}/\text{lb}[7500\,\text{gal}/\text{day}(160 - 20\,\text{mg}/\text{L})]8.34 \times 10^{-6}$$

$$P_x = 1.75\ \ \text{lb-VSS}\ \ \text{per}\ \ \text{day}$$

  - Solids production can also be estimated using Eq. 7.18

$$\frac{r'_g}{X_A} \approx \frac{Q_W X_W}{X_A V_A} \approx \frac{1}{SRT} \tag{7.18}$$

$$r'_g \approx \frac{X_A}{SRT} \approx \frac{3000\,\text{mg}/\text{L}}{90\,\text{day}} = 33.3\,\text{mg}/\text{L}/\text{day}$$

$$P_X = (33.3\,\text{mg}/\text{L}/\text{day})(7200\,\text{gal})(3.785\,\text{L}/\text{gal})\left(2.206 \times 10^{-6}\,\text{lb}/\text{mg}\right)$$

$$P_X = 2.0\,\text{lb-VSS}/\text{day}$$

Note: the estimate made using Eq. 7.19 is less than that using Eq. 7.18 since the former accounts for the BOD in the clarifier effluent.

7.109

- Determine the wasting rate required
  - Determine Q_W from the aeration basin (Eq. 7.26)
    - Daily wasting from the aeration tank

$$Q_W = \frac{V_A}{SRT} \tag{7.26}$$

$$Q_W = \frac{7200\,\text{gal}}{90\,\text{day}} = 80\,\text{gal}/\text{day}$$

    - Intermittent wasting every 90 days from the aeration tank

      At 0.2 lb-VSS/ lb-BOD removed, 180 lb of VSS are produced over a 90 day period

      180 lb of VSS = 3000 mg/L in 7200 gal

      For wasting at 90 days, 4800 gal of mixed liquor at 4500 mg-VSS/L would be pumped out from the aeration tank (about 67 % of V_A)

      This wasting approach yields a volume that is less than the 80 gal/day times 90 days which is 7200 gal

7.110

* Determine Q_W for wasting from the clarifier return line

$$Q_W = \frac{\left(\frac{R}{1+R}\right)V_A}{SRT} \tag{7.28}$$

$$Q_W = \frac{\left(\frac{0.6}{1+0.6}\right)7200\,gal}{90\,day}$$

$$Q_W = 30\,gal/day$$

7.111

○ Determine the oxygen requirements[a]
   * Assume complete oxidation of organic matter and complete conversion of organic and ammonia N to NO_3^-
   * Assume no denitrification and thus no O_2 impact from denitrification[b]

$$O_2R = \left[\frac{Q(S_I - S_E)}{f} - 1.42(Q_W X_W) + 4.57Q(N_I - N_E)\right]F \tag{7.35}$$

$$O_2R = \left[\frac{7500(160-20)}{0.70} - 1.42[(80\ \ gal/day)(3000\,mgVSS/L)] + 4.57(7500\,gal/day)(22-1\,mgNH_4/L)\right]8.34\times10^{-6}$$

$$O_2R = [1{,}500{,}000 - 340{,}800 + 719{,}775]8.34\times10^{-6}$$

$$O_2R = 15.7\,lb - O_2/day$$

[a]NO_3^- is formed from OrgN + NH_4^+ and OrgN + NH_4^+ = TKN.
[b]If denitrification is accounted for, O_2 requirements could decrease by about 50 % due to use of NO_3 as an electron acceptor by microorganisms for organic matter degradation during denitrification.

7.112

- Determine the area of the clarifier required
  * Estimate the initial settling velocity using Eq. 7.32 assuming:

$$V_{max} = 23.0 \ \text{ft/h}$$

$$K = 500 \ \text{L/mg}$$

$$X_A = 3750 \ \text{mg/L MLSS (based on VSS} = 80\% \text{ of TSS)}$$

$$V_i = V_{max}\exp\left(\left(-K \times 10^{-6}\right)X_A\right) \tag{7.32}$$

$$V_i = 23.0\exp\left(\left(-500 \times 10^{-6}\right)3750\right)$$

$$V_i = 3.5 \ \text{ft/h}$$

7.113

- Determine the clarifier overflow rate
  * The OR_D is given by Eq. 7.31 assuming a safety factor of 2.0 to account for predictions versus field conditions

$$OR_D = \frac{(V_i)(179.5)}{SF} \tag{7.31}$$

$$OR_D = \frac{(3.5)(179.5)}{2.0}$$

$$OR_D = 314 \ \text{gal/day per ft}^2$$

  – OR_D is okay and within experienced-based values for extended aeration (e.g., 200–400 gal/day per ft^2)

7.114

- Determine the solids loading rate (SLR) to the clarifier using Eqs. 7.33 and 7.34

$$A_c = \frac{Q_D}{OR_D} = \frac{7500\,\text{gal/day}}{314\,\text{gal/day/ft}^2} = 24\,\text{ft}^2 \tag{7.33}$$

$$SLR = \frac{(1+R)(Q_D)(X_A)\left(0.35\times10^{-6}\right)}{(A_c)} \tag{7.34}$$

$$SLR = \frac{(1+0.6)(7500\,\text{gal/d})(3750\,\text{mg/L})\left(0.35\times10^{-6}\right)}{\left(24\,\text{ft}^2\right)}$$

$$SLR = 0.65\,\text{lb} - \text{TSS per ft}^2\,\text{per h}$$

  * The SLR is okay as it is within experienced-based values for extended aeration (e.g., 0.1–1.0)

7.115

- 7EP-5. Design of a trickling filter
  - Given information
    - Alpine Meadows is a condominium complex that has a total of 32 dwelling units (DU)
    - It is in need of building a new onsite wastewater treatment system and is considering an extended aeration bioreactor to produce an effluent for turf irrigation
    - Based on monitoring over the past 2 years, $Q_D = 10{,}000$ gal/day and after primary treatment, the $BOD_5 = 150$ mg/L and the $TSS = 100$ mg/L
  - Determine
    - Determine the surface area, volume, and media depth of a low-rate trickling filter required to handle the design daily flow

7.116

- Solution
  - Trickling filter area
    - Assume the hydraulic loading rate = 25 gal/day/ft^2 given experienced-based values for low-rate trickling filters (Tables 7.16 and 7.17)

$$A_F = \frac{Q_D}{HLR} \tag{7.36}$$

$$A_F = \frac{(10{,}000\,\text{gal}/\text{day})}{25\,\text{gal}/\text{day}/\text{ft}^2}$$

$$A_F = 400\,\text{ft}^2$$

7.117

  - Trickling filter volume
    - Assume the organic loading rate = 5 lb-BOD$_5$/day per 1000 ft^3 given experienced-based values for low-rate trickling filters (Tables 7.16 and 7.17)

$$V_F = \frac{(Q_D)(BOD_5)\left(8.34 \times 10^{-6}\right)}{OLR} \tag{7.37}$$

$$V_F = \frac{(10{,}000\ \ \text{gal}/\text{day})(150\ \ \text{mg}/\text{L})\left(8.34 \times 10^{-6}\right)}{\left(5\ \ \text{lb}/\text{d}/1000\,\text{ft}^3\right)}$$

$$V_F = 2502\,\text{ft}^3$$

  - Trickling filter media depth

$$D_F = \frac{V_F}{A_F} \tag{7.38}$$

$$D_F = \frac{2502\,\text{ft}^3}{400\,\text{ft}^2} = 6.25\,\text{ft}.$$

7.118

Chapter 8

Treatment Using Porous Media Biofilters

8-1. Scope

Porous media biofilters are designed to exploit attached growth biological processes to achieve advanced secondary treatment. Primary treated wastewater is intermittently dosed onto the surface of the biofilter and migrates by unsaturated flow through a depth of media upon which biofilms grow. This chapter describes the principles and processes associated with porous media biofilters and covers the design and implementation of intermittent and recirculating sand filters and packaged media biofilters. The design of pressurized delivery and distribution networks is also covered in detail in this chapter.

8-2. Key Concepts

- Porous media biofilters (PMBs) can be established using different types of media in different configurations.
  - At a basic level they all include intermittent application of primary treated wastewater over the surface of a biofilter media, which is typically housed in some type of container. During unsaturated flow through the biofilter aerobic conditions are maintained (typically by passive aeration) so that advanced secondary treatment can occur, principally by attached growth biological processes.
  - Intermittent application and uniform distribution of the wastewater applied to the biofilter surface is normally accomplished using networks of pressurized piping and perforated distribution laterals. Wastewater effluent pumps or dosing siphons can be used for intermittent application and network pressurization.

R.L. Siegrist, *Decentralized Water Reclamation Engineering*,
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- PMBs are similar to trickling filters in that they both rely on attached growth biological processes, but PMBs are different in that they typically have finer media with less media bed depth and lower hydraulic and organic loading rates.
- This chapter describes the process principles and design of several types of PMBs that are commonly used in decentralized systems.

■ Single pass sand filters (SPSFs) have a long history of use in treatment of wastewaters generated by houses, residential developments, commercial and institutional establishments, and small towns.

- The media used in most SPSF is screened and washed medium to coarse sand. Critical media suitability parameters are the effective size (ES), uniformity coefficient (UC), and %fines content. Typical specifications are: $ES = 0.25–1.0$ mm, $UC \leq 4$, and %fines ($D < 0.074$ mm) ≤ 3 wt%. Alternative configurations to a typical SPSF include use of beds of stratified sand layers or beds of reactive materials, which are selected and designed to achieve targeted removal of nutrients.
- The influent to the SPSF is typically septic tank effluent (STE) (or similar) and design hydraulic loading rates (HLR_D) are limited (e.g., 1 gal/day/ft^2) and application is done intermittently in very small doses (e.g., 12–24 times per day). This is by design so that the wastewater applied to the SPSF migrates through an unsaturated bed of medium to coarse sand by film flow over grain surfaces while maintaining air-filled porosity between them. Biofilms grow on the grain surfaces and function much like they do in an attached growth bioreactor (e.g., a trickling filter).
- SPSFs are typically established as buried sand filters with an open-bottom where the sand filter effluent (filtrate) is released into the subsurface underlying the bottom of the filter installation. These SPSFs typically have sand bed depths of 2 ft. The organic loading rate (OLR) to the filter is limited and the flow regime is unsaturated, both of which help ensure the SPSF remains aerobic through passive aeration.
- While less common, SPSF can also be established in a container or basin that is lined on the sides and bottom. For smaller filters there can also be a cover. These SPSFs are outfitted with an underdrain system and filtrate is collected and transported out of the filter.
- Loss in hydraulic capacity of a SPSF can occur due to clogging caused by filtration of suspended solids and accumulation of biological solids at the filter surface. Controlling the HLR_D and OLR can help mitigate this. If clogging becomes severe, the SPSF infiltrative surface can be raked or replaced. This is far easier to accomplish in a surface-accessible SPSF compared to a buried SPSF unit.

  - Treatment in a SPSF occurs through a combination of biological, physical and chemical processes. With typical sand media, physical filtration and biological transformation processes are dominant. However, SPSF can be established with more reactive mineral media to provide physico-chemical treatment of constituents like phosphorus.
  - A well-designed and operated SPSF, can produce a consistent advanced secondary effluent with BOD_5 and TSS <10 mg/L, NH_4^+ < 5 mg-N/L, total N removal $=$ 15–40 %, and 99–99.99 % removals of fecal coliform bacteria.

- Recirculating sand filters (RSFs) are similar to SPSFs in many respects. However, key differences in a RSFs include use of coarser granular media, more frequent dosing, and recirculation of filtrate so it passes through the granular media multiple times.
  - The media used in RSFs is typically screened and washed coarse sand and pea gravel. Critical media suitability parameters are the effective size, uniformity coefficient, and %fines content. Design values are: ES $=$ 1.0–5.0 mm, UC $<$ 2.5, and %fines (diameter $<$0.074 mm) $<$3 % by wt.
  - Due to the coarse media used, RSFs employ recirculation of filtrate. This is typically done by diverting a portion of the filtrate back into the chamber or tank used for dosing the filter. The wastewater in a recirculation tank is a blend of STE and RSFs filtrate and as such, its strength is less than straight STE. Typical recirculation rates are such that the RSFs filtrate passes through the filter 3–5 times and the filter surface receives 3–5 times the daily forward flow rate.
  - The influent to the RSFs is primary effluent (e.g., STE or similar). Design HLRs to a RSFs are higher than to a SPSF (e.g., 3–5 gal/day/ft^2) and application is done more frequently (e.g., 48–72 times per day$_)$. This is by design so that the influent applied to the RSFs migrates down through an unsaturated and aerobic bed of coarse sand or pea gravel by film flow over grain surfaces on which biofilms grow.
  - RSFs are normally free access (established with open and accessible sand surfaces) using concrete tanks, other basin units, or lined excavations in the landscape. For small RSFs the tank or basin can have a removable cover, but for larger RSFs the filter media surface is often covered with a 6- to 9-in. layer of fine gravel to protect the distribution piping. RSFs typically have sand bed depths that are 2–3 ft deep with an underdrain system through which filtrate is transported from the filter. The underdrain system is unsaturated and vented, and the OLRs to the filter are limited to ensure the RSFs remains aerobic through passive aeration.
  - Loss in hydraulic capacity of a RSFs can occur due to clogging caused by the same processes that occur in a SPSF. However

clogging is normally not a problem with a properly designed and operated RSFs. In the event it did occur, the RSFs infiltrative surface could be raked or otherwise maintained. But this is not easily done if the filter surface is covered with a layer of fine gravel.

- Treatment in a RSFs occurs through a combination of biological, physical and chemical processes. With the typical coarse sand or pea gravel size media, biological transformation processes are dominant. However, RSFs can be established with more reactive mineral media to provide physico-chemical treatment of constituents like phosphorus.
- A well-designed and operated RSFs can produce consistent advanced secondary effluent that has median BOD_5 and TSS concentrations of <10 mg/L. RSFs should nitrify the applied NH_4-N and can remove 40–60 % or more of the applied total N by denitrification (modified recirculation schemes can help achieve high N removal efficiencies). Fecal coliform bacteria removals can be 99–99.9 %.

■ Packaged media biofilters (PBFs) involve different types of manufactured media that have properties conducive for commercial packaging, shipment, and handling as well as treatment of wastewater. Examples of PBFs media include: (1) peat material (e.g., Anua Puraflo®), (2) open-cell foam cubes (e.g., Waterloo Biofilter®), (3) textile fiber sheets (e.g., Orenco Systems® Inc. Advantex®), and (4) styrene plastic beads (E-Z Treat Re-Circulating Synthetic Sand Filter). PBFs with these media are designed with intermittent, uniform application of STE (or similar) in numerous small doses to achieve film flow through a bed of the media. Bed depths are typically 2–5 ft and there is an underdrain system to collect the filtrate and enable recirculation or discharge.

- PBFs are manufactured by several companies and available in self-contained modules with removable covers or access lids to enable easy access to the distribution piping and biofilter media. The modules have a specific treatment capacity based on a HLR_D and OLR (e.g., 150 gal/day of domestic STE per module). Multiple modules can be combined in parallel to provide the needed treatment capacity for a particular project or in series as a second stage (e.g., to accomplish nitrification).
- Peat has high void volume and surface area, high moisture retention and surface reactivity, and can support growth of diverse biomass. As a natural material, peat is degradable and media replacement could be needed every 8–15 years. Peat PBFs are normally operated in a single-pass, down-flow mode (like an SPSF) with HLR_D of 5–6 gal/day/ft^2 and OLRs of about 0.014 lb BOD/day/ft^2.
- Open cell polyurethane formed into cubes (2–3 in. on a side) was conceived to take advantage of the lightweight, high void volume, and non-degradable properties of the foam. Foam PBFs can be operated

in single-pass or recirculation mode. HLRs of 20 gal/day/ft^2 are possible and intermittent dosing is typically in the range of 15 doses/day.

- Textile media has properties conducive to use in a PBFs including lightweight, high porosity, and high surface area. It is also relatively non-degradable. The textile media itself is not reactive, but it does support biofilm growth and aerobic biological treatment. Textile sheet PBFs are normally operated in a recirculation mode. HLR_D can be 25 gal/day/ft^2 with OLRs of 0.040 lb BOD/day per ft^2.
- Styrene plastic beads function much like the coarse granular media in a RSFs, but the plastic beads are lighter, more uniform in size, and more easily transported. The beads are not reactive but they do support biofilm growth. The beads are packaged in pillow-like forms that are contained within a woven polypropylene mesh material and placed within a covered polyethylene tank. HLRs of 25 gal/day/ft^2 or more are possible and intermittent dosing is typically in the range of 48 doses/day.
- A well-designed and operated PBFs can produce consistent high quality effluent that has median values of BOD_5 and TSS concentrations that are $<$10 mg/L, with near complete nitrification of influent NH_4-N, and 99–99.9 % removal of fecal coliform bacteria. Higher levels of total N removal can typically be achieved (e.g., 60 %) in PBFs that are operated with filtrate recycling compared to single pass biofilters.

■ All types of PMBs need to be carefully designed and operated. PMBs require uniform, intermittent dosing, and commonly have electrical and mechanical components (e.g., pumps, valves). They do require routine and reliable operation and maintenance. A special concern occurs with commercial wastewaters that often have high concentrations of organic matter and nitrogen (e.g., schools, churches) and where nitrification is a treatment goal. RSFs and PBFs are often used for these applications and alkalinity may be depleted and hinder treatment. In these situations a supplemental alkalinity feed may be needed for proper process function and performance.

8-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 8 are presented in the Slides section.

8-4. Terminology

Terminology introduced and used in Chap. 8 is defined below.

Biofilter—See porous media biofilter.

BOD_5—Oxygen demand exerted over five days due to biological degradation of organic matter plus potentially bio-oxidation of ammonia.

Buried—(1) A term used to describe a pipeline, tank, or other component that is established below ground surface and covered with earthen materials. (2) Refers to porous media biofilter, soil treatment unit or similar unit operation that is established in the landscape with its wastewater infiltrative surface below the ground surface. Buried unit operations need to be designed to account for the fact that operation and maintenance functions can be difficult and rejuvenation may require excavation.

Carbonaceous BOD_5 ($cBOD_5$)—Oxygen demand exerted over five days due to biological degradation of organic matter.

Distal orifice—The orifice in lateral of a pressure distribution system that is furthest from the point at which the transport piping connects to the manifold to which the lateral and orifice are connected.

Dose—A volume of influent that is delivered and distributed to a treatment unit (e.g., a biofilter or soil treatment unit) or a zone within it.

Effective size (ES)—Term used for a mixture of particles that describes the diameter that 10 % by wt. of the particles present is smaller than.

Effluent—The liquid that is discharged from a treatment unit. For example, the effluent from a biofilter is the filtrate that is discharged (not recycled) and transported to a next treatment unit or discharged to the environment. Effluent can become the influent to another treatment unit operation. For example, in the context of landscape drip dispersal (LDU) effluent is produced by an upstream treatment unit (e.g., aerobic unit) and becomes the influent to the LDU.

First-order reaction rate—The rate of a reaction that is dependent on the concentration of one reactant (e.g., BOD_5). Zero-order reaction rates are only dependent on time and second-order reaction rates are dependent on the concentration of two reactants (e.g., O_2 and BOD_5).

Filtrate—The liquid that exits the bottom of a porous media biofilter. The filtrate can be discharged as biofilter effluent or it may be recycled back to a recirculation/dosing tank for blending with the incoming wastewater (e.g., septic tank effluent) before dosing to the biofilter.

Filtration—In the context of treatment of wastewater or other impaired waters, filtration refers to a physicochemical process that removes colloidal and particulate solids from the water during its movement through a membrane or porous media that have certain pore size and chemical properties that prevent the solids from passing through.

Foam media—A type of media used in package biofilters that is comprised of open cell foam typically configured in small cubes that are packaged in cylindrically containers.

Forward flow—The flow that is applied to the surface of a biofilter. For a single pass biofilter this is equal to the incoming daily flow. For a multiple pass biofilter (with recirculation) this flow is the equal to the incoming flow plus the recycled flow.

Free access—A term used to describe a biofilter that has an infiltrative surface that is easily accessible which facilitates operation and maintenance functions and rejuvenation if needed.

Hydraulic capacity (Q_C)—The volume of water or wastewater that can be processed through a unit operation such as a porous media biofilter from the inlet to the outlet while achieving a performance the unit operation was designed for.

Hydraulic gradient—A unit-less measure of the force causing water flow to occur through a channel or bed of porous media defined as the change in elevation head over a unit length of the flow path.

Hydraulic loading rate for design (HLR_D)—The areal loading rate applied to the surface area of a treatment unit such as a porous media biofilter or soil treatment unit that is used for design of the surface area required for a design daily flow rate.

Infiltration rate (IR)—The rate at which water passes through the infiltrative surface area of a bed of porous media.

Infiltrative surface (IS)—The horizontal surface area that comprises the top of a biofilter to which influent is distributed during a dose.

Influent—The effluent from an upstream treatment unit becomes the influent to a downstream treatment unit. For example, septic tank effluent is often used as the influent to a porous media biofilter.

Intermittent—A term that is used to describe a method of applying an influent to a treatment unit (e.g., a porous media biofilter) where there are periods of dosing and no dosing.

Lateral—A small diameter pipe with orifices in it or spray nozzles attached to it that is used for distribution of the influent uniformly over the infiltrative surface of a porous media treatment unit (e.g., porous media biofilter or soil treatment unit).

Long-term acceptance rate (LTAR)—The pseudo steady-state rate at which wastewater is transmitted through the infiltrative surface of a bed of porous media (e.g., within a porous media biofilter or a soil treatment unit) after a long period of operation and in the absence of continuous ponding of wastewater on top of the soil infiltrative surface.

Manifold—A small diameter solid wall pipe that is used to evenly distribute the influent to two or more laterals in a treatment unit (e.g., porous media biofilter or soil treatment unit)).

Monomedia—A term that describes a filter bed that is characterized by having a single layer of the same media.

Orifice—A perforation (typ. 1/8-in. diameter +/−) in the wall of a lateral pipe in a pressure distribution system through which the pressurized influent is discharged onto a porous media in a treatment unit.

Orifice shield—Refers to a cup or other protective capping device that helps disperse the discharge from an orifice while protecting the orifice from blockage by porous media used in a biofilter.

Organic loading rate (OLR)—The mass of organic matter (typically measured as lb-BOD_5/day/ft^2) that is applied to the surface of a treatment unit (e.g., porous media biofilter, constructed wetland, soil treatment unit).

Packaged media biofilter (PBF)—Packaged media biofilters are commercially manufactured in modular units containing porous media that is lightweight and suitable for shipping and has a high surface area per unit volume and weight. Examples of PBFs media include: peat fibers, foam cubes, textile sheets, and styrene beads.

Peat media—A type of media used in packaged media biofilters. Peat is a soil-like material that is a heterogeneous mixture of decomposed plant material that has accumulated in a water-saturated environment and in the absence of oxygen. When used in a PBF, peat media is containerized in manufactured modules or pods.

Physicochemical—A term used to refer to processes and reactions that have both physical and chemical characteristics. Sorption is an example of a physicochemical process.

Plug flow—A flow regime where the velocity of the fluid is assumed to be constant across any cross-section of the tank, basin or other unit perpendicular to the axis of the inlet to the outlet flow path.

Porous media biofilter (PMB)—A term used to describe a wastewater treatment unit operation that involves media placed in a container through which a wastewater effluent flows by gravity and receives treatment, primarily by attached growth biological processes. There is porosity within the bed of media by virtue of spaces between adjacent particles, filaments, or media objects. There can also be internal porosity within some types of media used in PBFs (e.g., foam cubes).

Recirculating sand filter (RSF)—A type of biofilter that is characterized by a bed of coarse sand or gravel to which primary or better quality effluent is intermittently dosed and filtrate is recycled for several passes through an unsaturated aerobic filter bed during which advanced secondary treatment can be achieved.

Recirculation—The process of directing a portion of the filtrate from a multiple-pass biofilter back to a recirculation/dosing tank where it is blended with the incoming wastewater (e.g., septic tank effluent) for dosing of the biofilter.

Recirculation ratio—The ratio of the daily filtrate flow that is recycled compared to the daily incoming flow. Recirculation ratios are typically 3–5.

Run—Refers to the length of time that a filter or other unit operation functions before maintenance or rejuvenation is needed.

Sand—A naturally occurring granular material composed of finely divided rock and mineral particles. Sand can be further defined as particles with a diameter of 0.05–2.0 mm. Sand is also a textural class of soil along with silt and clay.

Saturated hydraulic conductivity (K_S)—A term that is associated with the ability of a porous media to transmit water through it. K_S (e.g., gal/day/ft^2) is multiplied by the hydraulic gradient (e.g., typ. 1.0 for biofilters or soil treatment units) and infiltrative surface area (e.g., ft^2) to determine the hydraulic capacity (e.g., gal/day) through the porous media.

Single pass sand filter (SPSF)—A type of biofilter that is characterized by a bed of medium sand to which primary or better quality effluent is dosed and advanced secondary treatment can be achieved during a single pass through an unsaturated aerobic filter bed.

Stratified media—A term used to describe a filter bed that has multiple layers of media that have different physical and/or chemical properties.

Styrene media—A type of media used in packaged media biofilters that is comprised of uniform plastic beads that are packaged in pillow-like forms within a woven polyethylene mesh.

Surface area—(1) A term that refers to the horizontal infiltrative surface area of a biofilter that receives a dose of influent. (2) The area of the external or internal surfaces of a particle, filament or other object.

Textile media—A type of media used in package media biofilters that is comprised of textile fibers configured in sheets that are draped over rods within a module.

Total BOD (tBOD)—A measure of the total biochemical demand for oxygen exerted by microorganisms during complete degradation of organic matter and conversion of ammonium to nitrate.

Total dynamic head (TDH)—The pressure against which a pump or siphon must work to discharge a given flow rate.

Total Kjeldahl nitrogen (TKN)—A laboratory method of measurement that determines the concentrations of reduced forms of nitrogen. Total Kjeldahl nitrogen (TKN) includes organic N and ammonia N.

Transport piping—The solid-wall pipe that delivers wastewater effluent from a pump tank to the location of a treatment unit (e.g., a porous media biofilter).

Underdrain—That component of a biofilter or filter that exists at the bottom and is used to collect filtrate and convey it out of the biofilter or filter in a discharge pipe.

Uniformity coefficient (UC)—A measure of the uniformity of particles sizes in a mixture of particles (e.g., a volume of sand). The uniformity coefficient is defined as the ratio of D_{60} to D_{10} where D_{60} is the diameter that 60 % by wt. of particles are smaller than and D_{10} is the diameter that 10 % by wt. of particles are smaller than.

Unsaturated—A term used to describe the water content in porous media where the porosity is not completely liquid filled. In a porous media biofilter or a soil profile, unsaturated flow is important since the porosity in the media can contain liquid (i.e., the wastewater effluent being treated) plus air-filled porosity and this helps maintain aerobic conditions through passive aeration. It also helps ensure that the wastewater effluent applied percolates under film flow conditions with close contact to media surfaces.

Zone—Refers to a portion of a treatment unit to which influent is distributed during an individual dosing event. A treatment unit (e.g., porous media biofilter or a soil treatment unit) can have a single zone or a number of zones. Use of multiple zones can help with delivery and distribution (e.g., by reducing the discharge flow rate required by a dosing pump).

8-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 8 are listed below.

BOD	Biochemical oxygen demand
BOD_5	BOD exerted after 5 days
cBOD	Carbonaceous BOD
D_{10}	Diameter that 10 % by wt. of particles in a mixture is smaller than
D_{60}	Diameter that 60 % by wt. of particles in a mixture is smaller than
ES	Effective size (D_{10})
HLR_D	Hydraulic loading rate used for design
HRT	Hydraulic retention time
IR	Infiltration rate
IS	Infiltrative surface
LWA	Lightweight expanded clay aggregate
nBOD	nitrogenous BOD
OLR	Organic loading rate
O&M	Operation and maintenance
PBF	Packaged media biofilter
PMB	Porous media biofilter
RSF	Recirculating sand biofilter
SFE	Sand filter effluent
SPSF	Single pass sand biofilter
STE	Septic tank effluent
tBOD	Ultimate carbonaceous BOD plus nitrogenous BOD
TDH	Total dynamic head
TSS	Total suspended solids
UC	Uniformity coefficient (D_{60}/D_{10})
A_F	Cross-sectional area
A_S	PMB surface area required based on design flow and HLR_D
A_S'	PMB surface area provided based on a chosen L and W
C	Orifice discharge coefficient

C_E	Effluent concentration
C_I	Influent concentration
D	Depth of unsaturated media
dh/dz	Hydraulic gradient
D	Diameter, inside diameter
D_L	True inside diameter of the laterals
D_M	True inside diameter of the manifold
D_{PD}	Design doses per day
F	Conversion factor
G	Acceleration due to gravity
h_{dv}	Headloss through a distributor valve
h_f	Headloss caused by friction during flow in a length of pipe
h_{fl}	Headloss due to flow in a lateral between the inlet-end and the far-end orifices
h_{fm}	Headloss due to flow in the manifold piping
h_{ftp}	Headloss due to flow in the transport piping and fittings
h_{md}	Headloss in the pump discharge assembly
h_r	Residual head at the distal orifice
h_s	System static or elevation head
IR_t	Infiltration rate at time t
IR_o	Infiltration rate at startup
K	First-order reaction rate constant
K_S	Saturated hydraulic conductivity
k_T	Reaction rate at temperature, T
k_{20}	Reaction rate at 20 °C
L	Length
L_{FPer}	Length of the PMB perpendicular to the lateral orientation
L_L	Length of laterals in the PMB or zone of it
L_M	Length of the manifold
L:W	Length to width ratio
M_S	Separation of the first orifice from the manifold
n_e	Effective porosity
N_L	Number of laterals
N_O	Total number of orifices in the PMB or zone of it
N_{OLat}	Number of orifices in a lateral
N_Z	Number of zones
Q_A	Average daily flow rate
Q_C	Hydraulic capacity
Q_D	Design daily flow rate
Q_E	Effluent flow rate
Q_F	Forward flow rate
Q_{OF}	Flow rate out the orifice furthest away from the manifold
Q_{OI}	Flow rate out the orifice closest to the manifold
Q_I	Influent flow rate

Q_{Lat}	Flow rate into a lateral
Q_M	Flow rate into the manifold
Q_O	Orifice discharge rate
Q_R	Filtrate recirculation flow rate
P_{on}	Time a pump is running during a dosing event
P_{off}	Time a pump is off between dosing events
R	Ratio of recirculated flow to the influent flow
R_E	Removal efficiency
R_{FV}	Ratio of network piping to dose volume
R_Q	Ratio of discharge out of the distal orifice vs. the closest orifice
S_L	Separation distance between adjacent laterals
S_o	Distance between orifices in a lateral
S_W	Separation distance of a lateral from a sidewall
t	Time
T	Temperature
T_D	Portion of a day during which dosing occurs
V_{DE}	Volume of the dose event
V_{DT}	Volume of the dosing tank
V_{RT}	Volume of a combined dosing/recirculation tank
θ	Temperature activity coefficient

8-6. Problems

8.1. Wastewater treatment and water reclamation in a porous media biofilter (e.g., single pass sand filter or recirculating textile media biofilter) depends on what type of biological process?

8.2. Packaged biofilters are designed so that many small doses of wastewater are intermittently applied to the filter surface area each day. Which of the following best explain why this is very important to treatment in these filters: (1) maintain unsaturated film flow of effluent through the filter media, (2) maintain a short hydraulic retention time (e.g., 30 min) in the filter media, (3) maintain aerobic conditions in the air-filled porosity within the filter media?

8.3. Check which of the following best describe how the characteristics of a recirculating sand filter (RSFs) compare to those of a single pass sand filter (SPSF).

1. RSFs has media with an effective size (d_{10}) that is:
   $>$ ____; $=$_____; $<$_______ that in a SPSF
2. RSFs has media with %fines content that is:
   $>$ ____; $=$_____; $<$_______ that in a SPSF
3. The no. of doses per day used for a RSFs is:

> ____; =_____; <_______ that for a SPSF

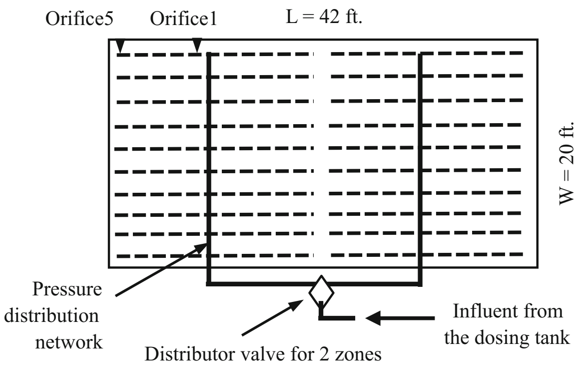
4. The RSFs surface area to treat 1000 gal/day is:

   > ____; =_____; <_______ that for a SPSF

8.4. Packaged media biofilters are made with manufactured media such as foam or textile filter media. Several characteristics of the biofilter media offer benefits compared to sand such that packaged media biofilters can be designed using higher daily hydraulic loading rates and organic loading rates compared to recirculating sand filters. Give two characteristics of the foam or textile media that are important to achieving high quality effluent when higher loading rates are used.

8.5. For the single pass sand filter shown below, answer the following design questions. (1) What average daily flow rate can this biofilter handle (gal/day)? (2) In the distribution network for each zone, there are how many manifold(s) and laterals? (3) Based on the biofilter design what is the volume of each dose (gal/dose)? (4) What flow rate would a pump deliver during a dosing event (gal/min)? (5) How long would the pump be on during a dosing event (min). (6) Would the pumping rate during dosing change if the daily flow rate were only 50 % of the design rate?

Given information and assumed values: The SPSF has two zones, each of which is 21 ft long and 20 ft wide. $Q_D = (Q_A)(PF)$ with PF = 2.0. Design HLR = 1.5 gal/day per ft^2. Lateral spacing = 2 ft and lateral wall separation = 1 ft. Orifices per lateral = 5. Distal orifice discharge rate = 0.22 gal/min. Timed dosing with 18 doses per day (from 6 a.m. to 11 p.m.).



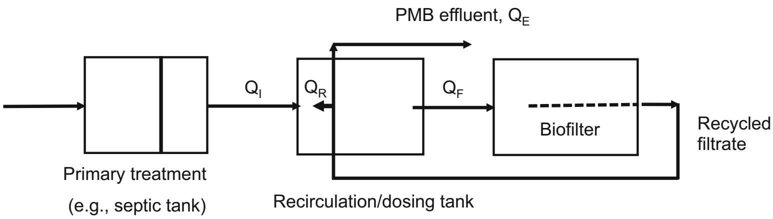
8.6. A recirculating sand filter is being designed for a motel that is projected to have an average daily flow = 2500 gal/day. The influent to the RSFs will be septic tank effluent. Based on the information provided, answer the following design questions: (1) What is the surface area required for the RSFs (ft^2)? (2) What is the volume of the recirculation/dosing tank (gal)? (3) What is the dose volume (gal/dose)? (4) What is the effluent flow rate (Q_E) under maximum daily flow conditions (gal/day)?

(5) Is either of following two sand media options suitable for use in the RSFs?

Given information and assumed values: The peaking factor for maximum daily flow = 2.0. The RSFs surface area required will be provided in 1 zone. RSFs hydraulic loading rate = 5 gal/day/ft^2. Doses/day = 48. Recirculation/dosing tank sizing HRT = 24 h and F = 0.8 The selected recirculation ratio = 4:1. Sand 1: $d_{10} = 2$ mm, $d_{60} = 4$ mm, %fines <1 %; Sand 2: $d_{10} = 0.5$ mm, $d_{60} = 5.0$ mm, %fines <1 %.

8.7. A packaged biofilter is being designed for to serve a seafood restaurant along the shore of a lake in Washington State. The PBFs is a recirculating textile media filter. Based on the given information, provide answers to the following design questions. (1) Surface area required based on the design HLR (ft^2)? (2) Surface area required based on the OLR (ft^2)? (3) PBFs surface area chosen for design (ft^2)? (4) Recirculation/dosing tank volume (gal). (5) Dose volume applied to the PBFs (gal)? (6) Average daily effluent flow rate (gal/day) if R is decreased from 4:1 to 3:1?

Given information and assumed values: Average daily flow = 5000 gal/day and peaking factor for maximum daily flow = 2.0. Influent BOD_5 = 300 mg/L. Recirculation/dosing tank sizing HRT = 24 h and F = 0.8. PBFs HLR < 25 gal/day per ft^2 and OLR < 40 lb-BOD_5/day per 1000 ft^2. Timed dosing every 30 min over 24 h/day with R = 4:1.



8.8. If you were interested in improving the removal of total nitrogen in a packaged media biofilter (e.g., recirculating textile media design) what change would you make in the flow diagram shown in Problem 8.7? With this change state what percent removal of total N might be achieved. How would the proposed change in the flow regime change the surface area required for the PBFs? With the proposed change, how would Q_E change?

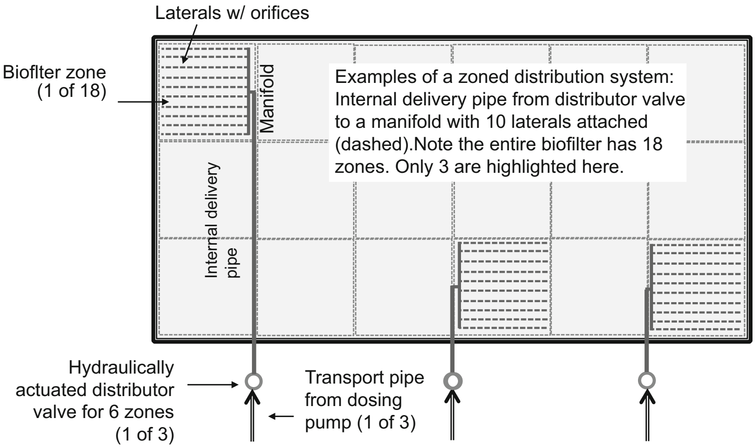
8.9. If dosing of a single pass sand filter is done using demand dosing, would the number of doses per day and the volume per dose (gal) be the same, lower, or higher on a day with a maximum flow (Q_P) compared to a day with an average flow (Q_A)?

8.10. If the dosing flow rate to a recirculating sand filter was 180 gal/min for the entire filter surface area what would the dosing flow rate be if you divided the filter into six zones and used a 6-position valve to sequentially dose each of the filter zones?

8.11. For a porous media biofilter where wastewater is applied to the filter using timed dosing, if the average daily and maximum daily flows turned out to be about 75 % of the flows used for the filter design, how would each of the following differ from that of the design (i.e., would they be the same, be lower, or be higher): the daily hydraulic loading rate (gal/day/ft^2), doses per day, volume per dose (gal/dose), and pumping rate (gal/min)?

8.12. For the Mines Park housing development located on the Colorado School of Mines campus in Golden, Colorado you are tasked with preliminary design of a recirculating sand filter. The RSFs should be capable of producing a high quality effluent (BOD_5, COD, and TSS $<$20 mg/L) that would be suitable for disinfection and reuse for lawn and garden irrigation around Mines Park. Based on the information given below, answer the following design questions. (1) Determine the total filter surface area (ft^2) and specify the sand bed depth (ft). (2) Using a 2:1 L:W ratio, what is the filter surface geometry (L and W) and area (ft^2) actually provided? (3) With the area provided (based on L and W chosen in (2)) can the RSFs handle the organic loading rate? (4) Assuming the total filter area is divided into 18 equal sized zones such as shown in the schematic below, determine the size of each zone (ft^2) and the layout of its influent distribution network—draw a schematic showing the end manifold and laterals attached to it assuming the laterals are spaced 2-ft apart with a wall separation of 1 ft. (5) Determine the volume of the recirculation/dosing tank required for the design daily flow (gal). (6) During a dosing event to a filter zone, what is the volume per dose (gal/dose), orifice discharge rate (gal/min) and total pumping rate (gal/min)? (7) Determine the head loss due to friction losses in the lateral distribution piping and check to be sure it is within generally recommended limits to ensure uniform flow out of the orifices in a lateral. (8) Calculate the total volume (gal) of the manifold and lateral piping for a zone and determine if this volume is acceptable to achieve uniform distribution in the filter zone. (9) Based on the volume/dose determined and the distribution layout for a zone, what is the total dynamic head (ft) during dosing of a distal zone. (10) For the daily dosing frequency and zone pumping rate (gal/min), what is the pump-on time (min) for a dose and pump-off time (min) between doses?

Given information and assumed values: The average daily flow from the development = 28,425 gal/day. A STEP collection system will convey septic tank effluent to the RSFs treatment site. The STE quality expected: COD = 220 mg/L, BOD_5 = 160 mg/L, TSS = 80 mg/L, TKN = 60 mg/L. RSFs design flow equals maximum recurring daily flow (PF = 2). RSFs HLR_D = 5.0 gal/day per ft^2 with R = 4:1. Filter L:W = 2:1. Filter dosing = 48 times over 24 h each day. Recirculation/dosing tank will be 50 ft away from the filter and 10 ft lower in elev. and will have a HRT = 1 day and a volume based on a sizing factor F = 1.0. The distribution network orifice size = 1/8-in. diameter and spacing between adjacent orifices = 2 ft with the distal orifice discharge head (h_r) set = 4 ft. Transport piping from the recirculation/dosing tank to a distributor valve = 3-in. Schedule 80 PVC, internal delivery piping from the valve to a zone = 2-in. Schedule 40 PVC, manifold piping = 2-in. Schedule 40 PVC, and lateral piping = 0.75-in. Schedule 40 PVC. C = 130 to account for PVC pipe aging. Head losses in the pump assembly and distributor valve are $h_{md} = 3$ ft and $h_{dv} = 15$ ft, respectively.



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Slides of Chapter 8

Decentralized Water Reclamation

Chapter 8: Treatment Using Porous Media Biofilters

Contents

8.1

8-1. Introduction

- Porous media biofiltration
  - Porous media biofilters (PMBs) use natural or manufactured media placed in a bed configuration for treatment of primary or better quality effluent during unsaturated flow through the media
  - PMBs are normally used to achieve advanced secondary treatment based on a combination of physical, chemical and biological processes
  - PMBs are quite different than filters used for water treatment or trickling filters used for biological wastewater treatment
    - Media used in a PMB is coarse but still of relatively small size
    - Hydraulic loading rates in a PMB are lower and the flow through the media is unsaturated with long travel times
    - Backwashing of a PMB is not used to remove accumulated solids and sloughing of biomass solids is not common

8.2

- PMBs can be classified based on the type of media used, the media packing, and the flow regime (Fig. 8.1)
  - The PMBs commonly used in decentralized systems include: single-pass sand filters (SPSFs), recirculating sand filters (RSFs) and packaged media biofilters (PBFs)

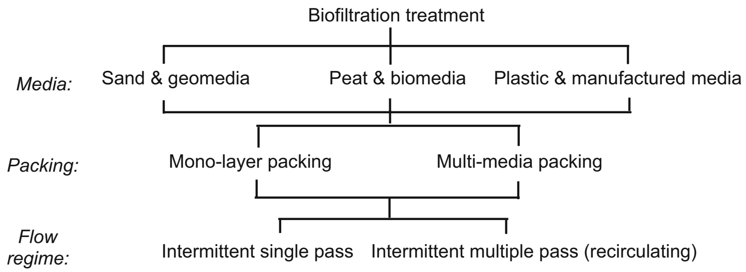


Fig. 8.1 Classification of porous media biofilters. *Note:* all PMBs considered in Chap. 8 normally rely on passive aeration during unsaturated downflow through the media

8.3

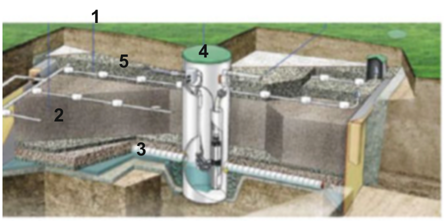
- Basic features of a single pass sand filter
  - Single pass sand filters (SPSFs) include packed beds of medium to coarse sand (Fig. 8.2a)
    - Media specifications balance hydraulic function with treatment performance
    - A SPSFs with finer grain media can remove more TSS and bacteria, but it is more subject to biofilter clogging
  - Primary treated effluent (commonly septic tank effluent (STE)) is dosed onto the biofilter surface in small volumes 12–24 times each day (e.g., 0.1 gal/ft^2 per dose) to yield daily hydraulic loading rates (HLR) in the range of 1–1.5 gal/day/ft^2
  - Each dose to the SPSF is uniformly delivered over the biofilter surface using a pressurized network of small diameter piping typically with 1/8-in. diameter orifices spaced about 2 ft apart

8.4

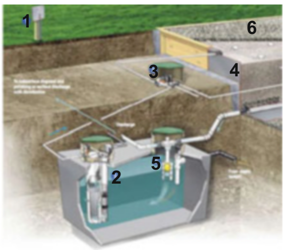
- Basic features of a recirculating sand filter
  - Recirculating sand filters (RSFs) include packed beds of coarse sand, fine gravel, glass or similar inert media (Fig. 8.2b)
  - The media size used in a RSF is coarser than that in a SPSF
    - Coarser media is needed for higher HLRs that occur with recirculation of flltrate back to a recirculation/dosing tank
    - RSF dosing is done in a manner that filtrate recirculates through the packed bed, 3–5 times or more
  - STE (or similar) is dosed onto the biofilter surface in very small volumes, typically 48–72 times/day due to the relatively large media size to yield daily HLRs in the range of 3–5 gal/day/ft^2
  - Uniform application of influent over the biofilter surface is achieved using pressurized piping networks (same as a SPSF)

8.5

- Illustration of SPSF and RSF unit operations



a. Single pass sand filter
1. Effluent distribution piping
2. Bed of medium sand media within a liner
3. Underdrain system for collecting sand filter percolate
4. Filter effluent pump basin
5. Filter bed surface is buried and access is limited



b. Recirculating sand filter
1. Control panel
2. Effluent pump with bioscreens
3. Distributing valve assembly (if used)
4. Bed of coarse sand media within a liner
5. Recirculating splitter valve
6. Filter bed surface is covered with gravel and accessible with difficulty

Fig. 8.2 Examples of (**a**) a single pass sand filter and (**b**) a recirculating sand filter including components commercially available from Orenco Systems® Inc.

8.6

- Basic features of a packaged media biofilter
  - Packaged media biofilters (PBFs) include manufactured media that is commercially packaged in modular units
    - Packaging requires PBF media to be extremely light
    - Examples of media conducive to packaging include:
      - Peat media
      - Open cell foam cubes
      - Textile media sheets
      - Polystyrene beads
  - In addition to light weight, PBF media types have some advantages over sand, gravel, glass, and similar media
    - Large media porosity and surface area per unit volume and weight (Fig. 8.3)
    - Manufacturing QA/QC can ensure media specifications
    - Relatively easier cleaning and replacement if needed

8.7

- Illustration of three commercially packaged PBFs

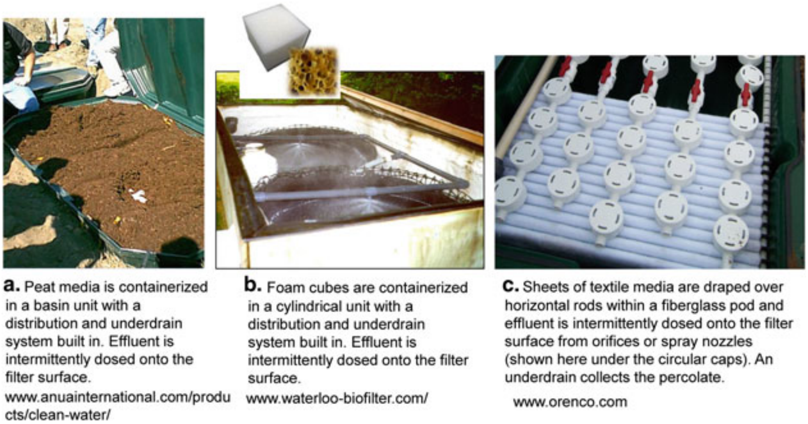


Fig. 8.3 Examples of three packaged media biofilters: (**a**) the peat media Puraflo® system by Anua, (**b**) the foam Waterloo Biofilter® and (**c**) the textile media Advantex® by Orenco Systems® Inc.

8.8

- Recirculation of filtrate through a PMB
  - RSFs and some PBFs rely on recirculation to achieve multiple passes of wastewater through the biofilter media (Fig. 8.4)
  - There are several potential benefits of employing recirculation
    - Wastewater applied to the biofilter surface has lower BOD and TSS (influent wastewater is diluted by the filtrate that is recycled), which aids treatment of higher strength wastewaters
    - With coarser media and diluted wastewaters, RSF and PBF can use a smaller biofilter surface area compared to a SPSF
    - Recirculation also helps equalize variations in influent flow and concentrations

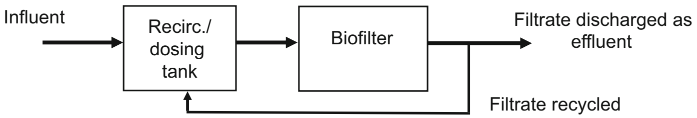


Fig. 8.4 An example of a basic flow regime for recirculation of filtrate

8.9

- Comparison of PMB unit operations
  - Table 8.1 presents key features of three PMB unit operations

Table 8.1 Comparison of typical design and operating parameters for three PMBs

Parameter	Single pass sand filters	Recirculating sand filters	Packaged media biofilters
Media used[a]	Medium to coarse sand with: $D_{10} = 0.25$–1.0 mm U.C. ≤ 4 <3 wt% fines	Coarse sand to pea gravel with: $D_{10} = 1$–5 mm U.C. ≤ 2.5 <3 wt% fines	Manufactured media: Peat, Foam, or Textile media
Media bed depth (typical)	2 ft	2–3 ft	2–5 ft
Hydraulic loading rate (gal/day/ft^2)	1–1.5	3–5	5–25
Organic loading rate (lb-BOD_5/day per ft^2)	<0.002–<0.005 Cold, Warmer	<0.008	<0.014–0.040 Peat, Textile
Recirculation ratio	Does not apply	3–5	0–5
Doses per day	12–24	48–72	48–72

[a]D_{10} = diameter that 10 % by wt. of the particles are smaller than. D_{60} = diameter that 60 % by wt. o particles are smaller than. U.C. = uniformity coefficient = D_{60}/D_{10}.

8.10

- Where are PMBs used?
  - Where secondary treatment, potentially including removal of total N, is warranted, e.g.:
    - For sources with highly variable flows which can benefit from PMB attached growth bioprocesses and timed-dosing application
    - To enable soil- or land-based treatment at sites with low permeability soils, shallow bedrock or groundwater
    - Where a high quality effluent is required for drip dispersal systems or higher rate subsurface soil infiltration systems
    - To enable surface discharge or reuse of effluent (the PMB effluent will likely have to be disinfected)
  - PMBs are used in decentralized systems serving:
    - Individual houses, residential complexes, schools, churches, businesses, etc.
    - Clusters of homes and businesses, mixed-use developments, small towns and communities

8.11

8-2. Treatment Performance

- Porous media biofilters are normally designed to achieve secondary treatment and partial nutrient removal
  - Biofilms grow on the media and biological treatment occurs
    - Biodegradable organics (dissolved, colloidal, particulate) are converted to cell mass and CO_2 which can be separated from the effluent
    - TSS in the form of suspended, colloidal and fine particulates are filtered out and organic TSS can be biodegraded
    - Reduced inorganics, some of which can exert BOD, can be converted to oxidized forms (e.g., NH_4^+ is converted to NO_3^-)
  - During normal secondary treatment there can also be some incidental removal of nutrients and pathogens
    - Nitrification typically occurs and there can be some degree of denitrification depending on system design and operation
    - Pathogens can be removed by filtration, die-off and predation

8.12

- PMBs can be specifically designed and operated to achieve high removal of nitrogen and phosphorus
  - To achieve high removal efficiencies for nitrogen
    - For conversion of NH_4^+ to NO_3^-, it is important to have adequate alkalinity in the PMB influent for nitrification to occur
    - For removal of total nitrogen, recirculation of PMB filtrate back to an anoxic zone (e.g., second compartment of a septic tank) can be used to accomplish denitrification
  - To achieve high removal efficiencies for total phosphorus
    - A reactive filter media that has a high affinity for sorption of phosphorus can be used such as:
      - Lightweight expanded clay aggregate (LWA)
      - Media rich in iron and other metal oxides

8.13

- Treatment efficiency
  - PMBs can produce a very high quality effluent that has little turbidity or color (Fig. 8.5)

Fig. 8.5 Photograph of domestic septic tank effluent (*left*) and single pass sand filter effluent (*right*)

  - Treatment efficiency can be calculated using Eq. 8.1 for the system configurations illustrated in Fig. 8.6

$$R_E = \left(\frac{C_I - C_E}{C_I}\right) \times 100\ \% \tag{8.1}$$

Where:
R_E = removal efficiency (%)
C_I = influent concentration (mg/L)
C_E = effluent concentration (mg/L)

8.14

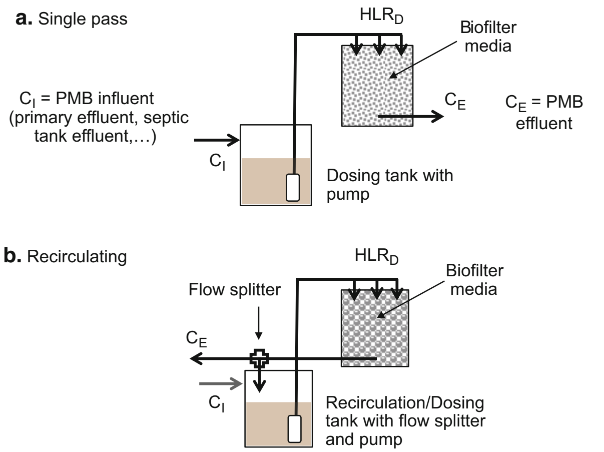


Fig. 8.6 Illustration of flow regimes within (**a**) single pass and (**b**) recirculating biofilters

8.15

- PMB treatment efficiencies for constituents of potential concern are presented in Table 8.2.

Table 8.2 Representative treatment efficiency achieved within a well designed and operated PMB

Constituent group	Effluent mg/L or % removal	Potential processes involved in treatment
BOD_5	<10 mg/L	Dissolved, colloidal and particulate organics are converted to cell mass and CO_2 which can be separated from the effluent
TSS	<10 mg/L	TSS in the form of colloidal and particulate solids are filtered out and separated from the effluent
Nitrogen[a]	<5 mgN/L NH_4^+ 20–60 % total N	Biological nitrification of NH_4+ compounds to NO_3^- with 20–60 % removal of total N by denitrification
Phosphorus	10–20 % total P	Incorporation of P into cell mass and sorption
Pathogens	99–99.99 %	Filtration, die-off and inactivation
Trace organics	0 to >90 %	Near zero removal for some compounds but up to 90 % or more removal of compounds that are susceptible to sorption and aerobic biodegradation

[a]To achieve high removal efficiencies for NH_4^+ and total nitrogen, it is important to have adequate alkalinity in the biofilter influent for nitrification and recirculation of filtrate back to an anoxic zone (e.g., second compartment of a septic tank) for denitrification.

8.16

- PMB effluent composition
  - Factors affecting treatment efficiency and effluent composition
    - Selection of the porous media with the proper attributes for PMB applications
    - Selection of a design hydraulic loading rate and organic loading rate appropriate for the influent wastewater quality and PMB media properties
    - Intermittent application of small doses of wastewater that are uniformly applied over the PMB surface area
    - Conditions in the PMB that are conducive to aerobic biological treatment (e.g., unsaturated flow with O_2 in the air-filled voids, moderate temperatures, absence of biotoxic agents)
    - Provision of required operation and maintenance (O&M)

8.17

8-3. Principles and Processes

- PMBs are attached growth biological treatment systems
  - Intermittent doses of wastewater (e.g., STE) are uniformly distributed over the surface of the PMB
  - Wastewater migrates under unsaturated conditions downward through the media (Fig. 8.7)
    - Flow through the voids between media in the PMB
    - Flow through the internal porosity of some media (e.g., foam)
  - Air-filled pore spaces in the media provide for aeration and biofilms grow on the media surfaces

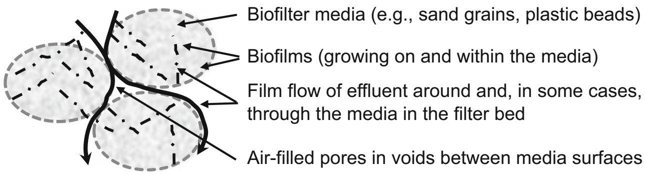


Fig. 8.7 Illustration of downward migration of wastewater through a bed of unsaturated and aerobic media in a PMB

8.18

- PMB media type and bed depth (Table 8.3)
  - Sand and fine gravel media—Washed media with durable rounded or sub-rounded grains and suitability based on sieve analysis
  - Peat, foam, textile media, plastic beads, etc.—Manufactured media with proprietary features and specifications
  - Media depths are typically 2–5 ft depending on media type

Table 8.3 PMB media and filter bed characteristics for three different types of PMBs

Parameter	SPSF	RSF[a]	PBF
Media type	Medium to coarse sand: $D_{10} = 0.25–1.0$ mm Unif. Coeff. $= D_{60}/D_{10} \leq 4$ <3 % by wt. fines (<0.074 mm)	Coarse sand to fine gravel: $D_{10} = 1–5$ mm U.C. $= D_{60}/D_{10} \leq 2.5$ <3 % by wt. fines (<0.074 mm)	Manufactured media: e.g., peat, foam, textile media, plastic beads, etc.
Bed depth	2 ft	2–3 ft	2–5 ft

[a]*Note:* Recirculating gravel, glass and other media filters are also used. These employ larger grair size media to enable higher HLR_D but generally have lower treatment efficiencies.

8.19

- Alternative configurations of PMBs
  - PMBs often use only one media in the filter bed
  - Alternative configurations are possible, but they add complexity
    - Stratified sand filters
      - Filter bed contains multiple layers w/ contrasting grain sizes
      - Goals can include: (1) using a coarse layer to enhance passive aeration within the filter or (2) using a fine layer to create an anoxic zone in the filter within which denitrification might occur
    - Reactive media filters
      - Filter bed can contain media with reactive properties
      - A primary goal is to achieve removal of nutrients, e.g.:
        - Fe^{o} filings to reduce NO_3 to N_2
        - Al_2O_3 or other minerals to sorb PO_4^{3-}
      - Reactive media filters are normally used for tertiary treatment of secondary effluent

8.20

- PMB filter hydraulics
  - Hydraulic capacity of a bed of clean sand or similar porous media is illustrated in Fig. 8.8 and quantified by Eq. 8.2

$$Q_C = (K_S)(A_F)\left(\frac{dh}{dz}\right) \tag{8.2}$$

Where:
Q_C = hydraulic capacity for flow through a bed of media (gal/day)
K_S = saturated hydraulic conductivity of a bed of clean media (gal/day/ft^2)
A_F = cross-sectional area through which flow occurs (ft^2)
dh/dz = hydraulic gradient from inlet to outlet (ft/ft)
= 1/1 for downflow under unsaturated flow conditions

8.21

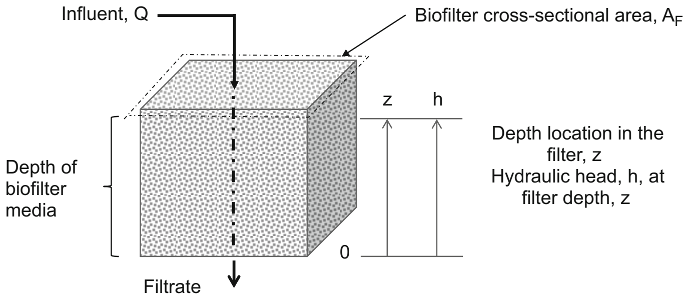


Fig. 8.8 Definition schematic for vertical flow through a bed of biofilter media in a PMB

8.22

- Estimating K_S values for a bed of clean biofilter media
  - For beds of homogeneous sand, fine gravels and plastic beads, K_S can be estimated using empirical equations
    - e.g., using the Hazen or Kozeny-Carman equations
  - For beds of heterogeneous media such as foam cubes and textile sheets, estimates of K_S are difficult to make given the complex media, packing, and flow regime which can include:
    - Flow around bulk media elements (e.g., foam cubes)
    - Flow into and through internal porosity of media elements
  - For design of PMBs an accurate quantitative estimate of K_S is normally not required
    - This is because a PMB is operated with a HLR_D that is much lower than the K_S for water through a clean bed of biofilter media

8.23

- Hydraulic capacity of some PMBs can decline during operation
  - The hydraulic capacity of a PMB at startup can gradually decline during months or years of wastewater treatment
    - Biofilter clogging processes can occur, the extent of which is based on the type of media used in the PMB, the influent wastewater composition and HLR, and the temperature
  - Finer grained media (e.g., sand in a SPSF) are particularly susceptible to filter clogging processes (Fig. 8.9)
    - Hydraulic capacity is reduced primarily by permeability loss at and near the surface of the biofilter
      - Filtration and accumulation of wastewater TSS in and on top of the PMB leads to a biomat
      - Pore-filling within the PMB by biopolymers and humic-substance like organic materials can also occur
    - The hydraulic capacity declines toward a long-term acceptance rate (LTAR) as illustrated in Fig. 8.10

8.24

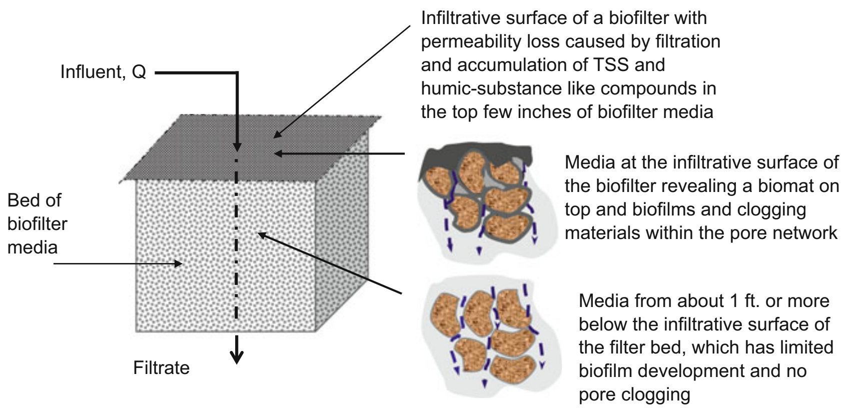


Fig. 8.9 Illustration of permeability loss at the PMB infiltrative surface due to clogging processes

8.25

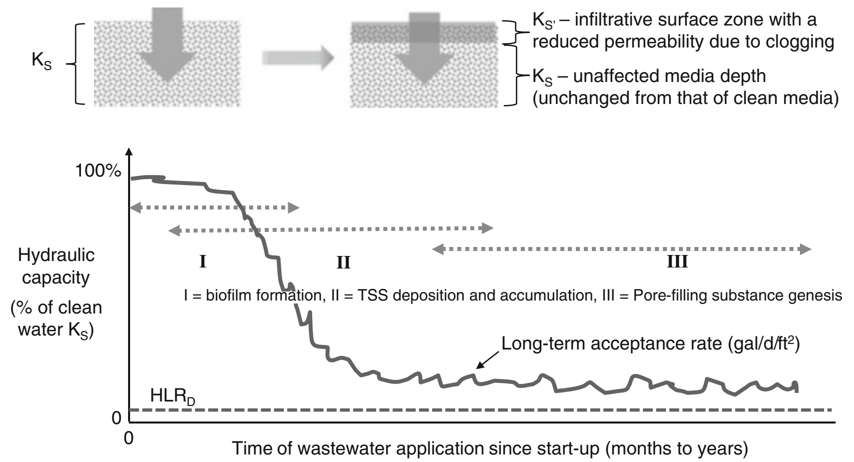


Fig. 8.10 Illustration of the decline in hydraulic capacity due to clogging processes at the PMB infiltrative surface that evolve during operation of a single pass sand filter

8.26

- Clogging causes the infiltration rate (IR) into a finer-grained PMB to decline
  - IR decline is impacted by wastewater HLR and composition
  - IR decline = ƒ (cumulative mass loading of tBOD and TSS) as expressed in Eq. 8.3

$$\frac{IR_t}{IR_o} = \frac{\exp[2.63 - 5.70(tBOD) + 41.08(TSS) - 0.048(tBOD \times TSS)]}{1 + \exp[2.63 - 5.70(tBOD) + 41.08(TSS) - 0.048(tBOD \times TSS)]} \quad (8.3)$$

Where:
IR_t = infiltration rate after a period of operation (m/day or gal/day/ft^2)
IR_o = infiltration rate at startup (m/day or gal/day/ft^2)
tBOD = cumulative mass loading of tBOD applied to the infiltrative surface after a period of operation (kg/m^2) (tBOD = ultimate cBOD plus nBOD)
TSS = cumulative mass loading of TSS applied to the infiltrative surface after a period of operation (kg/m^2)

Source: Siegrist and Boyle 1987.

8.27

- Design hydraulic loading rate (HLR_D) and run length
  - HLR_D is set ≪ the K_S for a bed of clean media, e.g.:

$$HLR_D \leq 0.01 \times K_S \quad (8.4)$$

Where:
HLR_D = hydraulic loading rate used in design (gal/day/ft^2)
K_S = saturated hydraulic conductivity of the new PMB (gal/day/ft^2)

  - Benefits of setting $HLR_D << K_S$
    - Provides unsaturated flow in the PMB which aids treatment
      - Wastewater flows in films over biofilm coated media
      - Wastewater stays in the PMB for a long hydraulic retention time
    - Allows for the reduction in hydraulic capacity over time due to biofilter clogging
    - Helps ensure that the HLR_D can be processed over the design run period

8.28

- Design run period
  - Design run period is the length of time the PMB should function before needing rejuvenation to restore its function
  - Rejuvenation requirements and methods vary
    - SPSFs with finer media
      - Rejuvenation can be required periodically
      - For a PMB with an accessible surface, raking and replacement of the top 2–4 in. of media is possible
      - For buried PMBs, excavation is needed
    - RSFs with coarse sand and similar media and PBFs with foam, textile, peat, or other media
      - Rejuvenation for sand, foam, textile media, and plastic beads is infrequently required
      - Peat and similar organic media can decompose requiring replacement
  - Design run period length depends on the operating HLR and OLR and the PMB type and surface accessibility for rejuvenation

8.29

- Illustration of a run period for two types of SPSFs (Fig. 8.11)
  - Buried SPSF with a low HLR_D (e.g., 1 gal/day/ft^2) and OLR—The run period is equal to the design life (e.g., 10–20 years or more)
  - Surface accessible SPSF with a higher HLR_D (e.g., 2 gal/day/ft^2) and OLR—The run period could be 6–12 months

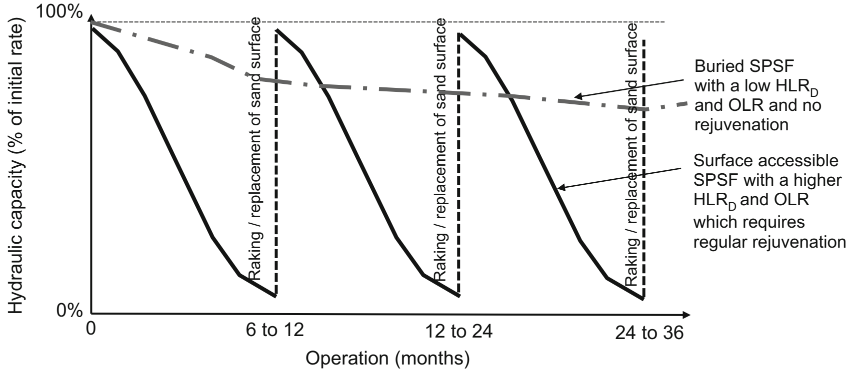


Fig. 8.11 Illustration of the run periods for two types of SPSFs

8.30

- Maintaining aerobic conditions within a PMB
  - The PMB should be dominantly aerobic
    - Oxygen is needed for aerobic biological treatment
  - Oxygen is normally added to the PMB by passive aeration
    - Air can be advectively drawn into a PMB bed following a dose of wastewater
    - Between doses, O_2 can migrate into the PMB bed by diffusion
  - Enabling aeration is normally achieved by:
    - Applying a very low HLR_D compared to the PMB hydraulic capacity
    - Limiting the OLR (e.g., lb-BOD/day/ft^2) to the PMB
    - Providing vents to introduce air into the PMB at multiple depths and through the underdrain
    - Limiting the cover depth and using a porous soil over buried single pass PMBs

8.31

- PMB constituent removal evolve over time (Fig. 8.12)
  - BOD_5 and TSS removal occurs soon after startup
  - Sorption and biouptake of NH_4^+ can occur during initial operation and nitrification of NH_4^+ develops with time
  - Phosphorus removal that occurs initially by sorption can decline
  - Bacteria and virus removal improves with operation as biofilms form and pore clogging occurs at and below the infiltrative surface

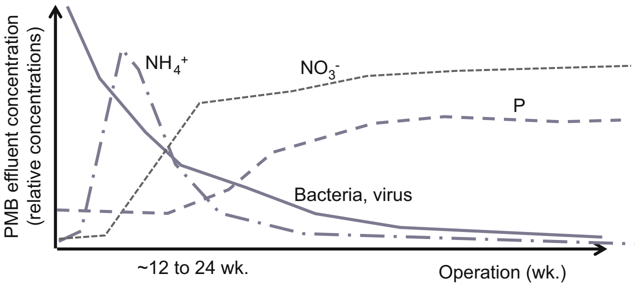


Fig. 8.12 Concentrations of constituents in the PMB effluent change over time

8.32

- Purification of many constituents is achieved by kinetic reactions—e.g., BOD removal, nitrification, bacterial die-off
  - As an illustration, for a homogeneous bed of filter sand or gravel, assuming uniform unsaturated flow and first-order kinetics, Eqs. 8.5 and 8.6 can be used (results are illustrated in Fig. 8.13)

$$\mathrm{HRT} = \frac{(d)(n_e)}{\mathrm{HLR}} \quad (8.5) \qquad R_E = \left(1 - e^{-kt}\right) \times 100\,\% \quad (8.6)$$

Where:
R_E = removal efficiency (%)
k = first-order reaction rate (h^{-1})
k: BOD = 0.04 to 0.09; NH_4^+ = 0.4 to 0.9; Fecal coli. = 0.1–0.3
HRT = hydraulic retention time in the PMB (h)
d = depth of unsaturated media in the PMB bed (ft)
n_e = effective porosity is the porosity that is actually involved with water flow and does not include dead end porosity (v/v)
HLR = hydraulic loading rate (gal/day/ft^2)
Source: Siegrist 2007.

8.33

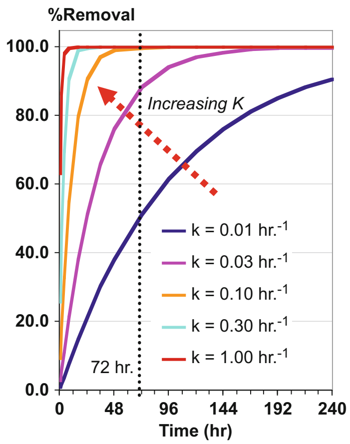


Fig. 8.13 Illustration of removal efficiencies in a PMB as a function of first-order rate constants (HLR = 1 gal/day/ft^2 and a filter bed depth = 2 ft with n_e = 0.2, HRT = 72 h) (after Siegrist 2007). (Data shown are based on calculations made using Eqs. 8.5 and 8.6)

8.34

- Effects of temperature
  - Temperature affects the rates of biological treatment processes
    - * Extremely low or high temperatures can inhibit or stop certain processes
    - * Within a range of moderate temperatures, increasing temperatures yield increasing reaction rates as expressed by Eq. 8.7

$$k_T = k_{20}\theta^{(T-20)} \quad (8.7)$$

Where:
k_T = reaction rate at temperature, T °C
k_{20} = reaction rate at 20 °C
T = temperature (°C)
θ = temperature activity coefficient (–)

In activated sludge biological systems, θ for BOD removal can be about 1.02–1.06 and some adopt values in this range for use with PMBs

8.35

8-4. Design and Implementation

■ Considerations for design and implementation (D&I) of a porous media biofilter to achieve secondary treatment and partial nutrient removal

- Development features and wastewater to be treated
- PMB type and media suitability and bed depth
- PMB surface area sizing to handle the daily flow
- PMB housing and underdrains
- PMB dosing and uniform distribution
- PMB installation at a site
- O&M that can be readily accomplished and reliably assured
- Other considerations (e.g., optional delivery and underdrain designs, tank and equipment access, watertightness)

8.36

- D&I considerations—Wastewater source
  - PMBs can treat wastewaters of widely different characteristics
    - Domestic wastewaters from houses and residential developments, wastewaters from commercial and institutional buildings
    - Other wastewaters and impaired waters including graywater, agricultural runoff, and stormwater
  - Pretreatment requirements
    - A suitable influent for application to a PMB will have characteristics typical of or better than a primary treated wastewater effluent
    - Consideration also needs to be given to water quality issues
      - Alkalinity additions may be needed to support nitrification
      - Toxic substances that can upset bioprocesses need to be absent (e.g., quaternary ammonium salts, zinc)

8.37

- D&I considerations—PMB type
  - Choice of PMB type depends on various factors (Table 8.4)
    - Availability of filter media with the proper specifications
    - Design flow and variability of flow rates to be treated
    - Wastewater composition to be treated and the HLR and OLR the PMB can handle
    - Power requirements and availability
    - Controls and complexity required for PMB operation
    - Effluent quality that the PMB is capable of producing
  - Generally:
    - SPSFs are used where design flow rates and wastewater concentrations are lower and day-to-day variations are limited
    - RSF and PBFs are used where design flow rates and wastewater concentrations are higher and where there can be day-to-day variations

8.38

Table 8.4 Characteristics of three PMBs commonly used for decentralized applications

Parameter	SPSF	RSF	PBF
Availability of media with the proper specs	Media with the proper specifications is generally available locally	Media can be difficult to obtain locally	Media is commercially available and can be shipped to a job site
Design flow (gal/day)	Used for smaller flows due to larger footprint area needed for lower HLR_D and periodic cleaning needs	Used for higher flows from commercial buildings and clustered developments due to robust performance at higher HLR_D and OLR and to ensure provision of O&M	
HLR_D (gal/day/ft^2)	1–1.5	3–5	5–25
OLR (lb-BOD_5/day per ft^2)	Low OLR are used (<0.002 to <0.005) to avoid biofilter clogging and this generally prevents use for high strength wastewaters	Higher OLR can be used (<0.008) due to recirculation	Higher OLR can be used (<0.014–0.040) due to recirculation
Power required?	Yes, unless a siphon can be used	Yes	Yes
Controls and complexity	Simpler controls for dosing without recirculation	More complex controls to enable timed dosing and recirculation	
Effluent quality	Potential for higher R_E of pathogens in finer grained media	Potential for higher R_E of total N due to recirculation	

8.39

- D&I considerations—Granular media suitability
  - Granular media used in SPSFs and RSFs needs to be durable with rounded or sub-rounded grains and be washed prior to placement in the filter bed housing
  - SPSF or RSF media specifications typically include the grain size values shown in Table 8.5, which are determined through a sieve analysis (Fig. 8.14)

Table 8.5 PMB media and filter bed characteristics for three different types of PMBs

PMB	SPSF	RSF	PBF
Media used[a]	Medium to coarse sand with: $D_{10} = 0.25$–1.0 mm UC ≤ 4 Fines $\leq 3\,\%$ by wt. <0.074 mm	Coarse sand to fine gravel with: $D_{10} = 1$–5 mm UC ≤ 2.5 Fines $\leq 3\,\%$ by wt. <0.074 mm	Manufactured media: Peat, foam, textile media, plastic beads, etc.

[a]D_{10} = diameter that 10 % by wt. of the particles are smaller than. D_{60} = diameter that 60 % by wt. o particles are smaller than. UC = uniformity coefficient = D_{60}/D_{10}. Fines content = wt.% of particles with diameters ≤ 0.074 mm.

8.40

Note: U.S. sieve no. correspond to sieve openings, e.g.

No. 10 = 2.00 mm
No. 20 = 0.85 mm
No. 40 = 0.425 mm
No. 60 = 0.25 mm
No. 100 = 0.15 mm
No. 140 = 0.106 mm
No. 200 = 0.075 mm

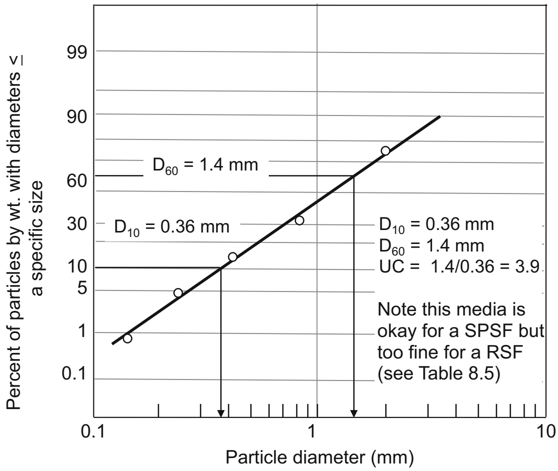


Fig. 8.14 Illustration of a sieve analysis apparatus and an example of the grain size results that are obtained

8.41

- D&I considerations—PMB surface area sizing
  - Area is determined based on the design flow and PMB surface area loading rates
    - Hydraulic loading rate for design = gal/day/ft^2
    - Organic loading rate = lb-BOD_5/day/ft^2
  - PMB loading rates include the values shown in Table 8.6
    - Loading rates are affected by temperature
      - In cold climates design rates are typically lower, especially if nitrification is needed

Table 8.6 Design loading rates for three different types of PMB

Parameter	SPSF	RSF	PBF
HLR_D (gal/day/ft^2)	1–1.5	3–5	5–25
OLR (lb-BOD_5/day per ft^2)	<0.002–<0.005 Colder, Warmer	<0.008	<0.014–0.040 Peat, Textile

8.42

- PMB surface area required based on HLR_D
  - The total PMB surface area required is calculated using Eq. 8.8
  - The length and width are then chosen to fit the landscape space available
    - Trying a L:W = 1:1 or 2:1 is a reasonable starting point
      - Use of fewer and somewhat longer laterals is preferred from an O&M perspective
    - The area provided by the L and W selected (A'_s) is calculated using Eq. 8.9 and should be approximately equal to, or greater than, A_s

$$A_s = \frac{Q_D}{HLR_D} \qquad (8.8) \qquad\qquad A'_s = L \times W \qquad (8.9)$$

Where:
A_s = PMB surface area required based on design Q and HLR_D (ft^2)
A'_s = PMB surface area provided based on chosen L and W (ft^2)
Q_D = design daily flow (gal/day) (e.g., typ. $Q_A \times$ peaking factor of 1.5)
HLR_D = Design hydraulic loading rate (gal/day/ft^2)

8.43

- PMB surface area required based on the OLR
  - With the area determined by the HLR_D and the geometry selected, the organic loading rate is calculated using Eq. 8.10
  - If the OLR is too high (e.g., see Table 8.4), additional PMB surface area is required or the influent BOD_5 must be reduced by treatment prior to the PMB

$$OLR = \frac{(Q_D)(BOD_5)(F)}{A'_S} \qquad (8.10)$$

Where:
OLR = organic loading rate (lb-BOD_5/day per ft^2)
Q_D = design daily flow (gal/day) (e.g., typ. $Q_A \times$ peaking factor of 1.5)
BOD_5 = Influent BOD_5 (mg/L)
A'_s = area of the PMB surface actually provided based on geometry selected (ft^2)
$F = 8.34 \times 10^{-6}$ = conversion factor for mg/L to lb/gal

8.44

- D&I considerations—PMB housing
  - Commercial PBFs are often delivered in manufactured housing (Fig. 8.15)
    - Plastic or fiberglass basins (also known as modules or pods)
  - SPSFs and RSFs are established in tanks or basins (Fig. 8.16)
    - Tanks or basins can be made of concrete, fiberglass, etc.
    - Basins are also established in excavations in the landscape
      - Lined with 30-mil PVC or similar
      - Bottomless (filtrate is released into the subsurface)
  - PMB surface accessibility is important to design and O&M
    - Small PMBs can be made accessible by a removable cover
    - Larger PMBs can be made somewhat accessible
      - For example, the PMB surface can be covered with 6–9 in. of pea gravel that can be moved aside to access a location

8.45

a. Example of a module for a PBF using peat manufactured by Anua Puraflo®

b. Example of a module for a PBF using textile sheets made by Orenco Systems® Inc.

Fig. 8.15 Photographs of two types of PBFs established in pre-manufactured pods and modules with covers that can be opened as needed

8.46

Fig. 8.16 Photograph of a RSF established in a neighborhood within a lined basin and the sand media surface is covered with a layer of pea gravel (Loudon et al. 2005)

8.47

- D&I considerations—Underdrains
  - Underdrain piping and bedding depends on the type of PMB
    - For SPSFs and RSFs, it is common to use 4-in. diam. plastic pipe with slots or holes in it (Fig. 8.17)
      - The plastic pipe is placed in a 4-in. layer of 0.75- to 1.5-in. washed stone overlain by a 4-in. layer of 0.375-in. pea gravel which provides bedding and a transition layer
      - This prevents movement of PMB media (e.g., sand) into the underdrain, which could clog the underdrain
    - For PBFs, the underdrain may be incorporated into the basin or pod construction
  - The underdrain piping and bedding (if used) should be designed to handle the daily flow without being submerged
  - The underdrain should be vented to allow air to enter it and help keep the bed of media in the PMB aerobic

8.48

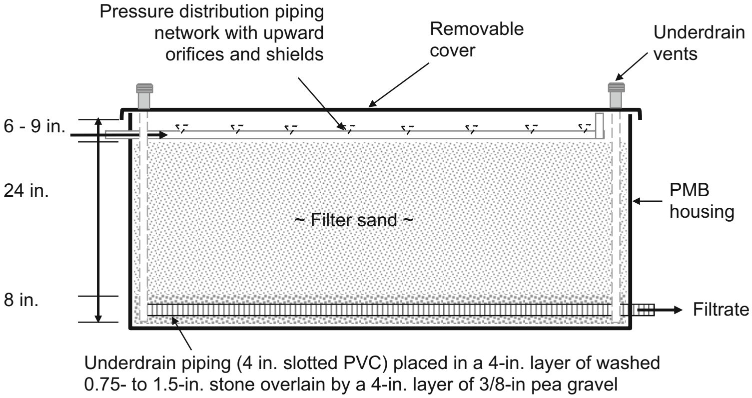


Fig. 8.17 Cross section of a SPSF illustrating the features of the underdrain

8.49

- D&I considerations—Influent delivery and distribution
  - To achieve relatively uniform distribution of influent over the entire surface area of a PMB, pressurized networks are used
    - The design process for this is described later in this chapter
  - Influent delivery into the pressurized distribution network can be accomplished using:
    - Submersible pumps
    - Siphons
  - To help achieve uniform distribution for larger flows, the PMB can be divided into zones, each of which is dosed sequentially (Fig. 8.18)
    - Influent distributor valves can control flow to different zones
      - Electrically operated valves and controls
      - Hydraulically actuated distributor valve assembly

8.50

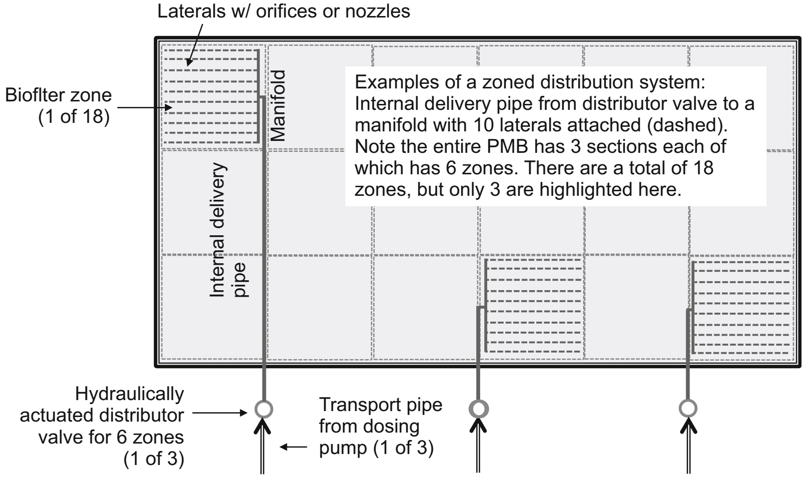


Fig. 8.18 Plan view of a recirculating sand filter for 50,000 gal/day that is divided into three sections that are dosed independently using a distributor valve that distributes influent to each of six zones

8.51

- D&I considerations—Recirculation and PMB loading
  - Recirculation is used with many types of PMBs (e.g., RSF, textile or foam PBFs)
  - A basic flow regime for recirculation is illustrated in Fig. 8.19

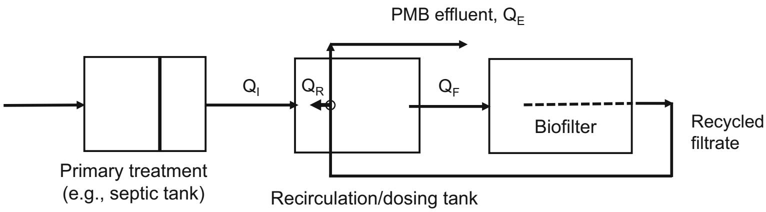


Fig. 8.19 Plan view of a basic flow regime for a porous media biofilter that employs recirculation

8.52

- For the recirculation regime shown in Fig. 8.19, Eqs. 8.11 and 8.12 can be used

$$Q_F = Q_I + Q_R = Q_I(R+1) \tag{8.11}$$

$$Q_E = Q_I \tag{8.12}$$

Where:
Q_I = influent flow (gal/day) (*Note*: Q_I varies depending on daily Q)
Q_E = effluent flow (forward flow) (gal/day)
Q_R = filtrate that is returned to a recirculation/dosing tank (gal/day)
Q_F = flow pumped onto the PMB = $Q_I + Q_R$ (gal/day)
R = ratio of recirculated flow to the influent flow = Q_R/Q_I (typ. 3–5)

8.53

- Recirculation is normally accomplished using flow splitters
  - Influent applied to the RSF or PBF flows by gravity into the underdrain and this filtrate is conveyed out of the PMB
  - A device or apparatus is used to split the filtrate flow
    - 15–25 % of the filtrate becomes effluent from the RSF or PBF
    - 75–85 % of the filtrate is returned to a recirculation/dosing tank for another pass through the PMB
  - Different approaches can be used to achieve the filtrate splitting function, including:
    - Floating buoys
    - Proportional splitters
    - Throttle valves
  - Figure 8.20 illustrates a floating buoy and Fig. 8.21 illustrates a proportional splitter

8.54

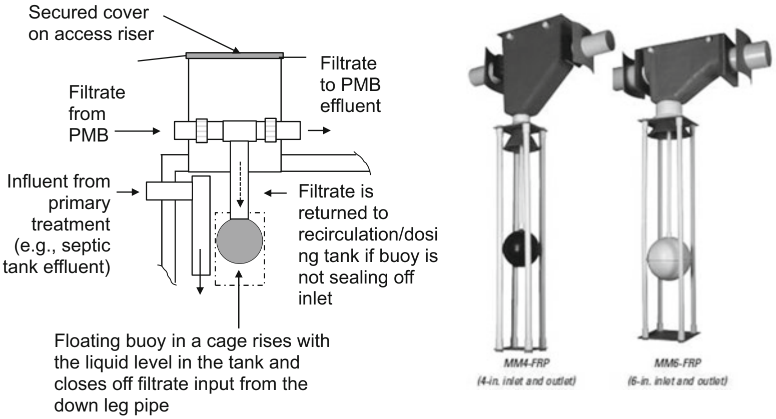


Fig. 8.20 Cross-section schematic of a floating buoy (*left*) and images of two sizes of floating buoy devices (*right*) (Images courtesy of SCG Enterprises, Inc.)

8.55

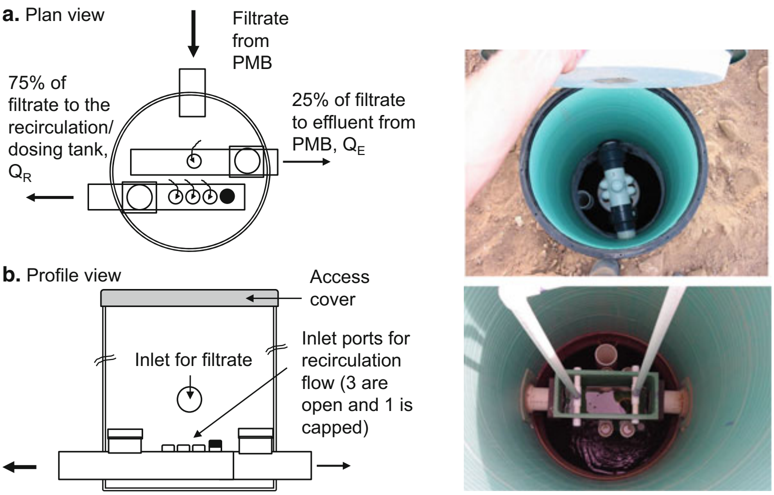


Fig. 8.21 Schematics of a proportional splitter (*left*) and photographs of two different installations (*right*). (Photographs courtesy of SCG Enterprises, Inc.)

8.56

- Modified flow regime to enhance denitrification
  - Flow regimes for recirculation can be modified to enhance denitrification and also return alkalinity to the process (which is important for nitrification)
  - Figure 8.22 illustrates a modified flow regime for this purpose

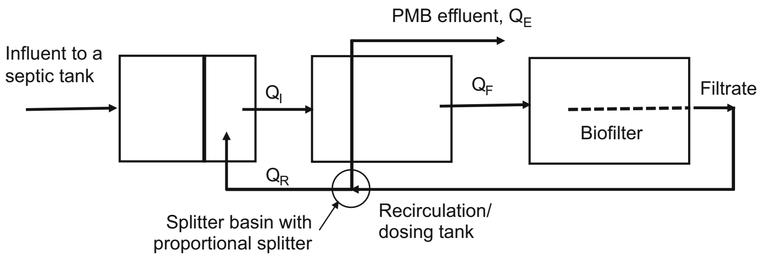


Fig. 8.22 Plan view of a modified flow regime for a PMB that employs recirculation to enhance total nitrogen removal by denitrification

8.57

- D&I considerations—Intermittent dosing to a PMB
  - Dosing tank or a combination dosing/recirculation tank
    - Used to enable intermittent dosing of influent to the PMB
    - The tank volume required is based on the design daily flow (Eq. 8.13)

$$V_{DT} \text{ or } V_{RT} = F(Q_D)(HRT) \tag{8.13}$$

Where:

V_{DT} = volume of the dosing tank for a single pass filter (gal)

V_{RT} = volume of a combined dosing/recirculation tank for a multiple pass filter (gal)

F = factor for sizing the tank volume (typ. F = 0.8–1.0)

Note: larger values of F can result in more anoxic conditions in the tank and this can aid total N removal by denitrification

Q_D = design daily flow (gal/day) (e.g., typ. $Q_A \times PF$, with PF = 1.5)

HRT = hydraulic retention time (days) (typ. 1 day)

8.58

- Providing the required dosing tank or combined dosing/recirculation tank volume
  - The tank volume can be provided by a separate compartment within a treatment unit (e.g., septic tank) or a separate tank or basin
  - For example, for a home or small business there could be a:
    - 1500-gal tank with a second compartment of 500 gal used for dosing, or a
    - 1250-gal septic tank followed by a 500-gal dosing tank
  - For larger systems
    - The volume of the dosing tank or combined dosing/recirculation tank volume is calculated
    - Flow equalization may also be desired and that could increase the volume of the tank chosen
    - The appropriate type and size tankage is then established

8.59

- Volume of an individual dose
  - Dosing consists of applying an equal volume of influent to a PMB, with the dose frequency and volume per orifice dependent on the PMB type
  - The dose volume is determined using Eq. 8.14

$$V_{DE} = \frac{(Q_A)(R+1)}{(N_Z)(D_{PD})} \tag{8.14}$$

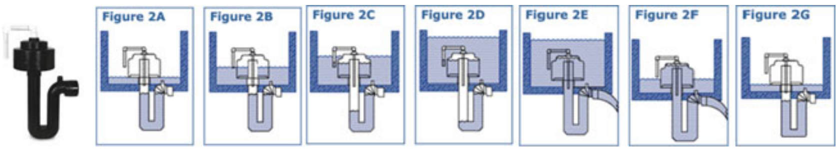
Where:
V_{DE} = volume of each dose event (gal)
Q_A = average daily flow (gal/day) (*Note:* this is Q_A not $Q_A \times PF$)
R = recirculation ratio (R = 3–5 for a multiple pass PMB or 0 for a single pass PMB)
N_Z = number of PMB zones to be dosed sequentially
D_{PD} = design doses per day per PMB zone (no./day)
= Typ. 12–24 for an SPSF and 48–72 for a RSF or PBF

8.60

- Dosing using submersible effluent pumps
  - Effluent pumps are commonly used for PMB dosing and they are available from various manufacturers
  - The pump(s) are installed in a dosing tank or basin
    - To avoid discharge of solids that accumulate, pump intakes should be off the tank or basin floor
      - Pumps can have long legs (e.g., 3 in. or more)
      - Pump can be placed on a 2- to 3-in. concrete pad
      - Pump can be suspended on hangers 12 in. or more off the floor
  - Pumping systems can be simplex (one pump) or, for larger flows where redundancy is important for reliability, duplex (two pumps that alternate)
    - Note that for very large flows four or more pumps or multiple duplex pump systems can be used

8.61

- Dosing using automatic dosing siphons
  - Siphons can be used especially where there isn't power
  - Siphons intermittently discharge a dose based on siphon design and hydraulics as shown in Fig. 8.23



✓ As liquid rises, air inside the siphon bell exhausts through a vent (2A)
✓ But then the vent is sealed by the rising liquid (2B)
✓ Air inside the bell is compressed into the long leg of the siphon trap (2C)
✓ Air reaches the invert in the trap at the high liquid level (2D)
✓ Liquid fills the bell and starts a siphon action (2E)
✓ Liquid is discharged as a dose until a low level is reached (2G)
✓ The dosing cycle repeats itself (2A…)

Fig. 8.23 Illustration of a commercially available automatic dosing siphon and how dosing occurs (www.siphons.com/index.html)

8.62

- Demand dosing versus timed dosing
  - Demand dosing (pumps or siphons) (Fig. 8.24)
    - Siphons operate automatically based on the liquid level and pumps are turned on and off by high- and low-level float switches
    - Demand dosing uses simpler controls but it can lead to more time-variable PMB loading and even overloading
  - Timed dosing (pumps only) (Figs. 8.25 and 8.26)
    - Pumps are turned on and off using a programmable timer
    - Timed dosing provides more uniform dosing, equalizing the wastewater delivery which can provide better performance
    - Siphons can not be used to achieve timed dosing
  - Dose volumes under demand dosing vs. timed dosing
    - Demand dosing usually has a few larger volume doses per day
    - Timed dosing normally is used to provide numerous smaller volume doses more uniformly throughout a normal day

8.63

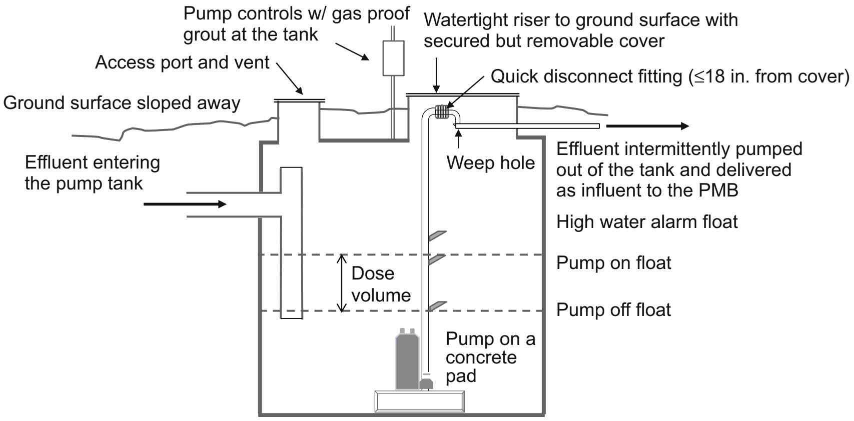


Fig. 8.24 Cross section view of a pump tank and pump set up with float switches for demand dosing. (*Note*: a siphon chamber with siphon could also be used for demand dosing.)

8.64

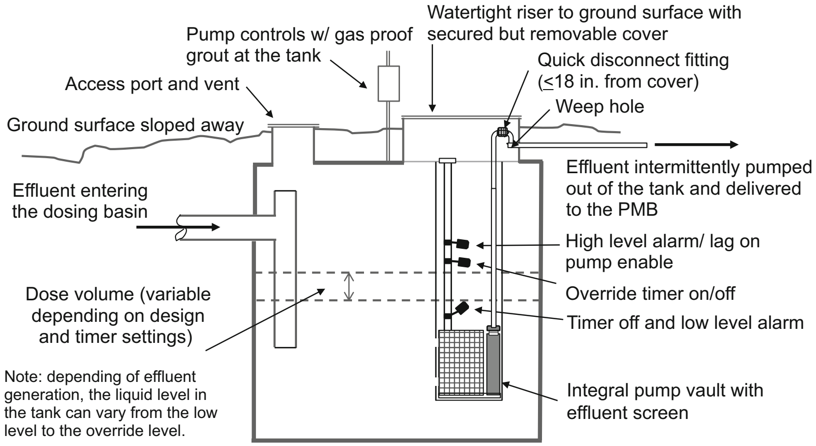


Fig. 8.25 Cross section view of a pump tank set up for timed dosing including an integral pump vault with effluent screen

8.65

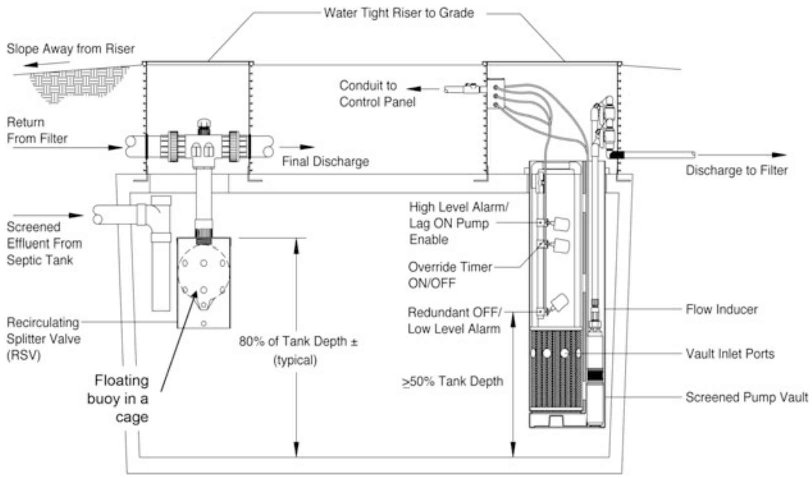


Fig. 8.26 Cross section view of a combined dosing/recirculation tank with a pump setup for timed dosing (Loudon et al. 2005)

8.66

- Distributor valves for dosing to multiple zones in a PMB
  - For large design flows (e.g., 10,000–50,000 gal/day), dosing of the entire PMB would require large pumps and piping sizes and these can be costly
  - To keep pump and piping sizes smaller for larger design flows, distributor valves can be used to dose individual zones of a PMB (see Fig. 8.18)
    - For example, if 120 gal/min is the dosing rate for an entire PMB, dividing it into 6 zones which are dosed sequentially reduces the dosing rate to 20 gal/min
  - Distributor valve options include:
    - Electrically operated valves and controls, or
    - Hydraulically actuated distributor valve assemblies
  - Figure 8.27 illustrates a distributor valve used to sequentially dose each of 6 zones in a PMB

8.67

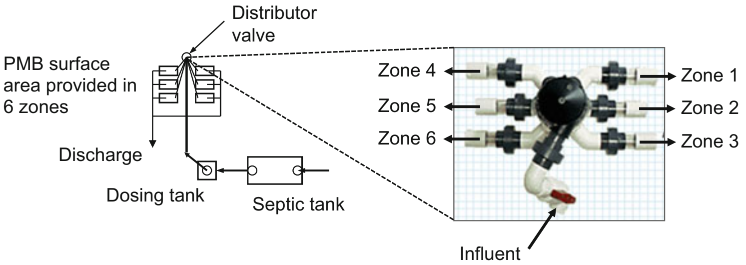


Fig. 8.27 Illustration of how the total PMB surface area required can be provided in six zones that are dosed sequentially with a 6-step hydraulically actuated distributor valve (www.orenco.com/ots/ots_rsf_disVavAss.asp).

Note: headloss through the distributor valve is about 5–25 ft depending on the type of valve and flow rate.

8.68

- D&I considerations—Uniform distribution of influent to the PMB
  - Pressurized networks are used to achieve uniform distribution
  - Pressurize distribution networks include several key components as illustrated in Fig. 8.28
    - Transport piping—Solid-wall pipe that delivers influent from the dosing tank to the distribution network in the PMB
    - Manifolds—Solid-wall pipe with laterals attached that delivers influent to the PMB through the laterals
    - Laterals—Small diameter pipe with 3/32- to 3/16-in. diameter orifices on 1.5–2.5 ft spacing
      - In some PMBs, spray nozzles are used instead of orifices
  - Pressurize distribution network operation involves several phases
    - Operational phases include: pressurization during filling, uniform distribution, depressurization and draining, and resting between doses

8.69

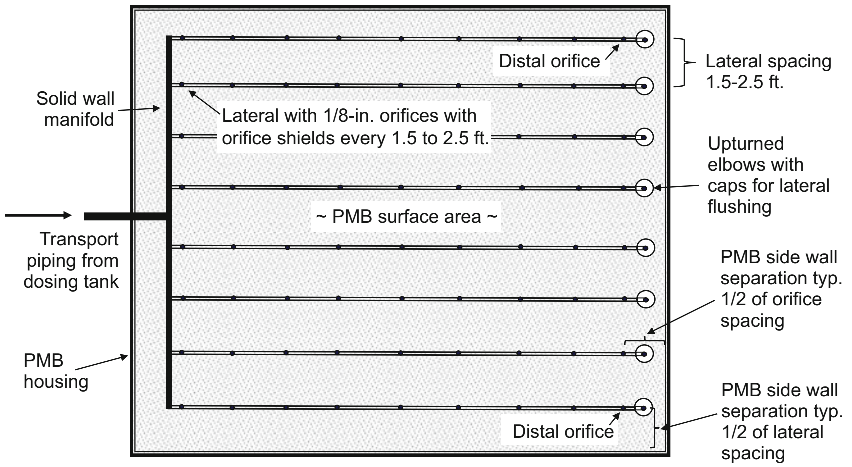


Fig. 8.28 Detailed plan view of an example pressurized network in a PMB

8.70

- Orientation of orifices in the laterals
  - Top of lateral for upward discharge (Figs. 8.29a and 8.30)
    - Need to have an orifice shield
    - Can have less clogging and less leakage while filling
    - Design for piping drainage is important in cold climates
      - Every fifth orifice can be down-facing to drain lateral
  - Bottom of lateral for downward discharge (Fig. 8.29b)
    - More prone to clogging
    - Still need to have an orifice shield
    - Piping drainage naturally occurs
      - Place at least 2 orifices up-facing with shields to facilitate quick drainage of a lateral to prevent freezing
- Features of a pressurized distribution network within a larger PMB are shown in Fig. 8.31

8.71

Fig. 8.29 Orifices can be oriented to discharge flow out of a lateral in an (**a**) upward or (**b**) downward direction

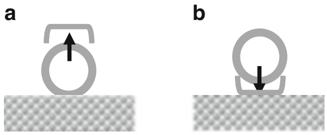


Fig. 8.30 Photograph illustrating the uniform distribution achieved using a pressure distribution network for a RSF that has 1/8-in. diameter orifices designed for discharge with a residual head of 5 ft (Photograph courtesy of Orenco Systems® Inc.).
(The orifices are oriented in an upward direction and will be covered with orifice shields before a layer of fine gravel is placed over the distribution network. The RSF serves 170 EDUs in Elkton, Oregon and is divided into four zones and each zone is pressurized by a 1-hp pump.)

8.72

Fig. 8.31 Photographs of a pressurized distribution network with upward oriented orifices and orifice shields and the pea gravel cover that is used to embed and protect it (*left*) and return outflow to help equalize pressures in the network and upturned legs for lateral flushing (*right*) (Loudon et al. 2005)

8.73

- Guidance concerning SPSF or RSF layout and effluent delivery
  - Design considerations and typical values are shown in Table 8.7

Table 8.7 Design considerations and typical values concerning SPSF or RSF layouts and effluent delivery[a]

Design considerations	Typical value	Reason
Biofilter zone size	$\leq$1000 ft^2	Maintain dosing flow rate <100 gal/min
Network piping diameters	Manifolds = 1.5 in. Laterals = 0.75 in.	Minimize head losses due to friction and enable uniform delivery
Orifice sizes	1/8 in. (3/32–3/16 in.)	Size range where orifice flow rates are <1.0 gal/min and clogging is not a major problem
Doses per day	12–24 for SPSF 48–72 for RSF	Needed for treatment since it helps maintain unsaturated flow through the sand media
Volume discharged from each orifice during a dose	$\leq$0.25 gal	
Difference in orifice discharge rates within a lateral	$\leq$10 %	Helps ensure uniform application over the entire PMB surface area
Manifold plus lateral pipe volume compared to dose volume	$\leq$20 %	For networks that drain between doses, avoid localized overloading while the network is filling up

[a]*Note*: PBF_s often include distribution piping networks and controls for timed dosing but these same typical values can apply.

8.74

- Design of a pressure distribution network
  - The design process involves several steps
    - * Layout and size the network piping for the PMB or zones of it
    - * Select an orifice size and spacing (spray nozzles are also used)
    - * Calculate the discharge rate for the distal orifice based on a chosen residual head
    - * Determine the dosing flow rate to the PMB or a zone of it
    - * Check the discharge rates for the orifices at the entry and end of a lateral to be sure the difference is within engineering practice limits
    - * Compare the volume of the manifold and lateral piping within the PMB or a zone to the dose volume to be sure the difference is within engineering practice limits
    - * Determine the total dynamic head during a dosing event
    - * Select a suitable pumping system
    - * Determine the settings for timed dosing (if timed dosing is used)
  - The design process for a pressure distribution network employing laterals with orifices is illustrated in the following pages

8.75

- Determining the number and length of laterals
  - * Number of laterals in a PMB or zone of it (Fig. 8.32)

$$N_L = \left[\frac{(L_{FPer} - (S_W \times 2))}{S_L} + 1\right] \tag{8.15}$$

Where:

N_L = number of laterals in the PMB or zone of it
L_{FPer} = length of the PMB perpendicular to the lateral orientation (ft)
S_W = separation distance of the lateral from the PMB wall (ft)
S_L = separation distance between adjacent laterals (ft)

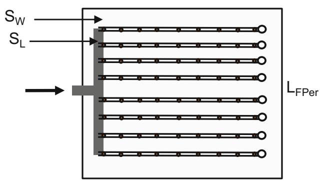


Fig. 8.32 Illustration of a pressurized distribution network with a generic layout of the network that can be used to determine the number of laterals in the PMB or a zone within it

8.76

* Length of each lateral (see Fig. 8.33)

$$L_L = (L_{FPar} - (S_W \times 2)) \quad (8.16)$$

Where:
L_L = length of laterals in the PMB or zone of it (ft)
L_{FPar} = length of the PMB parallel to the lateral orientation (ft)
S_W = separation of the lateral end from the PMB wall (ft)

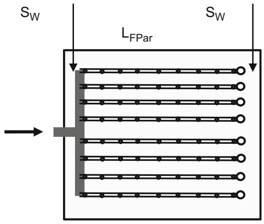


Fig. 8.33 Illustration of a pressurized distribution network within a PMB or zone of it illustrating how to determine lateral length

8.77

○ Determine the length of the manifold (Fig. 8.34)

$$L_M = (L_{FPer} - (S_W \times 2)) \quad (8.17)$$

Where:
L_M = length of the manifold in the PMB or zone of it (ft)
L_{FPer} = length of the PMB perpendicular to the lateral orientation (ft)
S_W = separation of the lateral end from the PMB wall (ft) (typ. = 1/2 the lateral spacing)

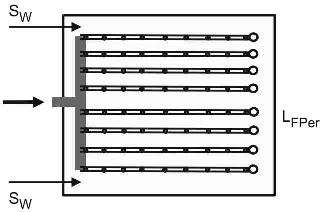


Fig. 8.34 Illustration of a pressurized distribution network illustrating how to determine the manifold length

8.78

- Determine the number of orifices in a lateral and in the PMB or a zone within it

$$N_{OLat} = \frac{(L_L - M_S)}{S_o} + 1 \qquad (8.18)$$

$$N_O = N_L \times N_{OLat} \qquad (8.19)$$

Where:
N_{OLat} = number of orifices in a lateral
N_O = total number of orifices in the PMB or zone of it (–)
N_L = number of laterals in the PMB or zone of it (–)
L_L = length of the manifold in the PMB or zone of it (ft)
M_S = separation of the first orifice from the manifold (ft)
S_o = distance between orifices in a lateral (ft)

8.79

- Select a residual head for the distal orifice(s) as depicted in Fig. 8.35
  - Residual head is the pressure available in the lateral that forces flow out of the distal orifice(s) (Q_O)
    - Residual head (h_r) is lost during pressurized discharge through the distal orifices
    - The residual head value is selected (typ. 3–5 ft)
  - After setting h_r the flow rate discharged for a selected diameter orifice (Q_O) can be calculated

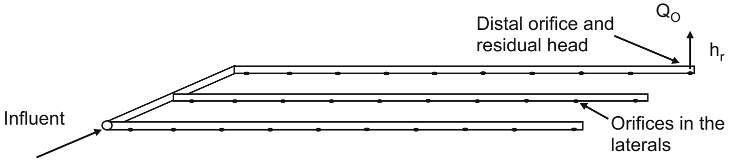


Fig. 8.35 Illustration of a pressure distribution network and the residual head in at the distal orifice in one of the laterals

8.80

- Determine the discharge rate for the distal orifice(s) (Fig. 8.36) using Eq. 8.20

$$Q_O = 2.45C\left(D^2\right)\sqrt{2gh_r} \tag{8.20}$$

Where:
Q_O = discharge from an orifice (gal/min)
C = orifice discharge coefficient (0.63 for sharp edge) *Note*: if holes are drilled by hand and not uniform, C can vary widely
D = diameter of orifice (in.) (typ. 3/32 to 3/16 in.)
g = acceleration due to gravity (32.2 ft/s^2)
h_r = residual pressure head in the lateral at the distal orifice (ft)

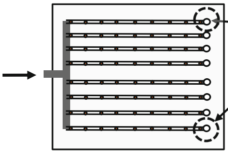


Fig. 8.36 Illustration of what are considered distal orifices in a pressurized distribution network

8.81

- Determine the lateral flow rate and the total flow rate during a dosing event using Eqs. 8.21 to 8.23

$$Q_{Lat} = Q_o \times N_{OLat} \tag{8.21}$$
$$Q_{DE} = Q_{OLat} \times N_L \tag{8.22}$$
$$Q_{DE} = Q_O \times N_O \tag{8.23}$$

Where:
Q_{Lat} = flow rate into a lateral during a dosing event to the PMB or zone of it (gal/min)
Q_O = discharge from the distal orifice (gal/min)
Q_{DE} = total flow rate during a dosing event to the PMB or zone of it (gal/min)
N_{OLat} = number of orifices in a lateral
N_O = total number of orifices in the PMB or zone of it
N_L = total number of laterals in the PMB or zone of it

8.82

- Figure 8.37 illustrates orifice discharge rates as a function of diameter and residual head as calculated using Eq. 8.20
  - For example, with orifices and laterals on 2-ft separation 1000 ft^2 of PMB surface area has 250 orifices and $Q_{DE} = 97.5$ gal/min for 1/8-in. diameter orifices and a residual head of 4 ft

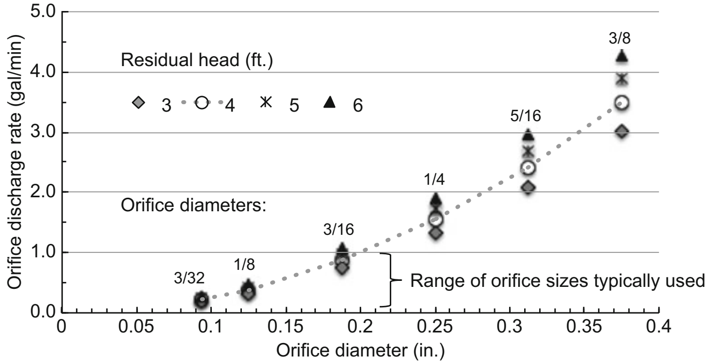


Fig. 8.37 Orifice discharge rates as a function of diameter and residual head

8.83

- Checking the uniformity of distribution within the network
  - The headloss per unit length of pipe in a manifold with laterals attached, or a lateral with orifices in it, declines with distance along the pipe length
    - This is due to the decline in the flow rate with distance along the manifold and along the laterals (Fig. 8.38)
    - The headloss in a manifold with laterals attached, or a lateral with orifices, equals about 0.33 of the headloss in an equal-length of solid wall pipe

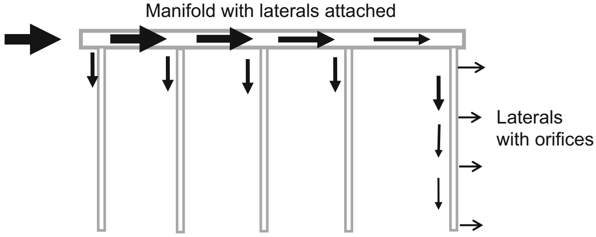


Fig. 8.38 Illustration of flow rate declining in a manifold and a lateral

8.84

* The head loss due to flow in a lateral distribution pipe can be calculated using Eq. 8.24

$$h_{fl} = 0.33 \times \left[10.5(L)\left(\frac{Q_{Lat}}{C}\right)^{1.85}\left(D^{-4.87}\right)\right] \tag{8.24}$$

Where:
h_{fl} = headloss caused by friction during flow in a length of lateral pipe (ft)
L = length of pipe (ft)
Q_{Lat} = flow into a lateral during a dosing event (gal/min)
C = Hazen-Williams coefficient (C = 150 for new plastic pipe, but C can be lower for used pipe (e.g., C= 130))
D = true inside diameter of the lateral pipe (in.)
0.33 = adjustment factor to account for the fact that the manifold has laterals attached to it and the flow rate declines with distance along the manifold (see following page)

8.85

* Based on the lateral head loss, the difference in discharge rates between the first and last orifice in a lateral can be calculated using Eq. 8.25
  – Want ΔQ_O between orifices to be <10 %, which means R_Q >90 %

$$R_Q = \left(\frac{Q_{OF}}{Q_{OI}}\right) \times 100\% = \sqrt{\frac{h_r}{h_r + h_{fl}}} \times 100\,\% \tag{8.25}$$

Where:
R_Q = ratio of the discharge out the distal orifice vs. the closest orifice in a lateral (%)
Q_{OF} = flow rate out the orifice furthest away from the manifold (gal/min)
Q_{OI} = flow rate out the orifice closest to the manifold (gal/min)
h_r = residual pressure head in the lateral at the distal orifice (ft)
h_{fl} = headloss due to flow in a lateral between the inlet-end and the far-end orifices

8.86

* The fraction of a dose used to fill the network can be calculated using Eq. 8.26
  – To avoid localized overloading during filling or draining, the network volume should be $\leq$20 % of a dose volume If R_{FV} is >20 %, the network can be redesigned

$$R_{FV} = \left\{ \frac{0.785\left[(D_M)^2 L_M + (D_L)^2 (N_L \times L_L)\right]\left(\frac{1\ \text{ft}^2}{144\ \text{in}^2}\right)\left(\frac{7.48\ \text{gal}}{\text{ft}^3}\right)}{V_{DE}} \right\} \times 100\ \% \quad (8.26)$$

Where:
R_{FV} = ratio of network piping to dose volume (%)
D_M = true inside diameter of the manifold (in.)
L_M = length of the manifold (ft)
D_L = true inside diameter of the laterals (in.)
N_L = number of laterals (no.)
L_L = length of the laterals (ft)
V_{DE} = volume of a dose to the PMB or zone of it (gal)

8.87

- Determine the total dynamic head during dosing
  * Eq. 8.27 includes the TDH components as illustrated in Fig. 8.39

$$\text{TDH} = (h_s + h_{md} + h_{dv} + h_r) + (h_{fl} + h_{fm} + h_{ftp}) \quad (8.27)$$

Where:
h_s = system static or elevation head (ft) (based on site topography)
h_{md} = headloss in the pump discharge assembly (ft) (based on equip.)
h_{dv} = headloss through a distributor valve (if present) (ft) (based on equip.)
h_r = residual head at the distal orifice in the network (ft)
h_{fl} = headloss due to flow in the lateral with the distal orifice in it (ft)
h_{fm} = headloss due to flow in the manifold piping from the connection of the transport piping to the lateral with the distal orifice in it (ft)
h_{ftp} = headloss due to flow in the transport piping and fittings from the recirculation/dosing tank to the manifold (ft)

8.88

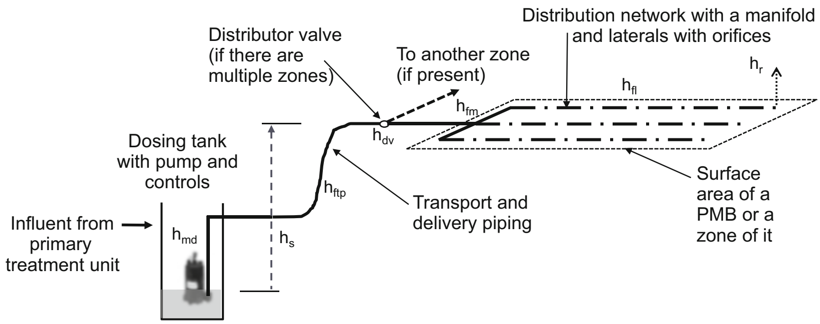


Fig. 8.39 Illustration of the layout and total dynamic head components of a dosing and pressurized distribution network for uniform application of wastewater to the surface area of a PMB

8.89

- Determining the headlosses associated with site conditions and network equipment
  - h_s—this is determined by the elevation change between the low water level in the recirculation/dosing tank and the distribution network
  - h_{md}—this is the headloss that occurs with flow through the pump assembly into the transport piping; values depend on equipment and assembly details (typ. values can be $<$3 ft)
  - h_{dv}—this is the headloss associated with a hydraulically actuated distributor valve (typ. values can be ~5 to 25 ft)
  - h_r—this is the residual head at the distal orifice, which is chosen to help ensure uniform delivery along the length of a lateral (typ. values are 3–5 ft)

8.90

- Determining the headlosses in the distribution network
  - Laterals—The headlosses in the laterals (h_{fl}) are determined as described earlier using Eq. 8.24
  - Manifolds—The headlosses in the manifold piping (h_{fm}) are determined using Eq. 8.28

$$h_{fm} = 0.33 \times \left[10.5(L)\left(\frac{Q_M}{C}\right)^{1.85}\left(D^{-4.87}\right)\right] \tag{8.28}$$

Where:

h_{fm} = headloss caused by friction during flow in the manifold piping (ft)
L = length of pipe (ft)
Q_M = flow rate from the transport pipe connection point into the manifold section that leads to the lateral with the distal orifice in it (gal/min)
C = Hazen-Williams coefficient (C = 150 for new plastic pipe, but C can be lower for used pipe (e.g., C = 130))
D = true inside diameter of the manifold pipe (in.)
0.33 = adjustment factor to account for the fact that the manifold has laterals attached to it and the flow declines with distance along the manifold

8.91

- Determining the headlosses in the transport piping that delivers a dose to the PMB or zone of it using Eq. 8.29

$$h_{ftp} = \left[10.5(L)\left(\frac{Q_{DE}}{C}\right)^{1.85}\left(D^{-4.87}\right)\right] \tag{8.29}$$

Where:

h_{ftp} = headloss caused by friction during flow in the transport piping (ft)
L = length of pipe (ft)
Q_{DE} = flow rate during a dosing event (gal/min)
C = Hazen-Williams coefficient (C = 150 for new plastic pipe, but C can be lower for used pipe (e.g., C = 130))
D = true inside diameter of the transport pipe (in.)

8.92

- Selecting a suitable pumping system
  - A suitable pumping system (or siphon) would have to be able to deliver a dosing flow rate (Q_{DE}) against the calculated TDH (Eq. 8.27)
  - Figure 8.40 shows an example head versus discharge curve for an effluent pump

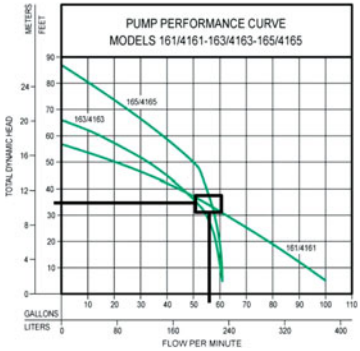


Fig. 8.40 Example of a pump head versus discharge performance curve

8.93

- Wastewater effluent delivery by pumping during timed dosing
  - Timed dosing involves periods where a pump is turned on (Eq. 8.30) and off (Eq. 8.31)

$$P_{on} = \frac{V_{DE}}{Q_{DE}} \qquad (8.30) \qquad P_{off} = \left[\frac{T_D}{N_Z \times D_{PD}}\right] - P_{on} \qquad (8.31)$$

Where:

Q_{DE} = pumping rate during a dose applied to the PMB or a zone of it (gal/min)
P_{on} = time a pump is running during dosing of the PMB or a zone (min)
P_{off} = time a pump is off between dosing events (min)
V_{DE} = volume of a dose to a PMB area (gal)
T_D = portion of a day during which dosing occurs (min/day)
N_Z = number of zones the pump will deliver doses to
D_{PD} = design doses per day (typ. 12–24 for a SPSF and 48–72 for a RSF or PBF)

8.94

- D&I considerations—Optional distribution and underdrain features
  - Intermittent dosing and uniform distribution to the surface of a PMB is often accomplished using pressure distribution systems with perforated laterals
    - However, optional designs are available that can achieve equal or better distribution
      - Instead of orifices, helical spray nozzles can be attached to pressurized piping (Fig. 8.41a)
      - Some PBFs utilize helical spray nozzles to ensure uniform distribution of wastewater over the surface of the biofilter media
      - Small diameter tubing outfitted with drip emitters can also be used to enable micro-dosing of biofilter surfaces (Fig. 8.41b)
  - Filtrate can be collected in optional underdrains
    - Underdrains for filtrate collection can use chambers rather than 4-in. diameter perforated pipe embedded in gravel (Fig. 8.41b)

8.95

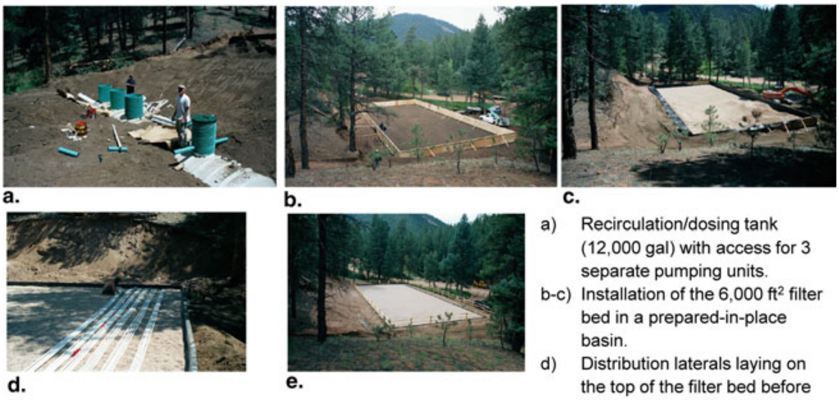
a) Helical spray nozzle distributing septic tank effluent over foam filter media used in a Waterloo Biofilter®. (Photographs courtesy of Waterloo Biofilter®)
b) Drip dispersal tubing containing drip emitters placed within a set of chambers and used to disperse septic tank effluent over the surface of a sand filter.
Chambers and chamber-like piping can be used for underdrains also.

Fig. 8.41 Examples of optional distribution and underdrain systems that can be used PMBs

8.96

- D& I considerations—Installation at the site
  - PMBs can be constructed in containers that are pre-manufactured or in prepared-in-place tanks or basins
    - These types of PMB containers need to be watertight
  - SPSFs can be established in unlined excavations in the landscape and these are watertight with the exception of an open bottom
  - Dosing systems, timing controls, and delivery networks need to be properly established and tested
  - All types of PMBs (SPSF, RSF, and PBF) can be constructed using commercially available materials and equipment packages
    - However, for SPSF and RSF the proper media needs to be locally acquired and delivered to the site of the PMB installation
  - Photographs showing construction details for different PMB projects are presented in Figs. 8.42, 8.43, 8.44 and 8.45

8.97



a) Recirculation/dosing tank (12,000 gal) with access for 3 separate pumping units.
b-c) Installation of the 6,000 ft^2 filter bed in a prepared-in-place basin.
d) Distribution laterals laying on the top of the filter bed before assembly.
e) Completed RSF.

Fig. 8.42 Photographs taken during the installation of a recirculating sand filter serving a YMCA camp in Colorado. (Photographs courtesy of SCG Enterprises, Inc.)

8.98

a. b. c.

a) Six AX20 textile media filters (Orenco Systems®, Inc.).
b) Installation of a 1,000 gal dosing tank to deliver filter effluent to infiltration trenches.
c) Subsurface infiltration trenches (1,800 ft^2) outfitted with chambers (Infiltrator® Water Technologies)
-) 4,500 gal septic tank with effluent screens and a 2,500 gal recirculation/dosing tank (not shown).

Fig. 8.43 Photographs taken during the installation of a textile media PMB serving a monastery in Colorado. (Photographs courtesy of SCG Enterprises, Inc.)

8.99

c.

a. b.

a) Delivery and off loading of AXmax textile media filters packaged in shipping containers (Orenco Systems®, Inc.).
b) AXmax textile media filters installed with one shown with a lid open to reveal the media inside.
c) Control room building.
d) Inside the control room building.

d.

Fig. 8.44 Photographs taken during installation of a textile media PMB serving a large mixed use service area in North Dakota. (Photographs courtesy Orenco Systems® Inc.)

Note: Alexander, North Dakota has an existing gravity collection system that serves around 1100 people, restaurants, offices, and other commercial facilities. The service area consists of around 50 % commercial and 50 % residential. The PBF design average daily flow is 300,000 gal/day

8.100

a. b. c.

a) Waterloo Biofilter® Basket system during installation at a residence (from left to right are septic tank, demand dosing pump chamber, concrete biofilter tank housing two Baskets containing foam filter media.
b) Waterloo Biofilter® shipping container tertiary treatment unit installed to treat 21,100 gal/d at a Provincial campground in Ontario, Canada – foam filter media visible through top access hatches.
c) Waterloo Biofilter® shipping container housing tertiary treatment unit, chemical dosing for phosphorus removal, and operator's room - installed to treat 4,000 gal/d at a public school in Ontario, Canada.

Fig. 8.45 Photographs of the Waterloo Biofilter® established in pre-manufactured baskets placed in tanks (**a**) and foam media placed in shipping containers (**b**, **c**). (Photographs courtesy of Waterloo Biofilter®)

8.101

- D&I considerations—Operation and maintenance
  - Routine operational inspections
    - Site inspections are required at least annually if not more often, with the first visit in the first month of operation
      - For larger PMB systems more frequent site visits are necessary
    - Basic inspection routine involves several elements
      - Inspect the septic tank (or other unit) and pump tank for obvious structural or operation problems
      - Check pump controls for proper operation
      - Read and record pump run-time meter and counters
      - Based on the recorded flow data, if needed adjust dosing to the PMB surface and reset the recirculation ratio
      - Check observation vents in the PMB for surface ponding
      - Check underdrain vents to ensure they are open to air

8.102

- Water quality testing
  - * Before flushing lines or servicing the system, field test for turbidity, DO, pH, NH_4^+-N
  - * If nitrification is expected, test for NO_3^--N and alkalinity too
  - * If turbidity is high, consider testing for BOD, TSS, and FOG
- Flush the pressurized piping network as needed

- Maintenance functions
  - Maintenance functions are highly variable and depend on the site conditions, PMB type, design, operation, and construction
  - Maintenance can include:
    - * Removing solids from dosing tanks (plus any other unit operations as applicable (e.g., septic tanks, aerobic units))
    - * Cleaning orifices or piping in pressure distribution networks
    - * Repair of piping in pressure distribution networks
    - * Raking, cleaning, or replacement of PMB media

8.103

8-5. Summary

- Porous media biofilters rely on biological treatment using attached growth bioprocesses
  - PMBs can utilize various media including sand, gravel, peat, foam, textile media, plastic beads, etc.
  - Treatment is primarily achieved by attached growth biological processes with some removal of solids by filtration
  - Small doses yield film flow over grain surfaces and biofilms
  - Aeration of the PMB bed typically occurs by passive means
  - The PMB surface area is determined by a design HLR_D and OLR, both of which can be set lower in colder climatic regions
- PMBs can produce advanced secondary effluents
  - Effluents are consistently very low in BOD_5, TSS, and NH_4^+ with appreciable removal of total N and pathogens as long as conditions support aerobic treatment and there is reliable O&M

8.104

8-6. Example Problems

- 8EP-1. Evaluating media suitability for a SPSF
  - Given information
    - A sieve analysis yielded the results shown in Table 8EP.1.
  - Determine
    - Based on grain size data, choose the best sand for a SPSF

Table 8EP.1 Data measured during a mechanical sieve analysis

Sieve size	Size (mm)	% Retained		
		Sand A	Sand B	Sand C
3/8	9.51	0	0	0
10	2.00	1	0.1	28
16	1.19	15	3.9	22
30	0.595	30	26	20
50	0.297	29	48	18
60	0.250	19	19	8
100	0.149	2	2.9	3
Pan		4	0.1	1

8.105

- Solution
  - Convert the %Retained data to % Passing data (Table 8EP.2)
    - * % passing = 100 %—% retained on a particular sieve plus all larger sieves

For example, for Sand A:

% passing no. 30 = 100 %—(%retained on no. 30 plus no. 16, 10, and 3/8 sieves)

% passing no. 30 = 100 %—(0 + 1 + 15 + 30) = 54 %

Table 8EP.2 Analysis of sieve data shown in Table 8EP.1

Sieve size	Size (mm)	% Passing		
		Sand A	Sand B	Sand C
3/8	9.51	100	100	100
10	2.00	99	99.9	72
16	1.19	84	96	50
30	0.595	54	70	30
50	0.297	25	22	12
60	0.250	6	3	4
100	0.149	4	0.1	4
Pan		0	0	0

8.106

- Plot %Passing data (Fig. 8EP.1) to determine D_{10} and D_{60} values

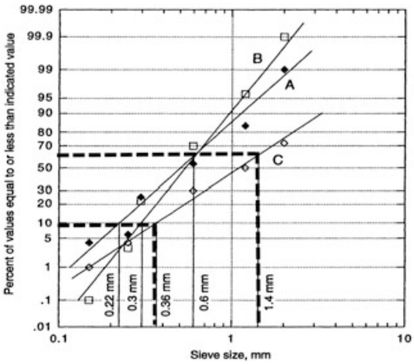


Fig. 8EP.1 Analysis of sieve data shown in Table 8EP.2

- Sand suitability for a SPSF based on typical specifications (Table 8.5)
  - B or C are okay, with the best choice depending on cost

8.107

- 8EP-2. Treatment efficiency of a single pass sand filter
  - Given information
    - A SPSF is being designed and an estimate of the BOD_5 removal efficiency is to be made for two HLR_D values and two bed depths as shown in Table 8EP.3
    - Assume the influent $BOD_5 = 220$ mg/L, $k_{20} = 0.05$ or $0.09\ h^{-1}$, and the PMB bed $n_e = 0.2$ and $T = 20\ °C$

Table 8EP.3 Hydraulic loading rate and depth conditions to evaluate

Parameter	Units	No.1	No.2	No.3	No.4
HLR_D	gal/day/ft^2	1	1	1.5	1.5
Depth	ft	1	2	1	2

  - Determine
    - The BOD_5 removal efficiency and effluent concentration for each of the conditions shown in Table 8EP.3

8.108

- Solution
  - Calculate the estimated %removal for the four conditions
    - * Example calculation for No. 1

$$\mathrm{HRT} = \frac{(\mathrm{d})(\mathrm{n_e})}{\mathrm{HLR}} \tag{8.5}$$

$$\mathrm{HRT} = \frac{(1\ \mathrm{ft})(0.2)}{\left(1\,\frac{\mathrm{gal/d}}{\mathrm{ft}^2}\right)\left(\frac{1\,\mathrm{cm/d}}{0.245\,\mathrm{gal/d/ft}^2}\right)\left(\frac{\mathrm{ft}}{30\,\mathrm{cm}}\right)\left(\frac{1\,\mathrm{d}}{24\mathrm{h}}\right)} = 35.28\,\mathrm{h}$$

$$\mathrm{R_E} = \left(1 - e^{-\mathrm{kt}}\right) \times 100\% = \left(1 - \mathrm{e}^{-(0.05)(35.28)}\right) \times 100\% \tag{8.6}$$

$$\mathrm{R_E} = (1 - 0.17) \times 100\% = 83\%$$

$$\mathrm{C_E} = (1 - 0.83) \times 220\,\mathrm{mg/L} = 38\,\mathrm{mg/L}$$

8.109

- Results for all four conditions appear in Table 8EP.4
  - * For this problem, $k = 0.05\ h^{-1}$ was used for all conditions
    - – To illustrate the effects k has on the estimate of R_E, $k = 0.09\ h^{-1}$ was also used for No.1 instead of $0.05\ h^{-1}$
    - – Note that k is affected by temperature and k would be lower at lower temperatures

Table 8EP.4 Results of analysis of four loading rate and depth conditions for a SPSF

Parameter	Units	SPSF conditions				
		No.1	No.2	No.3	No.4	No.1
Assumed k	h^{-1}	0.05				0.09
HLR_D	gal/day/ft^2	1	1	1.5	1.5	1
Depth	ft	1	2	1	2	1
HRT	h	35.3	70.6	23.5	47.0	35.3
R_E	%	83	97	69	90	96
C_E	mg/L	38	6	68	21	9

8.110

- 8EP-3. Design of a single pass sand filter
  - Given information
    - Complex of four homes in south Texas
    - $Q_A = 650$ gal/day; PF for maximum recurring day $= 1.5$
    - SPSF is treating domestic STE with $BOD_5 = 160$ mg/L
    - PMB surface area $HLR_D = 1.5$ gal/day/ft^2
    - Doses per day $= 12$ per day during 18 h under average flow
  - Determine
    - SPSF surface area required
    - Dosing volume and tank size

8.111

- Solution
  - Determine the size of the sand filter surface area

$$A_s = \frac{(Q_D)}{HLR_D} \tag{8.8}$$

$$A_S = \frac{(650\,\text{gal/day} \times 1.5)}{1.5\ \text{gal/day/ft}^2}$$

$$A_S = 650\ \text{ft}^2$$

  - Select a filter surface area geometry (assume only 1 zone)
    * A SPSF that is 27 ft long by 24 ft wide provides 648 ft^2 which is very close to the A_S calculated—✓okay

$$A_{S'} = 27\ \text{ft} \times 24\ \text{ft} = 648\ \text{ft}^2 \tag{8.9}$$

8.112

- Check the organic loading rate

$$\text{OLR} = \frac{(Q_D)(\text{BOD}_5)\left(8.34 \times 10^{-6}\right)}{A_S'} \tag{8.10}$$

$$\text{OLR} = \frac{(975\,\text{gal/day})(160\,\text{mg/L})\left(8.34 \times 10^{-6}\right)}{648\,\text{ft}^2}$$

$$\text{OLR} = 0.002 - \text{lb}\ \ \text{BOD}_5\ \ \text{per}\ \ \text{day}$$

✓ OLR is okay since it is less than the recommended limit of 0.005 lb BOD_5 per day

8.113

- 8EP-4. Design of a recirculating sand filter
  - Given information
    - A recirculating sand filter will serve an apartment building
    - $Q_A = 3500$ gal/day; peaking factor for RSF design = 1.5
    - Effluent applied to the RSF = domestic STE
    - RSF $HLR_D = 5$ gal/day/ft^2; Recirculation ratio = 4:1
    - Recirculation/dosing tank HRT = 1 day and sizing F = 0.8
    - Doses per day = 48; dosing during $T_D = 24$ h
  - Determine
    - RSF surface area required
    - Recirculation tank size
    - For one area with one versus two zones, layout the RSF area and distribution network and determine the RSF dose volume (Fig. 8EP.1)

8.114

- Solution
  - Determine the design flow rate
    - * $Q_A = 3500$ gal/day
    - * $Q_D = 3500 \times 1.5 = 5250$ gal/day
  - Determine the total surface area of the RSF

$$A_s = \frac{(Q_D)}{HLR_D} = \frac{(3500 \times 1.5)}{5} = 1050 \ \text{ft}^2 \tag{8.8}$$

  - Example RSF layouts to provide 1050 ft^2 of A_s

	1 zone	1 zone	2 zones
$L =$	32.4 ft	34 ft	34 ft ea.
$W =$	32.4 ft	32 ft	16 ft ea.
$A_s' =$	1050 ft^2	1088 ft^2	1088 ft^2

8.115

- Examples of two network layouts are shown in Fig. 8EP.2

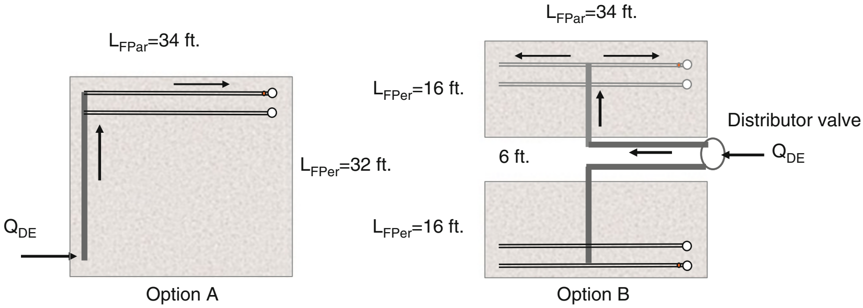


Fig. 8EP.2 Two network layouts that could be established to provide the same total PMB surface area but with different Q_{DE} and TDH values. (*Note:* Q_{DE} and TDH for Option A (*left*) are reduced with a zoned layout such as shown in Option B (*right*). Q_{DE} is reduced by 50 % and the TDH during dosing is reduced by approx. 50 % depending on the details of the network layout. Also for sake of simplicity, all the laterals included in the network are not shown in Figure 8EP.2

8.116

- Determine the recirculation tank size
  * Volume of the recirculation tank

$$V_{RT} = F(Q_D)(HRT) \tag{8.13}$$

$$V_{RT} = 0.8(3500\,\text{gal/day} \times 1.5)(1\,\text{day})$$

$$V_{RT} = 4200\ \text{gal}$$

- Determine the volume per dose
  * Compare the volume per dose for the two network options shown in Fig. 8EP.3

$$V_{DE} = \frac{(Q_A)(R+1)}{(N_S)(D_{PD})} \qquad V_{DE} = \frac{(Q_A)(R+1)}{(N_S)(D_{PD})} \tag{8.14}$$

$$V_{DE} = \frac{(3500)(4+1)}{(1)(48)} = 365\ \text{gal} \qquad V_{DE} = \frac{(3500)(4+1)}{(2)(48)} = 182\ \text{gal}$$

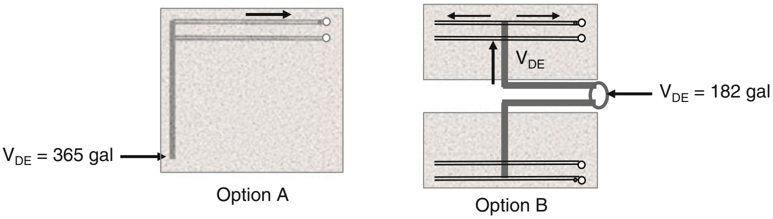


Fig. 8EP.3 The volume per dose for two optional network layouts. (*Note*: for sake of simplicity, all the laterals included in the network are not shown in Figure 8EP.3.)

- 8EP.5. Design of a RSF influent distribution system
  - Given information
    - A RSF comprised of a 2-zone layout (Fig. 8EP.4)
    - $A'_S = 1088\ ft^2$ provided in two, $544\ ft^2$ zones
    - $V_{RT} = 4200$ gal and $V_{DE} = 182$ gal/dose to a zone
    - The RSF unit is 50 ft from the recirculation/dosing tank and 10 ft higher in elevation

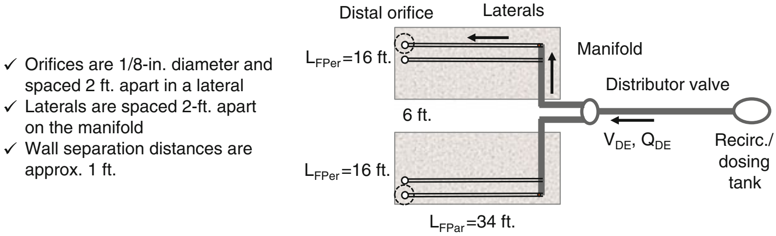


Fig. 8EP.4 Layout of a RSF with 2 zones. (*Note:* for sake of simplicity, all the laterals included in the network are not shown in Figure 8EP.4)

8.119

- Determine
  - The number of laterals and orifices for one zone
  - The orifice discharge rate and total dosing flow rate to a zone
  - The lateral headloss and network volume and check to be sure they are within recommended limits
  - The TDH for the network layout in one zone, and a suitable pumping unit
- Solution
  - Determine the lateral and manifold layout for a single zone
    - Number of laterals at 2 ft spacing on a center manifold with 1 ft wall separation

$$N_L = \left[\frac{(L_{FPer} - (S_W \times 2))}{S_L} + 1\right] = \left[\frac{(16 - (1 \times 2))}{2} + 1\right] = 8 \qquad (8.15)$$

8.120

* Length of laterals at 2 ft spacing on a center manifold with 1 ft wall separation

$$L_L = [L_{FPar} - (S_W \times 2)] \tag{8.16}$$

$$L_L = [34 - (1 \times 2)] = 32 \text{ ft}$$

* Determine the length of the manifold in each zone

$$L_M = [L_{FPer} - (S_W \times 2)] \tag{8.17}$$

$$L_M = [16 - (1 \times 2)] = 14 \text{ ft}$$

8.121

○ Determine the number of orifices in a lateral and in a zone
  * Number of orifices at 2-ft spacing with 1-ft manifold separation

$$N_{OLat} = \frac{(L_L - M_S)}{S_O} + 1 = \frac{(32 - 1)}{2} + 1 = 16.5; \quad \text{use } 17 \tag{8.18}$$

  * Number of orifices in a RSF zone

$$N_O = N_L \times N_{OLat} \tag{8.19}$$

$$N_O = 8 \times 17 = 136 \text{ orifices}$$

8.122

- Determine the discharge from an orifice
  - For the distal orifice (1/8-in. diameter) with 4 ft residual head:

$$Q_O = 2.45C\left(D^2\right)\sqrt{2gh_r} \tag{8.20}$$

$$Q_O = 2.45(0.63)(0.125)^2\sqrt{(2)(32.2)(4)}$$

$$Q_O = 0.39 \ \ \text{gal/min}$$

- Determine the flow rate into a lateral during a dosing event

$$Q_{Lat} = Q_o \times N_{OLat} \tag{8.21}$$

$$Q_{Lat} = 0.39\,\text{gal/min} \times 17\,\text{orifices/lateral} = 6.63\,\text{gal/min}$$

- Determine the total flow rate during a dosing event

$$Q_{DE} = Q_O \times N_O \tag{8.23}$$

$$Q_{DE} = (0.39 \ \ \text{gal/min})(136) = 53.0 \ \ \text{gal/min}$$

8.123

- Check the uniformity of distribution in a lateral
  - Determine the head loss in a lateral distribution pipe
    - Try 0.75-in. diam. Sch40 PVC (true ID = 0.824 in.)

$$h_{fl} = 0.33 \times \left[10.5(L)\left(\frac{Q_{Lat}}{C}\right)^{1.85}\left(D^{-4.87}\right)\right] \tag{8.24}$$

$$h_{fl} = 0.33 \times \left[10.5(16)\left(\frac{6.63}{130}\right)^{1.85}(0.824)^{-4.87}\right]$$

$$h_{fl} = 0.33 \times [168 \times 0.00406 \times 2.56]$$

$$h_{fl} = 0.33 \times [1.75]$$

$$h_{fl} = 0.58 \ \ \text{ft}$$

8.124

* Based on the lateral head loss, determine the difference in discharge between the first and last orifice in a lateral
  – Want ΔQ_O to be <10 %, which means R_Q >90 %

$$R_Q = \left(\frac{Q_{OF}}{Q_{OI}}\right) \times 100\% = \sqrt{\frac{h_r}{h_r + h_{fl}}} \times 100\% \qquad (8.25)$$

$$R_Q = \sqrt{\frac{4.0}{4.0 + 0.58}} \times 100\% \quad = 93.5\%$$

✓ Uniformity of orifice discharge in a lateral is very good

Note: If R_Q was <90 %, the network could redesigned (e.g., with smaller orifices or fewer orifices per lateral).

8.125

○ Check the fraction of a dose used to fill the manifold and laterals
  * Using 0.75-in laterals with a 1.5-in. Sch40 PVC manifold with $V_{DE} = 182$ gal

$$R_{FV} = \left[\frac{0.785\left[(D_M)^2 L_M + (D_L)^2 (N_L \times L_L)\right]\left(\frac{1\text{ft}^2}{144\text{in}^2}\right)\left(\frac{7.48\text{gal}}{\text{ft}^3}\right)}{V_{DE}}\right] \times 100\ \% \qquad (8.26)$$

$$R_{FV} = \left[\frac{0.785\left[(1.61)^2 14 + (0.824)^2 (8 \times 32)\right]\left(\frac{1\text{ft}^2}{144\text{in}^2}\right)\left(\frac{7.48\text{gal}}{\text{ft}^3}\right)}{182}\right] \times 100\ \%$$

$$R_{FV} = \left[\frac{8.6\ \text{gal}}{182\ \text{gal}}\right] \times 100\ \% = 4.7\ \% \qquad ✓R_{FV}\ \text{is}\ 20\ \%$$

Note: If R_{FV} were >20 %, the network could be redesigned or if climate conditions are suitable, you could use upward/downward orifice configuration to enable drainage.

8.126

- Determine the total dynamic head during dosing

$$\mathrm{TDH} = (h_s + h_{md} + h_{dv} + h_r) + (h_{fl} + h_{fm} + h_{ftp}) \quad (8.27)$$

- * Given and assumed values
  - $h_s = 10$ ft (based on elevation difference)
  - $h_{md} = 3$ ft (assumed)
  - $h_{dv} = 15$ ft (assumed)
  - $h_r = 4$ ft (selected)
- * Previously calculated lateral headloss
  - $h_{fl} = 0.58$ ft
- * Calculate the headloss in the manifold and transport piping

8.127

- * Determine the head loss in the manifold piping
  - For 1.5-in. diam. Sch40 PVC (true ID = 1.61 in.)
  - $L = 14$ ft, $Q_{DE} = 56.2$ gal/min, $Q_M = Q_{DE}$

$$h_{fm} = 0.33 \times \left[10.5(L)\left(\frac{Q_M}{C}\right)^{1.85}(D^{-4.87})\right] \quad (8.28)$$

$$h_{fm} = 0.33 \times \left[10.5(14)\left[\frac{(56.2)}{130}\right]^{1.85}(1.61)^{-4.87}\right]$$

$$h_{fm} = 0.33 \times [147 \times 0.211 \times 0.098]$$

$$h_{fm} = 0.33 \times [3.04]$$

$$h_{fm} = 1.00 \ \text{ft}$$

8.128

* Determine the head losses in the transport piping
  – Try 2.0-in diam. Class 200 PVC pipe (true ID = 2.149 in.)
  – L = 50 ft and Q_{DE} = 56.2 gal/min

$$h_{ftp} = \left[10.5(L)\left(\frac{Q_{DE}}{C}\right)^{1.85}\left(D^{-4.87}\right)\right] \tag{8.29}$$

$$h_{ftp} = \left[10.5(50)\left[\frac{(56.2)}{130}\right]^{1.85}(2.149)^{-4.87}\right]$$

$$h_{ftp} = 525 \times 0.211 \times 0.0241$$

$$h_{ftp} = 2.67 \text{ ft}$$

8.129

* The TDH is equal to the sum of the individual head components:

$$TDH = (h_s + h_{md} + h_{dv} + h_r) + \left(h_{fl} + h_{fm} + h_{ftp}\right) \tag{8.27}$$

$$TDH = (10 + 3 + 15 + 4) + (0.58 + 1.00 + 2.67)$$

$$TDH = 36.2 \text{ ft}$$

○ Determine a suitable pumping system
  * A suitable pumping system would have to be able to deliver a dosing event flow rate of Q_{DE} = 56.2 gal/min against a TDH = 36.2 ft

8.130

- Select a suitable pumping unit
  * Example of a pump capable of delivering the Q_{DE} of 56.2 gal/min against the 36.2 ft TDH of the system

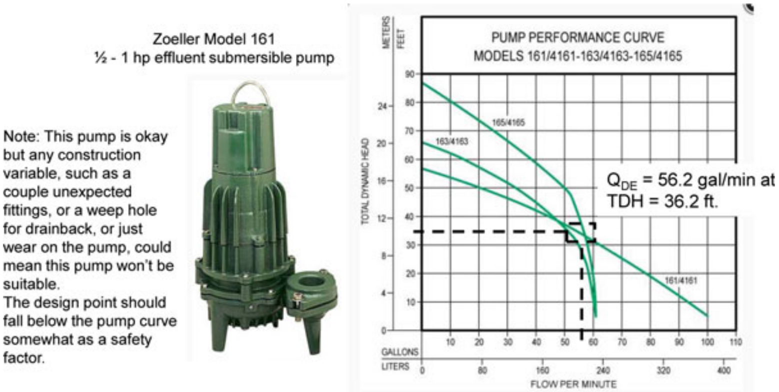


Fig. 8EP.5 Pump performance curve for a Zoeller Model 161 submersible pump (www.zoellerpumps.com/ProductBenefit.aspx?ProductID=95)

8.131

- Calculations for one pump and timed dosing of the RSF

$$P_{on} = \frac{V_{DE}}{Q_{DE}} \tag{8.30}$$

$$P_{on} = \frac{182 \ \text{gal/dose}}{56.2 \ \text{gal/min}} = 3.24 \text{ min}$$

$$P_{off} = \left[\frac{T_D}{N_Z \times D_{PD}}\right] - P_{on} \tag{8.31}$$

$$P_{off} = \left[\frac{24 \ \text{h} \times 60 \ \text{min/h}}{2 \ \text{zones} \times 48 \ \text{doses/day}}\right] - 3.24 = 11.8 \text{ min}$$

  * There are 48 doses/day to each of two zones with each cycle being 15 min. long; the pump is on for ~3.2 min. and then off for ~11.8 min.

8.132

- 8EP.6. Compare the surface area of different PMBs
  - Given information
    - Apartment complex
    - $Q_A = 650$ gal/day; peaking factor for PMB design $= 1.5$
    - Effluent applied to the PMB = domestic septic tank effluent
    - HLR_D (gal/day/ft^2): SPSF = 1.50; RSF = 5; PBF = 25
  - Determine
    - The surface area required for each of three PMB types: single pass sand filter (SPSF), recirculating sand filter (RSF), and a textile media packaged biofilter (PBF)

8.133

- Solution
  - The area of each PMB type was determined using Eq. 8.8

$$A_S = \frac{(Q_D)}{HLR_D} \tag{8.8}$$

  - The results are summarized in Table 8EP.5 and illustrated in Fig. 8EP.6

Table 8EP.5 Comparison of the total area required by each of three PMBs

Parameter	SPSF	RSF	Textile media PBF
Hydraulic loading rate (gal/day/ft^2)	1.5	5	25
Design flow for A_S (gal/day)	$Q_D = 650 \times 1.5 = 975$		
PMB surface area (ft^2)	650	195	39

8.134

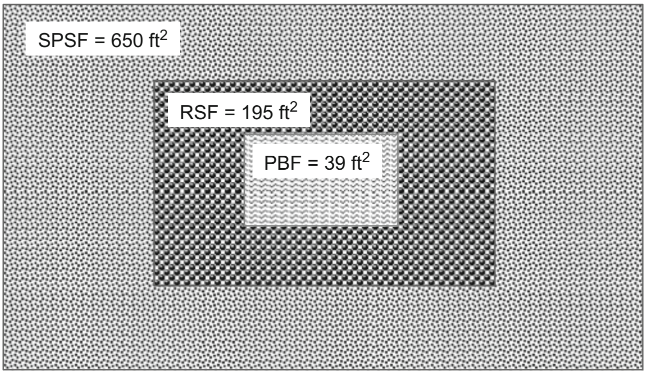


Fig. 8EP.6 Comparison of the total PMB surface area required to handle a design flow of 975 gal/day

8.135

Chapter 9

Treatment Using Membrane Bioreactors

9-1. Scope

Membrane bioreactors were developed as a hybrid treatment technology combining a membrane filtration unit with an activated sludge bioreactor. The membrane unit serves to separate and retain biomass in the activated sludge process instead of a secondary clarifier and also achieves solids separation through micro- and ultrafiltration processes. Membrane bioreactors are capable of producing tertiary quality effluents with disinfection to enable surface discharge and water reuse applications. This chapter describes the principles and processes that are involved in membrane filtration processes and the design and implementation of membrane bioreactors.

9-2. Key Concepts

- Membranes can be used in filtration processes to remove particles, molecules, ions and other constituents from water.
  - There are several types of membrane filtration processes based on the size range of the constituents they are typically used for removal of.
    - Particle filtration—Used for removal of micro- and macro-particles that are larger than 1 μm in size (e.g., bacteria, protozoan cysts, pollen, human hair).
    - Microfiltration—Used for removal of micro-particles and macro-molecules that are in the size range of 0.05–2.0 μm (e.g., bacteria, asbestos, paint pigments).

R.L. Siegrist, *Decentralized Water Reclamation Engineering*,
DOI 10.1007/978-3-319-40472-1_9

- Ultrafiltration—Used for removal of molecules and constituents that are in the size range of 0.003–0.1 μm (e.g., viruses, proteins, colloidal silica).
- Nanofiltration—Used for removal of molecules and constituents in the size range of 0.0008–0.006 μm (e.g., viruses, sugars, dyes).
- Reverse osmosis—Used for removal of ions in the size range of 0.0001–0.001 μm (e.g., metal ions, aqueous salts).

- Membrane filtration processes can be used in unit operations for advanced treatment of wastewaters and other impaired waters (e.g., contaminated groundwater).

■ A membrane bioreactor (MBR) combines a membrane filtration process with a suspended growth biological treatment process.

- There are different configurations of MBRs, but aerobic MBRs are common and typically include an anoxic zone followed by an aerobic zone that is connected to an aerated compartment where submerged membrane units are placed.
- Membrane units used in MBRs typically rely on ultrafiltration and microfiltration to separate constituents that are about 0.04 μm or larger in size.
  - The membrane unit provides for biomass separation and retention in a compartment connected with the tank in which aerobic treatment occurs.
  - The membrane unit can also achieve removal of molecular and colloidal constituents and most pathogenic microorganisms.

■ MBRs are more capital intensive than conventional aerobic treatment systems, but MBRs can produce a higher quality effluent and require less space, both of which can be required if discharge to sensitive waters or nonpotable reuse is planned and where landscape area is limited or very expensive.

■ The influent to a MBR is commonly primary effluent such as the effluent from an upstream primary settling tank or septic tank. MBRs can also be designed to receive raw wastewater after fine screening (e.g., 1–2-mm size screen opening).

■ Membranes used in MBRs can be made of different materials to balance criteria such as separation efficiency, inertness, resistance to fouling, amenability to cleaning, and cost.

- The membrane units typically operate in the microfiltration or ultrafiltration range (e.g., 0.4–0.04 μm size range) and operate under transmembrane pressures (TMP) with an outside-to-inside flow path.
- MBR effluent is the permeate flux that passes through the membrane and leaves the MBR. Permeate flux rates can average 10–15 gal/day/ft^2 with short-term peak fluxes as high as 18–30 gal/day/ft^2.

- MBRs normally produce tertiary quality effluents with disinfection if they are correctly designed and properly operated and maintained.
  - The MBR effluent should be of consistent high quality with BOD_5 <2 mg/L, TSS <1 mg/L, and NH_4-N <2 mg-N/L. With recirculation of aerobic tank biomass to the influent, 50–85 % removal of total nitrogen is achievable.
  - Protozoan cysts, bacteria, and most viruses should be absent (based on membrane filtration process features).
  - Depending on site-specific conditions and regulatory requirements, MBR effluent can be surface discharged or used for toilet flushing, cooling, irrigation or other beneficial uses.

- The MBR design process typically includes selection of a design solids retention time (SRT) based on the influent wastewater characteristics, the temperature, and the need for nitrification. These choices determine the net solids production rate (Y_N). Then the hydraulic retention time (HRT) can be determined along with the average rate of wasting sludge from the MBR (Q_W) to maintain a target SRT and mixed liquor volatile suspended solids level (MLVSS).

- Several companies commercially manufacture MBRs in packaged configurations. The MBR features vary based on membrane type, orientation, and bioreactor connection.

- Due to their complicated electrical and mechanical components (e.g., pumps, valves, aerators, membrane modules, controls), MBRs require routine and reliable operation and maintenance (O&M).
  - In particular, beyond the typical O&M of an aerobic treatment unit, MBRs have O&M requirements associated with the membrane unit.
  - Membranes are subject to fouling due to adsorption or deposition of biopolymers or inorganic scales. Fouling can affect TMP levels required for constant permeate production.
  - Fouling can be hindered by pretreatment of the feed water to the membrane, backflushing the membrane unit with water/air, relaxation of the membranes by cyclic operation, and chemical cleaning.
  - Depending on attributes and usage, membranes can require periodic replacement.

- In a MBR, just like in other aerobic treatment methods, excess solids accumulate and need to be removed periodically and properly managed as described in Chap. 15.

9-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 9 are presented in the Slides section.

9-4. Terminology

Terminology introduced and used in Chap. 9 is defined below.

Activated sludge—A biological process where microorganisms are grown under aerobic conditions using organic matter in the influent wastewater as a source of food and energy.

Aeration zone—A term that describes the physical system within which active aeration occurs. Examples include an aeration chamber or compartment in a larger tank, a stand-alone tank of basin devoted to aeration, and so forth.

Aerobic—Refers to a biochemical state where microorganisms require oxygen to survive and function by using oxygen as an electron acceptor.

Aerobic treatment unit (ATU)—Refers to a physical system of compartments, tanks or basins used to establish an aerobic bioreactor and the supporting components and appurtenances used to achieve aerobic biological treatment of wastewater. Aerobic treatment unit (ATU) may also be used to refer to a small-scale packaged plant used for aerobic biological treatment of wastewater.

Biomass—Biological material derived from living microorganisms involved in biological wastewater treatment.

Biosolids—Biosolids refers to treated sewage sludge that is made suitable for beneficial use through incorporation into soil and agriculture. In the United States the U.S. Environmental Protection Agency sets pollutant and pathogen requirements for biosolids relative to use for land application and surface disposal in Federal regulation 40 CFR Part 503, which sets standards for the use or disposal of sewage sludge.

Clean in place (CIP)—Refers to a method to maintain a high flux rate through a membrane without having to remove it from the membrane bioreactor.

Denitrification—Denitrification involves the reduction of nitrate to N_2O and N_2 gas under anoxic conditions (DO $<$ 0.5 mg/L) by heterotrophic bacteria (different anaerobic and facultative bacteria) that utilize organic matter as a source of energy and organic carbon. Denitrification can also be carried out under anoxic conditions by autotrophic bacteria (*Thiobacillus denitrificans* and *Thiomicrospira denitrificans*) that can use sulfur as an electron donor and NO_3^- as an electron acceptor.

Flux—(1) The rate of liquid or mass flow through a membrane given in units of volume per time per surface area (e.g., gal/day/ft^2, lb/day/ft^2). (2) The rate of liquid or mass flow through a horizontal or vertical plane in the subsurface given in units of volume per time per surface area (e.g., gal/day/ft^2, lb/day/ft^2).

Fouling—A term that refers to the process of pore plugging or surface coating of a membrane or other filter material that reduces the rate of liquid flow through the membrane.

Hydraulic retention time (HRT)—(1) In the context of confined treatment operations the hydraulic retention time is a design parameter that describes how long liquid remains in a specified chamber, basin, or tank. HRT is defined as the volume of the chamber, basin, or tank (e.g., gal) divided by the flow rate passing it through it (e.g., gal/day).

Membrane bioreactor (MBR)—A unit operation for wastewater treatment that combines an aerobic bioreactor with a membrane unit for biomass separation and retention.

Microfiltration—Filtration though a membrane that is used for removal of micro-particles and macromolecules that are in the size range of 0.05–2.0 μm (e.g., bacteria, asbestos, paint pigments).

Mixed liquor—A term used to refer to the contents of the aeration zone (compartment, tank, basin) within an activated sludge biological treatment system.

Mixed liquor volatile suspended solids (MLVSS)—A measure of the biomass cells in the mixed liquor within an aeration zone (i.e., a compartment, tank, or basin).

Nanofiltration—Filtration though a membrane that is used for removal of molecules and constituents in the size range of 0.0008–0.006 μm (e.g., viruses, sugars, dyes).

Nitrification—Nitrification involves the conversion of ammonia nitrogen to nitrite and nitrate nitrogen under aerobic conditions by autotrophic bacteria that utilize O_2 as an electron acceptor and CO_2 as a carbon source.

Solids retention time (SRT)—A term that describes the length of time that solids produced during biological wastewater treatment are retained in the aeration zone. SRT is equal to the total mass of mixed liquor volatile suspended solids in the aeration zone divided by the mass of volatile suspended solids wasted from the system.

Soluble microbial products (SMP)—A mix of organic materials that are produced by microorganisms during growth when they are degrading organic materials in an activated sludge process.

Substrate—A term that describes the organic matter that microorganisms involved in biological treatment use as a source of organic matter and organic carbon.

Suspended growth—Refers to an aerobic biological process where the microorganisms involved in treatment are suspended in the liquid wastewater.

Trans-membrane pressure (TMP)—Refers to the pressure required to drive liquid flow through a fine pore membrane.

Ultrafiltration—Filtration through a membrane that is used for removal of molecules that are in the size range of 0.003–0.1 μm (e.g., viruses, proteins, colloidal silica).

Yield (Y_N)—The net production of solids during aerobic biological treatment that is typically based on the characteristics of the wastewater being treated, the solids retention time, and the temperature.

9-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 9 are listed below.

BOD	Biochemical oxygen demand
CIP	Clean in place
DO	Dissolved oxygen
HRT	Hydraulic retention time
MBR	Membrane bioreactor
MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids
OM	Organic matter
PF	Peaking factor
SMP	Soluble microbial products
SRT	Solids retention time
TMP	Trans-membrane pressure
TSS	Total suspended solids
VSS	Volatile suspended solids

A_C	Area provided by each unit
A'_M	Approximate total surface area of membranes required
A_M	Final total surface area of membranes required
C_E	Effluent concentration
C_I	Influent concentration
F_M	Design membrane flux limit
F'_M	Membrane flux producing effluent from the MBR
F_C	Membrane flux used for cleaning
N'_C	Estimated number of units required
N_C	Number of units required
Q_A	Average daily flow rate
Q_C	Daily permeate flow used for cleaning each unit
Q_D	Design daily flow rate
Q_E	Effluent flow rate
Q_I	Influent daily flow
Q_P	Permeate produced

Q_R	Recycle flow rate
Q_W	Average daily flow of waste solids from the MBR
R	Recycle ratio (–)
R_E	Removal efficiency
S_I	Influent substrate (BOD_5 or COD) concentration
V_A	Volume of the aerobic tank
V_M	Volume of the tank with the membrane module in it
X_A	Mixed liquor VSS in the aerobic tank
X_E	Effluent concentration of VSS
X_M	VSS concentration in the tank with the membrane in it
Y_N	Net solids production

9-6. Problems

9.1. In a membrane bioreactor the membrane module can be viewed as replacing what component of an aerobic treatment unit that relies on a suspended growth activated sludge process?

9.2. Membrane bioreactors rely on a membrane module for separating biomass solids and keeping them in the bioreactor. Which of the following best describes the pore size of the membranes used: 3 mm, 100 μm, or 400 nm?

9.3. Which of the following do membrane bioreactors require in order to function properly (check all that apply): (1) source of power, (2) clarification basin to settle solids and return them to the bioreactor, (3) method to properly handle solids that are wasted from the bioreactor?

9.4. Membrane bioreactors represent a tertiary treatment unit operation that can consistently produce a very high quality effluent. Which one of the following statements best represents the effluent quality from a MBR?

(a) BOD_5 and TSS = 30 mg/L *E. Coli* bacteria = 1000 organisms per 100 mL

(b) BOD_5 and TSS = 5 mg/L *E. Coli* bacteria = 200 organisms per 100 mL

(c) BOD_5 and TSS < 2 mg/L *E. Coli* bacteria < 2 organisms per 100 mL

9.5. What is the total flow rate capacity (in gal/day of permeate) of a membrane unit that has four cassettes, each of which has 300 ft^2 of membrane surface area and an average flux rate of 15 gal/day/ft^2?

9.6. A membrane bioreactor is being designed to serve a subdivision development and treat 15,000 gal/day of wastewater to produce effluent for irrigation of a local golf course. The MBR being designed includes a series of submerged membrane cassettes within the bioreactor chamber (HRT = 15 h, SRT = 15 days) and daily cleaning of the cassettes recycles 2000 gal/day back to the bioreactor. How much MBR effluent will be available for irrigation purposes (in gal)?

9.7. A membrane bioreactor is being designed to serve a subdivision development with a maximum daily flow of 25,000 gal/day. The MBR will be used to produce effluent for irrigation of a local golf course. The MBR includes a series of submerged membrane cassettes within a bioreactor chamber. Calculate the following: (1) volume of the aeration tank. (2) hydraulic retention time (h). (3) irrigation water produced (gal/day). (4) irrigation water available if the permeate return for cleaning has to be increased from 1000 to 2000 gal/day?
Given information and assumed values: Influent $BOD_5 = 250$ mg/L. Target MLVSS = 8000 mg/L. SRT = 30 days and $Y_N = 0.39$ lb-VSS/lb-BOD. Total amount of permeate return for cleaning = 1000 gal/day.

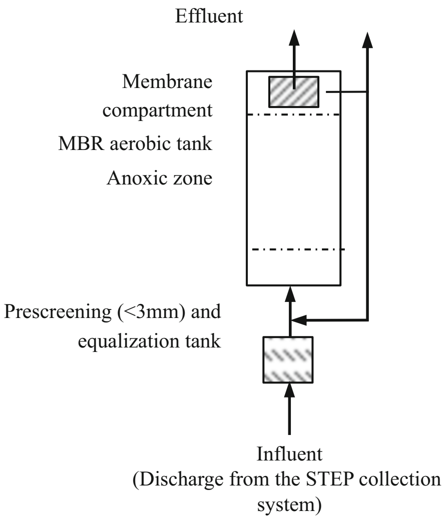
9.8. Check which one of the following is the most plausible cause of why the effluent from a membrane bioreactor might abruptly change with *E. Coli* bacteria levels rising from <2 to >1000 organisms per 100 mL: (1) increased bacteria levels in the bioreactor, (2) fouling of the membrane surface, (3) a tear in the membranes in one of the membrane units, or (4) increased return of permeate for membrane cleaning?

9.9. A membrane bioreactor can be operated with infrequent removal of biomass solids. Which one of the following statements best explains why: (1) MBRs can use very high concentrations of MLVSS and long SRTs which leads to low net yields of biomass or (2) the membranes in an MBR can filter out particulates as small as 400 nm in size?

9.10. Is the effluent from a membrane bioreactor more consistent in concentrations of BOD_5 and TSS compared to that from an aerobic treatment unit and if so, briefly explain why?

9.11. For the Mines Park housing development located on the Colorado School of Mines campus in Golden, Colorado you are tasked with preliminary design of a membrane bioreactor (MBR). The MBR should be capable of producing a high quality effluent (BOD_5, COD, and TSS ≤ 5 mg/L and *E. coli* <2/100 mL) that should be suitable for reuse for lawn and garden irrigation around Mines Park. Based on the information given below, answer the following design questions. (1) What is the volume (gal) of the MBR aerobic tank needed to handle the design daily flow and what geometry would you propose (length and width in ft)? (2) What is the HRT (h) and is it within the range of typical values for a MBR? (3) What is the rate of wasting (gal/day) required to maintain the target MLVSS? (4) How much MBR effluent would be produced and available for irrigation on an average flow day? (5) What is the membrane area required (ft^2) and how many membrane units will be needed? (6) What is the flux rate (gal/day/ft^2) under maximum daily flow conditions and is this acceptable?

Given information and assumed values: The average daily flow, $Q_A = 28{,}425$ gal/day. A STEP collection system will convey septic tank effluent (STE) to the treatment site. The STE quality expected: COD = 220 mg/L, BOD_5 = 160 mg/L, TSS = 80 mg/L, TKN = 60 mg/L. The design flow (Q_D) equals the maximum recurring daily flow (PF = 2). The average wastewater temperature = 15 °C and the target SRT = 30 days with $Y_N = 0.39$ lb-VSS/lb-BOD removed. The target MLVSS in the MBR aeration tank, $X_A = 8000$ mg/L and the recycle ratio, R = 4. The membrane compartment tank volume, $V_M = 20\,\%$ of the MBR aerobic tank volume, V_A. Each membrane unit has 300 ft^2 of surface and a design flux rate under average conditions = 15 gal/day/ft^2 with a maximum tolerable rate for peak flow = <25 gal/day/ft^2. The permeate returned for cleaning amounts to about 300 gal/day per membrane unit. The MBR aerobic tank will have a water depth = 10 ft and L:W ratio = approx. 3:1.



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[1]References cited in Chap. 9 are listed along with other references that have content relevant to the topics covered in Chap. 9.

Slides of Chapter 9

Decentralized Water Reclamation

Chapter 9: Treatment Using Membrane Bioreactors

Contents

9.1

9-1. Introduction

- Membrane processes
  - Membrane filtration processes can be used in unit operations for advanced treatment of wastewaters and other impaired waters
  - The types of processes and relevant size ranges include the following:
    - Particle filtration: >1 μm
    - Microfiltration: 0.05–2.0 μm
    - Ultrafiltration: 0.003–0.1 μm
    - Nanofiltration: 0.0008–0.006 μm
    - Reverse Osmosis: 0.0001–0.001 μm
  - Table 9.1 summarizes the size ranges and gives examples of materials in each range

9.2

Table 9.1 Particles, molecules and ions can be removed by different filtration processes

Process	Size range (μm)		Example materials in the size range
Particle filtration	>1.0	Micro- to macro-particle range	Protozoan cysts, pollen, human hair
Microfiltration	0.05–2	Macromolecular range	Bacteria, asbestos, paint pigments
Ultrafiltration	0.003–0.1	Molecular to macro-molecular range	Virus, proteins, colloidal silica
Nanofiltration	0.0008–0.006	Molecular range	Sugars, pesticides, herbicides
Reverse osmosis	0.0001–0.001	Ionic range	Metal ions, aqueous salts

9.3

- Membrane bioreactors
  - A membrane bioreactor (MBR) is a bioreactor which has an integral membrane filtration unit
  - A common MBR technology is essentially a suspended growth activated sludge process in which the secondary clarification process is replaced by a membrane filtration process
  - By inclusion of micro- to ultrafiltration membranes with pore sizes as small as about 40 nm, MBRs can also achieve advanced separation of pollutants and pathogens
  - MBRs can treat raw wastewaters (after screening) or primary effluents
  - MBRs can achieve tertiary treatment with disinfection and produce a very consistent, high quality effluent

9.4

- Basic features of a MBR
  - A simplified schematic of a typical aerobic MBR with basic components is shown in Fig. 9.1
    - Aerobic MBRs can have different configurations than the one illustrated in Fig. 9.1 and anaerobic MBRs are also being developed

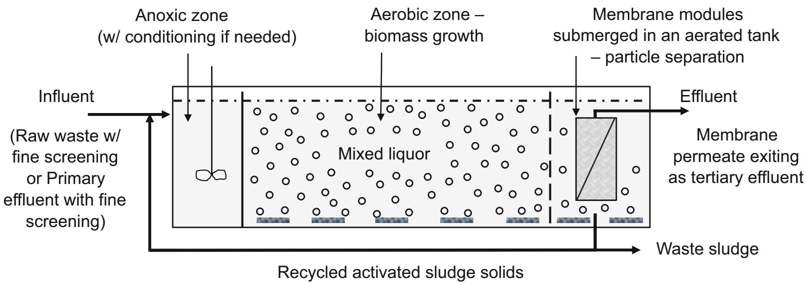


Fig. 9.1 Cross-section view of a representative aerobic membrane bioreactor

9.5

- Figure 9.2 shows an example of a non-MBR equivalent
  - *Note:* this illustration includes an aerobic treatment unit with clarification combined with an ultrafiltration unit

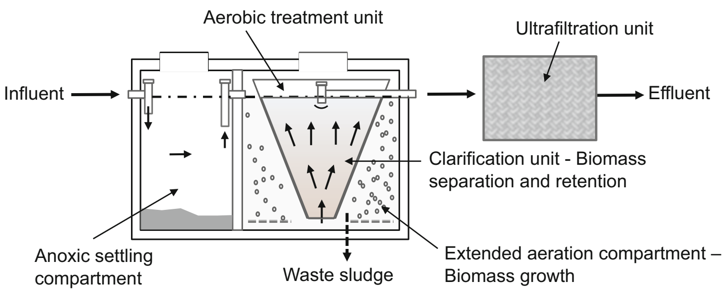


Fig. 9.2 Cross section of an aerobic treatment unit combined with an ultrafiltration unit to yield process function and performance similar to a MBR such as shown in Fig. 9.1

9.6

- MBRs for smaller decentralized applications can be “packaged” just like aerobic treatment units
  - Examples of small-scale MBRs appear in Figs. 9.3 and 9.4
    - Bio-Microbics BioBarrier MBR—500–9000 gal/day
    - Huber smartMBR—2600–19,800 gal/day
    - Busse-GT MBR—250–2000 gal/day

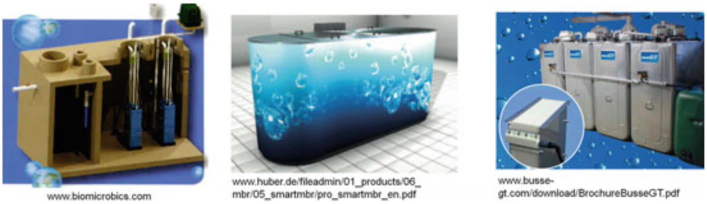
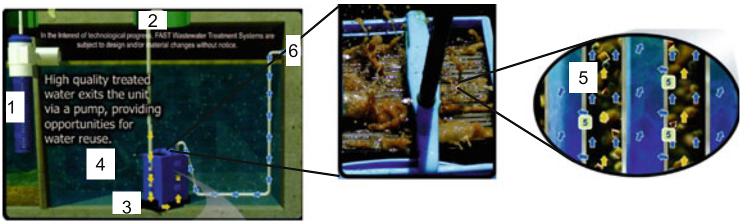


Fig. 9.3 Examples of three commercially packaged small-scale MBRs. *Source*: www.biomicrobics.com, www.huber.de/fileadmin/01_products/06_mbr/05_smartmbr/pro_smartmbr_en.pdf, www.busse-gt.com/download/BrochureBusseGT.pdf

9.7



1. Primary treatment with fine screen
2. Above-ground blower
3. Air grid under membrane cartridges
4. Aeration zone
5. Micro-and ultrafiltration membranes
6. MBR effluent

Effluent quality:
BOD < 2 mg/L
TSS < 2 mg/L
Ammonia < 1 mg/L
E. Coli < 10 CFU/L

Fig. 9.4 Design features of the Bio-Microbics, Inc. Biobarrier® MBR (www.biomicrobics.com)

9.8

- Where are MBRs used?
  - For use in decentralized infrastructure where one or more of the following apply:
    - Very high quality effluent is required for discharge
    - High strength wastewaters require advanced treatment to enable discharge options
    - Footprint area limitations exist
      - Area is very limited and not sufficient for other unit operations and systems
      - Land area is very expensive
    - The goal is to reclaim effluent water for toilet flushing, turf irrigation, cooling, or ornamental fountains
    - Operation and maintenance requirements can be reliably accomplished

9.9

9-2. Treatment Performance

- A membrane bioreactor can be designed to achieve tertiary treatment with disinfection
  - A MBR can consistently yield a very high quality effluent
    - Benefits from flexibility in biological treatment operations
    - Particle separation in micro- or ultrafiltration processes
  - Compared to a conventional activated sludge aerobic treatment unit, a MBR improves the removal efficiency for solids, organics, and nutrients
  - A MBR can also achieve high removals of pathogenic microorganisms (protozoa, bacteria, virus) and some trace organic chemicals depending on membrane pore size and material properties

9.10

- Treatment efficiency
  - Treatment efficiency is illustrated in Fig. 9.5 and can be calculated using Eq. 9.1

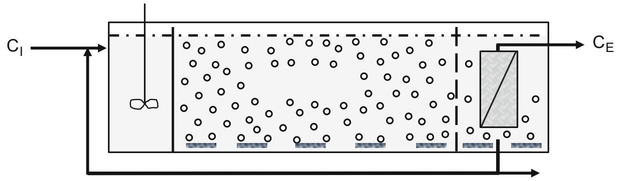


Fig. 9.5 Illustration of treatment efficiency achieved within a membrane bioreactor

$$R_E = \left(\frac{C_I - C_E}{C_I}\right) \times 100\% \qquad (9.1)$$

Where:
R_E = removal efficiency (%)
C_I = influent concentration (mg/L)
C_E = effluent concentration (mg/L)

9.11

- Membrane bioreactor treatment efficiencies for constituents of potential concern are presented in Table 9.2.

Table 9.2 Representative treatment efficiency achieved within a well designed and operated MBR

Constituent group	Effluent mg/L or % removal	Potential processes involved in treatment
BOD_5	<2 mg/L	Dissolved, colloidal and particulate organics are converted to cell mass and CO_2 which can be separated from the effluent
TSS	<1 mg/L >99 %	TSS in the form of colloidal and particulate solids are filtered out and separated from the effluent
Nitrogen[a]	<2 mg N/L NH_4^+ 50–85 % total N	Biological nitrification of NH_4^+ compounds to NO_3^- with removal of total N by biodenitrification
Phosphorus	10–20 % total P	Incorporation of P into cell mass and sorption
Pathogens	>99.99 %	Membrane filtration, die-off and inactivation
Trace organics	0 to >90 %	Near zero removal for some compounds but up to 90 % or more removal of compounds that are susceptible to aerobic biodegradation, biosolids sorption, or membrane rejection

[a]To achieve high removal efficiencies for NH_4^+ and total nitrogen, it is important to have adequate alkalinity in the influent for nitrification and recirculation of mixed liquor back to an anoxic zone for biodenitrification.

9.12

- MBR effluent composition
  - Factors affecting treatment efficiency and effluent composition
    - Sizing of the bioreactor component appropriate for the influent quality (C_I), design flow rate, and temperature conditions
    - Maintaining conditions in the bioreactor that are conducive to biological treatment
      - Adequate aeration to maintain dissolved oxygen (DO) and achieve mixing
      - Adequate pH for nitrification
      - Absence of toxic conditions (e.g., quaternary ammonium salts, dewaxer chemicals)
    - Provision of adequate membrane area to handle the design flow at a design flux rate the membrane can handle
    - Avoidance of excessive membrane fouling and rupturing
      - Sufficient SRT for nitrification that also limits biofouling
      - Provision of routine cleaning and replacement as needed

9.13

9-3. Principles and Processes

- Aerobic MBRs can be implemented in different configurations but the following elements are common
  - Influent wastewater to the MBR
    - Raw wastewater after fine screening (e.g., 1–2 mm typ.)
    - Primary treated wastewater (e.g., septic tank effluent)
  - Initial mixed zone
    - There can be an initial mixed anoxic zone that also provides some flow equalization
    - A chemical conditioning step can be used if needed (e.g., pH or alkalinity adjustment)
  - Bioreactor zone
    - Wastewater is then treated in an activated sludge process

9.14

- Activated sludge treatment occurs in an aerobic zone
  - The membrane enables 100 % retention of biomass solids and there are no issues with solids settleability in a clarifier
  - Thus the design and operating parameters for the bioreactor in the MBR can be targeted at values outside the range of conventional activated sludge
  - Example MBR operating parameters appear in Table 9.3

Table 9.3 Example operating parameters for a MBR

Parameter	MBR	Conventional activated sludge with clarifier
Hydraulic retention time (h)	2 to >12	24–120
Solids retention time (days)	8–40	20–40
Aerobic zone MLVSS (mg/L)	8000–12,000	2000–4500

Note: these parameter values are given as examples only and do interact so they can not be set independently.

9.15

- Membrane separation and biomass retention
  - The mixed liquor in the aerobic zone is processed by a membrane filtration unit
  - Membranes in MBRs typically operate in the microfiltration and ultrafiltration range (Table 9.4)

Table 9.4 Membrane filtration processes typically employed in MBRs

Process	Size range (μm)		Example materials in the size range
Microfiltration	0.05–2	Macromolecular range	Bacteria, asbestos, paint pigments
Ultrafiltration	0.003–0.1	Molecular to macromolecular range	Virus, proteins, colloidal silica

9.16

- Membrane features
  - Materials from which membranes are made are important to satisfying criteria including:
    - Separation efficiency, inertness, resistance to fouling, and amenability to cleaning
  - Membranes are manufactured from various materials
    - Organic membranes
      - Polyethylene, polyethersulfone, polysulfone, polyolefin, cellulose acetate, polyamides, etc.
    - Inorganic membranes
      - Metallic
      - Ceramic

9.17

- For MBR applications, membranes can be manufactured as hollow fibers grouped in bundles or as membrane sheets assembled in spiral wound or flat plates (Fig. 9.6)

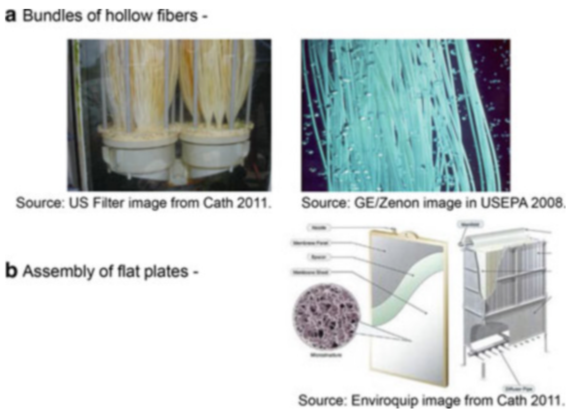
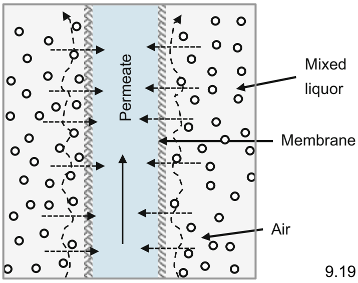


Fig. 9.6 Membranes as (**a**) fibers grouped in bundles or (**b**) sheets in flat plates

9.18

- Flow paths through the membranes in a MBR
  - Early membranes used pressurized filtration with permeate produced by a flow path of inside-to-outside
    - These membranes had high energy use and required frequent backwashing and chemical cleaning
  - Modern membranes use a low vacuum with permeate produced by a flow path of outside-to-inside (Fig. 9.7)
    - These membranes have more membrane surface area, lower energy use, and are more easily cleaned

Fig. 9.7 Membrane fiber showing the permeate flow path from outside-to-inside that is typical for a modern membrane



9.19

- Solids production and need for wasting
  - Net solids production (Y_N) depends on wastewater characteristics, solids retention time (SRT), and temperature
    - Example solids production data are provided in Table 9.5
  - In general, net solids production is higher in a MBR receiving a lower quality influent and operated with a shorter SRT under lower temperature conditions, e.g.:
    - Lower quality influent (e.g., without primary treatment compared to an influent after primary treatment)
      - $Y_N \approx 1.4\times$ to $1.8\times$ higher
    - Shorter solids retention time (e.g., 5 days vs. 30 days)
      - $Y_N \approx 1.4\times$ to $1.6\times$ higher
    - Lower temperatures (e.g. 10 °C vs. 30 °C)
      - $Y_N \approx 1.2\times$ to $1.4\times$ higher

9.20

Table 9.5 Net solids production in an MBR (lb VSS/lb BOD removed) as affected by wastewater characteristics, solids retention time and temperature (after Tchobanoglous et al. 2014)[a]

Solids retention time (days)	With primary treatment[b]			Without primary treatment[c]		
	10 °C	20 °C	30 °C	10 °C	20 °C	30 °C
1	0.87	0.72	0.61	1.18	1.05	0.94
5	0.71	0.62	0.52	1.03	0.92	0.85
15	0.55	0.48	0.42	0.83	0.76	0.71
30	0.44	0.39	0.34	0.70	0.65	0.60

[a]Values in Table 9.5 are approximate since they were scaled off of Fig. 8.7 in Tchobanoglous et al. 2014.
[b]COD/BOD = 1.9–2.2; TSS/BOD = 0.5–0.7; Primary treatment at 60 % removal of TSS, with primary effluent inert TSS = 30 %.
[c]COD/BOD = 1.9–2.2; TSS/BOD = 0.9–1.1; Primary treatment inert TSS = 50 %.

9.21

- MBR effluent
  - The effluent from the MBR is the water that passes through the membrane, which is known as permeate
  - MBR effluent quality should be very consistent over time unless there is an operational problem or maintenance need
    - For example, effluent quality could deteriorate if there was a cross-connection or a ruptured membrane unit
  - Further treatment (if any) of the MBR effluent may be needed
    - Discharge or reuse plan requirements, e.g.:
      - MBR effluent has been treated by reverse osmosis to produce water for potable reuse or industrial purposes
    - Regulatory requirements, e.g.:
      - Regulatory requirements could dictate that the MBR effluent had to be treated with a disinfection agent technology before discharge or reuse

9.22

9-4. Design and Implementation

- Considerations for design and implementation (D&I) of a membrane bioreactor to achieve tertiary treatment and disinfection[a]
  - Development features
  - Wastewater source and pretreatment
  - MBR bioreactor sizing and membrane contact
  - Membrane flux
  - Membrane area sizing
  - Membrane fouling and control
  - MBR installation at the site
  - Operation and maintenance (O&M) requirements
  - Other considerations (e.g., commercially available MBRs)

[a]*Note*: Disinfection is covered in Chap. 14

9.23

- D&I considerations—Development features
  - Land use and development attributes
    - Type and number of wastewater sources
      - Potential for clustering sources
    - Site land use and topography
      - Construction space near the wastewater source(s)
      - Locations of reuse or discharge options
    - Ability to provide O&M as required
  - Subsurface characteristics
    - Can affect construction
      - Depth to shallow bedrock
      - Depth to ground water
      - Depth of freezing zone

9.24

- D&I considerations—Wastewater source and pretreatment
  - MBRs can treat wastewaters of widely different characteristics
    - For decentralized applications typical sources include:
      - Domestic wastewaters from residential developments
      - Commercial and institutional wastewaters (e.g., restaurants, resorts, office buildings, etc.)
      - Wastewaters from mixed-use developments
  - Pretreatment requirements
    - Minimum requirement is fine screening (<3 mm, 1–2 mm typ.)
    - Primary treatment can be beneficial
    - Conditioning may be needed for some treatment processes
      - e.g., alkalinity feed if the MBR is designed to achieve nitrification but the influent alkalinity is too low (e.g., 50 mg/L)

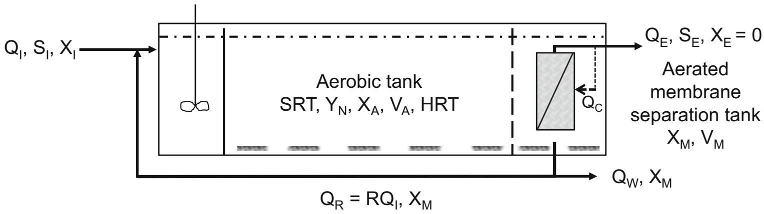
9.25

- D&I Considerations—Aerobic tank sizing and solids wasting
  - Aerobic tank sizing and solids wasting parameters are given in Table 9.6 and Fig. 9.8 and described in the following pages

Table 9.6 Key design parameters and method of selection or calculation for MBR sizing

Design parameter	Selection or calculation	Typical values
SRT	A designer selects a SRT based on temperature conditions and nitrification needs	8–40 days
Y_N	Y_N is chosen based on the influent wastewater characteristics, SRT, and temperature	See Table 9.5
X_A	The target MLVSS concentration is chosen	8000–12,000 mg/L
HRT	HRT is determined as a function of the selected values for SRT, Y_N, and X_A (Eq. 9.5)	2 to >12 h
R	Need for solids wasting depends on the recycle ratio used for return of solids from the membrane tank to the aerobic tank	4–6

9.26



Q_I = influent daily flow (gal/day)
S_I = influent substrate (BOD) concentration (mg/L)
X_I = influent concentration of VSS (mg/L)
Y_N = net solids production (lb VSS/lb BOD)
SRT = design solids retention time (days)
X_A = mixed liquor VSS in the aerobic tank (mg/L)
V_A = volume of the aerobic tank (gal)
HRT = hydraulic retention time (days)
X_M = VSS concentration in the tank with the membrane in it (mg/L)
V_M = volume of the tank with the membrane module in it (gal)
Q_W = average daily flow of waste solids from the MBR (gal/day)
Q_C = daily permeate flow used for cleaning each unit (gal/day)
Q_R = recycle flow rate (gal/d)
R = recycle ratio (-)
Q_E = effluent flow rate (gal/d)
S_E = effluent concentration of BOD (mg/L)
X_E = effluent concentration of VSS (mg/L)

Fig. 9.8 Illustration of an aerobic MBR flow regime that can be used for determining the aerobic tank volume and solids wasting

9.27

- Aerobic tank volume
  - The volume can be determined based on the solids retention time chosen in a fashion similar to that used for conventional activated sludge biological treatment[a]
  - The SRT is defined by Eq. 9.2

$$\mathrm{SRT} = \frac{X_A V_A + X_M V_M}{Q_W X_M} \tag{9.2}$$

Where:
SRT = design solids retention time (days)
X_A = mixed liquor VSS in the aerobic tank (mg/L)
V_A = volume of the aerobic tank (gal)
X_M = VSS concentration in the tank with the membrane in it (mg/L)
V_M = volume of the tank with the membrane module in it (gal)
Q_W = average daily flow of waste solids from the MBR (gal/day)

[a]See Chap. 7 for additional information on aerobic biological treatment.

9.28

- Based on a chosen SRT, the volume of the aerobic tank can be determined using Eq. 9.3

$$V_A = \frac{(SRT)(Q_D)(S_I)(Y_N)}{X_A} \tag{9.3}$$

Where:
V_A = volume of the aerobic tank (gal)
SRT = design solids retention time (days)
Q_D = design average daily flow rate (gal/day)
S_I = influent BOD_5 or COD substrate concentration (mg/L)
Y_N = net yield coefficient (lb-VSS produced/lb-BOD_5 or -COD removed) as a function of SRT (–)
X_A = average mixed liquor volatile suspended solids (MLVSS) (mg/L). *Note:* MLVSS ≈ a fraction of mixed liquor total suspended solids

9.29

- Hydraulic retention time
  - With the volume determined, the hydraulic retention time, HRT, can be calculated using Eq. 9.4

$$HRT = \frac{V_A}{Q_D} \tag{9.4}$$

Where:
HRT = hydraulic retention time (day). *Note:* HRT can be calculated for an average day, peak day, etc.
V_A = aeration tank volume (gal)
Q_D = design daily flow rate (gal/day) (e.g., Q_A or $Q_A \times PF$)
PF = peaking factor, varies depending on application (–) (typ. 1.0–3.0)

9.30

- MBR operating parameters are inter-related
  - Rearranging and substituting terms in Eqs. 9.3 and 9.4 yields the following relationship for HRT, SRT, MLVSS, and Y_N

$$HRT = \frac{(SRT)(S_I)(Y_N)}{(X_A)} \tag{9.5}$$

Where:
HRT = hydraulic retention time (days)
SRT = design solids retention time (days)
S_I = influent BOD_5 or COD substrate concentration (mg/L)
Y_N = net yield coefficient (lb-VSS produced/lb-BOD_5 or COD removed) (–)
X_A = average mixed liquor volatile suspended solids (MLVSS) (mg/L)

9.31

- Wasting of solids
  - Wasting of solids is required to maintain a target SRT and MLVSS
  - A definition schematic for wasting from an aerobic MBR is shown in Fig. 9.9
  - The daily waste sludge flow rate can be computed for a given return sludge recycle ratio (R) and SRT using Eqs. 9.6 and 9.7
    * Note that wasting does not have to occur every day but can vary as an average value over the duration of the target SRT

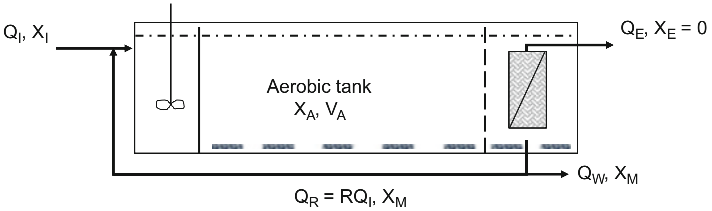


Fig. 9.9 Definition schematic for solids recycling and wasting from a membrane bioreactor

9.32

- A mass balance on the MBR in Fig. 9.10 yields Eq. 9.6

$$(RQ_I)(X_M) + (Q_I)(X_I) = (Q_I + RQ_I)X_A$$
$$(RQ_I)(X_M) + (Q_I)(\sim 0) = (Q_I + RQ_I)X_A$$
$$X_A = \left(\frac{R}{1+R}\right)X_M \tag{9.6}$$

- Substituting Eq. 9.6 into Eq. 9.3 and rearranging yields Eq. 9.7

$$Q_W = \frac{\left(\frac{R}{1+R}\right)V_A + V_M}{SRT} \tag{9.7}$$

  * Values of the recycle ratio, R, typically range from 4 to 6
  * Values of V_M vary but can be in the range of 20–30 % of V_A

9.33

■ D&I considerations—Membrane unit configurations

- Membrane units are assembled in modules or cassettes that contact the contents of the aerobic tank in two basic configurations as shown in Fig. 9.10

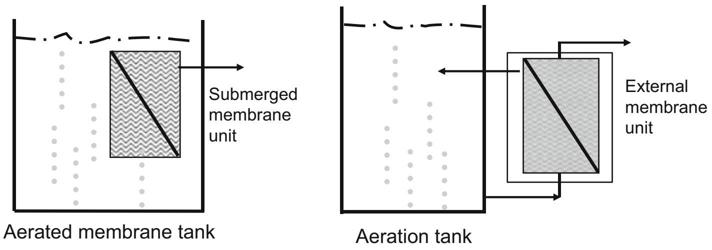


Fig. 9.10 MBRs can be configured with membrane units that are submerged in the aeration tank (*left*) versus membrane units external to the aeration tank (*right*)

9.34

- D&I considerations—Membrane flux
  - Flux is a design and operation parameter (Fig. 9.11)
    - Flux = Vol./area/time (gal/day/ft^2, m^3/m^2/day, m/day, or L/m^2/h)
  - Flux depends on many factors including:
    - Pore size
    - Trans-membrane pressure (TMP)
    - Cross flow velocity
    - Membrane resistance
    - Membrane fouling
  - Example fluxes
    - Average = 10–15 gal/day/ft^2
    - Maximum = 18–30 gal/day/ft^2 (for $\leq$6 h)

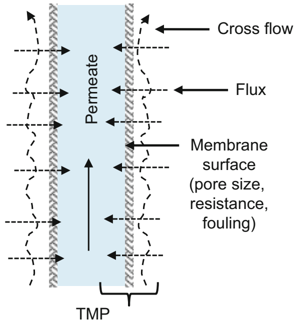


Fig. 9.11 Flux through a membrane depends on membrane design and operating parameters

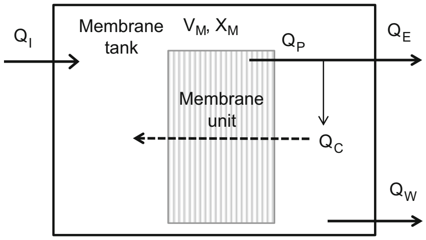
Note: 1 L/m^2/h = 0.59 gal/day/ft^2

9.35

- D&I considerations—Membrane surface area sizing
  - Membranes can be assembled in units
    - Each unit (also known as cassettes or modules) has a certain membrane surface area for producing permeate
    - Total number of units is based on the total membrane area required and the surface area provided by each unit
  - Total membrane surface area required is based on the membrane flux and the daily flow to be processed
    - Membrane area is needed to produce permeate which is discharged as effluent (Q_E)
    - Membrane surface area may also be needed to produce permeate that is used for membrane cleaning (Q_C)

9.36

- Flow regime for membrane sizing is given in Fig. 9.12



Q_I = influent daily flow (gal/d)
Q_P = permeate produced (gal/d)
Q_E = permeate which leaves the MBR as effluent (gal/d)
Q_C = permeate used for cleaning is recycled back to the aeration tank (gal/d)
Q_W = wasting of bioreactor contents to maintain SRT (gal/d)

Fig. 9.12 An example MBR flow regime that can be used for membrane sizing

9.37

- Membrane surface area required and number of membrane units can be calculated using Eqs. 9.8–9.12

$$F_M' = F_M - F_C \tag{9.8}$$

$$A_M' = \frac{(Q_I - Q_W)}{F_M'} \tag{9.9}$$

$$N_C' = \frac{A_M'}{A_C} \Rightarrow N_C \tag{9.10}$$

$$A_M = (N_C)(A_C) \tag{9.11}$$

9.38

$$F_M = \frac{(Q_I - Q_W) + N_C(Q_C)}{A_M} \Rightarrow \text{okay?} \tag{9.12}$$

Where:
A'_M = approximate total surface area of membranes required (ft^2)
A_M = final total surface area of membranes required (ft^2)
Q_I = influent daily flow (gal/day)
Q_W = daily wasting flow of biosolids to maintain SRT (gal/day)
Q_C = daily permeate flow used for cleaning each unit (gal/day)
F_M = design membrane flux limit (gpd/ft^2) (e.g., 10–15 gal/day/ft^2)
F'_M = membrane flux producing effluent from the MBR (gal/day/ft^2)
F_C = membrane flux used for cleaning (gal/day/ft^2)
N'_C = estimated number of units required (–)
N_C = number of units required (–)
A_C = area provided by each unit (ft^2)

9.39

- D&I considerations—MBR effluent
  - The MBR effluent is the permeate that passes through the membrane and exits the MBR
  - The flow rate of membrane effluent is given by Eq. 9.13

$$Q_E = Q_I - Q_W \tag{9.13}$$

Where:
Q_E = permeate which leaves the MBR as effluent (gal/day)
Q_I = influent wastewater flow (gal/day)
Q_W = wasting of sludge from the MBR to maintain a target SRT (gal/day)

  - The MBR effluent may require further treatment based on discharge or reuse plans (e.g., chemical or physical disinfection)

9.40

- D&I considerations—Membrane fouling and control
  - Membrane fouling is due to adsorption or deposition
    - Biopolymers or soluble microbial products (SMP)
    - Inorganic scales
  - Membrane fouling affects flux or trans-membrane pressure
    - In constant flux operation, TMP increases due to fouling
    - In a constant TMP operation, flux declines due to fouling
  - Membrane fouling is a key parameter affecting operation and process economics
    - Membrane fouling potential generally increases with higher MLSS levels, but it can also occur at low MLSS
    - Fouling causes increased O&M due to membrane cleaning and membrane replacement

9.41

- Fouling can be hindered or eliminated by different methods
  - Pretreatment of the influent to the MBR
  - Clean in place (CIP) maintenance cleaning
    - Methods
      - Back flushing with water/air
      - Relaxation
    - Frequency
      - Can be done every hour or more often
  - Chemical cleaning
    - Methods
      - Remove membrane module from service and chemically soak and clean it
    - Frequency
      - Every 3–6 months or more

9.42

- D&I considerations—Installation at the site
  - Proper installation is critical to achieving performance of biological treatment systems including MBRs
    - The location of the MBR must enable access for construction equipment and service vehicles
    - Tankage and piping needs to be watertight and structurally sound
    - Materials of construction need to be corrosion resistant
    - Power is required for pumps, aerators, controls, etc. so power sources need to be robust and reliable
  - Proper startup of the MBR is also critical
    - Seeding of the aeration basin with activated sludge solids from another biological treatment unit can be helpful
  - Figures 9.13 and 9.14 present photographs illustrating the installation of packaged MBRs

9.43

Fig. 9.13 Photographs taken during installation of a MBR system to serve a commercial development located along the coast in Malibu, California. The system included MBRs, UV disinfection units, and high rate subsurface infiltration units. (Photographs courtesy of Integrated Water Services, Inc.)

9.44

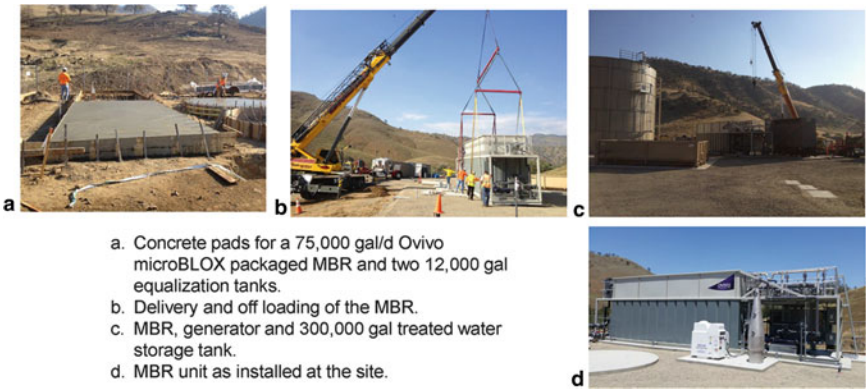


Fig. 9.14 Photographs taken during installation of a MBR system to serve an Indian Tribe in California. The system consists of installation of a wastewater conveyance piping to a packaged MBR with equalization tanks, a treated water storage tank, a disinfection system, and land-based based dispersal system. (Photographs courtesy of Integrated Water Services, Inc.)

9.45

- D&I considerations—Operation and maintenance
  - O&M can be complicated and include a variety of activities and events including:
    - Typical O&M required for aerobic bioreactors (Chap. 7)
    - O&M of membrane pumps and aeration units
    - Membrane cleaning, maintenance and recovery cleaning
    - Membrane replacement
  - Examples of three MBRs with operating parameters and routine O&M are summarized in Table 9.7

9.46

Table 9.7 Examples of routine O&M provided for three MBR units (Larsson and Persson 2004)

Vendor	Zenon	US Filter	Kubota
Membrane type and pore size	Hollow fiber 0.04 μm	Hollow fiber 0.04 μm	Plate 0.4 μm
Pre-screening (mm)	≤2	≤2	≤3
Aerobic tank MLSS (mg/L) Aerobic tank SRT (days)	≤10,000 10–15	≤10,000 10–15	≤10,000 15
Flux mgmt.: ft^2/gal/h	0.015	0.007	0.022
Flux rate: Avg. gal/ft^2/h Peak hr.—gal/ft^2/h for ≤6 h	10–15 <25	10–15 <30	10–15 <35
Aeration cycle	10 s on 10 s off	Constant	Constant
Maintenance cleaning	Backpulse and relax hourly	Backpulse or relax; 1 min per 15 min	Backpulse 1 min per 15 min
Recovery clean and frequency	Chemical soak of drained cell ≥3 months	Chemical soak of drained cell ≥3 months	Chlorine backwash in situ ≥6 months

9.47

9-5. Summary

- Membrane filtration can remove particles, molecules, ions and micro-organisms out of wastewaters and other impaired waters
- In an aerobic MBR, the mixed liquor in a bioreactor is treated using micro- to ultrafiltration membranes to retain biomass and filter out other constituents
- For use in a MBR, membranes typically need to have uniform pore size distribution and high porosity, neutral charge, be inert and non-biodegradable, be easy to clean, and be durable and easy to replace
    - The membranes in MBRs can achieve average flux rates of about 10–15 gal/day/ft^2
- MBRs can achieve tertiary treatment with disinfection and produce a consistent high quality effluent as long as O&M is reliably provided

9.48

9-6. Example Problems

- 9EP-1. Design of an aerobic MBR
  - Given information
    - Student housing development with an average daily flow estimated to be 30,000 gal/day and a maximum daily flow of 45,000 gal/day (based on a PF = 1.5)
    - The operating temperature for the MBR is estimated at 20 °C
    - The influent to the MBR will be primary effluent
    - Each membrane unit has a surface area = 323 ft^2
    - The design membrane flux to handle the average daily flow and produce permeate for effluent plus cleaning = 12 gal/day/ft^2
    - Membrane permeate returned to the membranes for cleaning = 325 gal/day per unit
    - The SRT chosen for the aerobic tank of the MBR was 15 days to enable nitrification and limit net solids production

9.49

- Determine
  - The volume (V_A) of the aerobic tank (in gal)
  - The hydraulic retention time (HRT) in the aerobic tank (days)
  - Average volume/day of sludge (Q_W) that needs to be wasted from the MBR (in gal/day)
  - On an average day, how much MBR effluent water will be produced and available for irrigation (in gal/day)
  - How many membrane units will be required
  - What the maximum flux rate would be during the maximum daily flow
- Solution
  - A flow regime for solving this problem appears in Fig. 9.8

9.50

- Determine the aerobic tank volume, V_A
  - Select a value for the net solids production, Y_N
    - With the SRT set at 15 days for a MBR receiving primary effluent and operated at 20 °C, a value for Y_N can be selected for the aerobic treatment operation
    - From Table 9.5, $Y_N = 0.48$ lb-VSS/lb-BOD removed
  - The target X_A is chosen to be 10,000 mg VSS/L based on typical values (Table 9.6)
  - The aerobic tank volume, V_A, is calculated using Eq. 9.3

$$V_A = \frac{(SRT)(Q_D)(S_I)(Y_N)}{X_A} \tag{9.3}$$

$$V_A = \frac{(15\,\text{days})(30{,}000\,\text{gal/day})(150\,\text{mg-BOD/L})(0.48\,\text{lb-VSS/lb-BODrem.})}{10{,}000\,\text{mg-VSS/L}}$$

$$V_A = 3{,}240\,\text{gal}$$

9.51

- The hydraulic retention time can be calculated using Eq. 9.4 or 9.5

$$HRT = \frac{V_A}{Q_D} \tag{9.4}$$

$$HRT = \frac{3{,}240\,\text{gal}}{30{,}000\,\text{gal/day}} = 0.108\,\text{day} = 2.6\,\text{h}$$

$$HRT = \frac{(SRT)(S_I)(Y_N)}{(X_A)} \tag{9.5}$$

$$HRT = \frac{(15\text{ days})(150\,\text{mg-BOD/L})(0.48\,\text{lb-VSS/lb-BODrem.})}{(10{,}000\,\text{mg-VSS/L})}$$

$$HRT = 0.108\,\text{day} = 2.6\,\text{h}$$

9.52

- Calculate the average volume/day of activated sludge (Q_W) that needs to be wasted from the MBR (in gal/day)
  * Equation 9.7 is used to calculate Q_W
  * Assume $V_M = 25\,\%$ of V_A or $V_M = 810$ gal
  * Assume R = 4 (typ. value = 4–6)

$$Q_W = \frac{\left[\left(\frac{R}{1+R}\right)V_A\right] + V_M}{SRT} \tag{9.7}$$

$$Q_W = \frac{\left[\left(\frac{4}{1+4}\right)3{,}240\,\text{gal}\right] + 810\,\text{gal}}{15\,\text{days}}$$

$$Q_W = 227\,\text{gal}/\text{day}$$

9.53

- Determining how much MBR effluent water will be produced and available for irrigation on an average day
  * The effluent produced on an average day can be calculated using Eq. 9.13

$$Q_E = Q_I - Q_W \tag{9.13}$$

$$Q_E = 30{,}000\,\text{gal}/\text{day} - 227\,\text{gal}/\text{day}$$

$$Q_E = 29{,}773\,\text{gal}/\text{day}$$

9.54

- Membrane area required and number of membrane units can be calculated using Eqs. 9.8–9.12
  * Initial determination of the number of membrane units required is given by Eqs. 9.8–9.10

$$F'_M = F_M - F_C = 12\,\text{gal/day/ft}^2 - \frac{325\,\text{gal/day}}{323\,\text{ft}^2} = 11\ \text{gal/day/ft}^2 \tag{9.8}$$

$$A'_M = \frac{(Q_I - Q_W)}{F'_M} = \frac{(30{,}000\ \text{gal/day} - 227\,\text{gal/day})}{11\ \ \text{gal/day/ft}^2} = 2{,}707\ \text{ft}^2 \tag{9.9}$$

$$N'_C = \frac{A'_M}{A_C} \Rightarrow N_C \tag{9.10}$$

$$N'_C = \frac{2{,}707\ \text{ft}^2}{323\,\text{ft}^2/\text{unit}} = 8.4 \Rightarrow N_C = 9\ \text{units}$$

9.55

  * The membrane surface area provided by the 9 membrane units is checked to see if it is sufficient for the design flow and cleaning required using Eqs. 9.11 and 9.12

$$A_M = (N_C)(A_C) \tag{9.11}$$

$$A_M = (9\ \text{units})(323\ \text{ft}^2/\text{unit}) = 2{,}907\ \ \text{ft}^2$$

$$F_M = \frac{(Q_I - Q_W) + N_C(Q_C)}{A_M} \Rightarrow \text{okay?} \tag{9.12}$$

$$F_M = \frac{(30{,}000\ \text{gal/day} - 227\ \text{gal/day}) + 9\ \ \text{units}(325\ \text{gal/unit})}{2{,}907\ \text{ft}^2}$$

$$F_M = 11.2\ \ \text{is} \le 12\ \text{gal/day/ft}^2$$

✓Okay

9.56

- The maximum flux rate during the maximum daily flow can be determined using Eq. 9.12

$$F_M = \frac{(Q_I - Q_W) + N_C(Q_C)}{A_M} \tag{9.12}$$

$$F_M = \frac{(45{,}000 \text{ gal/day} - 227 \text{ gal/day}) + 9 \text{ units}(325 \text{ gal/day/unit})}{2{,}907 \text{ ft}^2}$$

$$F_M = 16.4 \text{ gal/day/ft}^2$$

✓ This F_M appears okay since it is less than the maximum flux rate which typically ranges 18–30 gal/day/ft^2 (typically during a peak period of $\leq$6 h).

Chapter 10

Treatment Using Constructed Wetlands

10-1. Scope

A constructed wetland is an engineered natural plant and water system that is designed to treat wastewater by exploiting the processes that occur within natural wetland ecosystems. This chapter describes the principles and processes that occur in constructed wetlands that are important to wastewater treatment and water reclamation. Then the chapter describes the design and implementation of free water surface and subsurface vegetated bed wetlands for secondary treatment in decentralized systems.

10-2. Key Concepts

- Constructed wetlands exploit the hydraulic and purification processes that occur in natural wetlands to remove constituents of concern from wastewaters and other impaired waters. There are three types of constructed wetlands that are relevant for use in decentralized systems.
  - Horizontal flow free water surface wetlands (FWS) are typically used for tertiary polishing of secondary wastewater effluents while also providing habitat and aesthetic benefits.
  - Horizontal flow vegetated subsurface bed wetlands (VSB) are typically used to achieve secondary treatment of primary effluents such as septic tank effluent but without a free-water surface and the potential issues associated with that feature.
  - Vertical flow subsurface bed wetlands (VVSB) can be used to treat higher strength wastewaters and dewater and stabilize sludges.

R.L. Siegrist, *Decentralized Water Reclamation Engineering*,
DOI 10.1007/978-3-319-40472-1_10

- Of these three, VSB wetlands are most widely used for smaller flows in decentralized systems while FWS wetlands also used, but for larger design flows. Both of these are described in detail in Chap. 10.

- In a typical FWS wetland, the water surface is visible and flow is horizontal through a shallow pond containing a bed of gravel on the bottom and various types of vegetation. Gravel media bed depths are typically 1 ft or more to support plant rooting. The top of the gravel bed may be 2–4 ft or more below the water surface. There may also be fully vegetated beds of porous media in an inlet and outlet zone. FWS plants can include floating, submerged, and emergent species. FWS wetlands are typically used to treat larger flows where the influents are secondary effluents from wastewater treatment plants and the goal is to produce a tertiary effluent quality while creating wildlife habitats and yielding aesthetic benefits.
- In a typical VSB wetland, the water surface is just below the top of the wetland surface (typ. 0.5 ft below) and flow is horizontal through a shallow bed of gravel (or similar coarse media) planted with emergent vegetation. Gravel bed depths are typ. 1.5–2.5 ft in a VSB wetland. VSB wetlands are typically used to treat smaller flows where the influents are primary or possibly secondary effluents and the goal is to produce a secondary or higher quality effluent.
- Hydraulic design of a FWS or VSB wetland needs to ensure that wastewater flow through the wetland will occur without backup or surfacing, that plug-flow like conditions will exist without major short-circuiting, and pollutant loading rates, hydraulic retention times and conditions will be sufficient to produce a desired effluent quality. The flow characteristics are generally determined by the wetland surface area, aspect ratio, and water depth along with the hydraulic gradient and flow controls.
- Clean porous media are used throughout a VSB wetland and also in the inlet and outlet zones of a FWS wetland. The properties and use of porous media are important to the function of a constructed wetland.
  - Porous media are chosen that are clean, durable, and of a grain size that yields a desired hydraulic conductivity. The saturated hydraulic conductivity (K_S) of the clean porous media is typically very high. However, after years of operation, the K_S can decline due to vegetation growth and porous media pore clogging. This decline can yield an effective hydraulic conductivity (K_E) for flow through the wetland that is much lower than the K_S of the clean porous media.
  - If the porous media are too fine for the influent wastewater loading rates (e.g., lb-TSS/ft^2/day), the extent of loss in K_S can be up to 90–99 % in the inlet zone of a VSB (or FWS) wetland due to pore clogging. In the bulk of the treatment zone of a VSB wetland the

extent of loss in K_S is typically more like 50 %, principally due to vegetation growth.
- Selecting the proper value of K_E to use for determining the cross-sectional area required for flow through a wetland needs to account for the operational loss in the wetland hydraulic capacity. This is particularly true in VSB wetlands since flow is entirely through a bed of porous media and there are no open water zones such as those that exist in a FWS wetland.

■ Vegetation is important and can have multiple functions in a constructed wetland.
- In a FWS wetland vegetation can include floating, submerged, and emergent species while in a VSB wetland, emergent macrophytes are dominant.
- Locally available plants that are noninvasive are typically used.
- The importance of vegetation to wetland function and performance varies depending on the characteristics of the wastewater being treated, the wetland type, the wetland design specifics, and the geographic and climatic conditions.

■ Biochemical conditions and temperatures can affect constructed wetland function and performance.
- Constructed wetlands can have zones that are aerobic, anoxic, or even anaerobic. This is due to the dissolved oxygen sources and sinks that can occur. Wetland plants normally adapt to, and interact with, the different redox zones.
- The temperature in constructed wetlands can fluctuate daily and seasonally, and FWS and VSB wetlands can be used in very cold and very hot climate conditions.

■ Constructed wetlands can treat wastewaters and other impaired waters (e.g., stormwater, mine drainage) and produce high quality effluents.
- FWS wetlands are normally designed to polish secondary effluents (e.g., nitrified aerobic treatment unit effluent) and can be designed to produce a wetland effluent that has BOD_5 and TSS <10 mg/L with nitrification and total N concentrations <10 mg/L. Total P removals as well as bacterial removals are normally limited.
- VSB wetlands are normally designed to treat primary effluents (e.g., septic tank effluent) and can reliably produce effluents with BOD_5 and TSS <30 mg/L. VSBs can be designed for nitrification, but total N and P removals as well as bacterial removal are normally limited. To enhance treatment of nutrients and microbes, VSB wetlands can be established with more reactive media (e.g., lightweight expanded clay aggregate).

- Design and implementation of a constructed wetland requires consideration of a variety of factors and choices that must be made.
  - Design considerations for constructed wetlands include: the wetland type features and suitability for a particular project, the wastewater source and treatment prior to the wetland, the wetland surface area required to handle the daily flow, the wetland water depth and rooting depth, the hydraulic retention time, the porous media used and the flow rate capacity through it, the surface area geometry and use of flow controls.
  - Design approaches to sizing the surface area required for a constructed wetland generally include use of empirical data and areal loading rates (ALR), plug-flow modeling of constituent removal based on area-based reaction rates, or a combination of the two.
    - Achieving a certain treatment efficiency requires that the constituent loadings to the wetland be limited to a certain level and that the hydraulic retention time (HRT) is sufficient for reactions to occur. HRTs are typically in the range of 2–6 days for BOD removal but longer for N removal.
    - To produce an effluent with 30 mg/L BOD, organic loading rates (OLR) are typically limited (e.g., <0.14 lb-BOD/day/ft^2).
  - Considerations related to installation of the wetland include: the wetland location on a site, the wetland configuration, wetland containment methods and bed depth, water inlet and outlet flow controls, and vegetation establishment.
  - FWS constructed wetlands can take a year or more to 'start up' and during this period, treatment performance can be transient and less than optimal. VSB wetlands tend to startup more quickly due to the flow through porous media where attached growth biological processes can occur.

- Most constructed wetlands are designed and implemented as passive systems so operation and maintenance requirements are limited. Routine requirements can include simple inspections while maintenance activities can include periodic harvesting of plants (e.g., to achieve true removal of nutrients taken up by plants). Maintenance activities can also involve dealing with pore clogging and associated hydraulic capacity problems, though these activities should be infrequent if the wetland is properly designed and operated.

10-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 10 are presented in the Slides section.

10-4. Terminology

Terminology introduced and used in Chap. 10 is defined below.

Area-based reaction rates (k_A)—An approach to expressing reaction rates for constructed wetlands that is based on the horizontal surface area of the wetland and has units of length per time (e.g., ft/day).

Areal loading rate—An approach to determining the horizontal surface area of a constructed wetland based on empirical data concerning a loading rate specification such as mass of constituent per unit area per time that yields a particular effluent quality (e.g., the ALR in lb-BOD/ft^2/day that yields a wetland effluent BOD of 30 mg/L).

Background concentrations (C*)—Refers to the concentrations of constituents that are present within a wetland based on additions through means other than the influent wastewaters being treated (e.g., by plant decay, wildlife presence, atmospheric deposition).

Cell—(1) Refers to the structural, functional and biological unit of a living organism. (2) Refers to a portion of a constructed wetland that is intentionally established to help support plug-flow like conditions so flow occurs through the wetland from the inlet to the outlet with minimized short-circuiting.

Clogging—In the context of a constructed wetland, clogging refers to the filling of porosity within a porous media like gravel by the deposition of suspended solids, accumulation of biomass and microbial byproducts, and the growth of plant roots. Clogging can reduce the saturated hydraulic conductivity of the porous media from a value for the clean media (K_S) to an effective value that accounts for the loss in porosity and permeability due to clogging (K_E).

Constructed (treatment) wetland—Refers to a physical system that is designed and implemented to mimic and exploit the processes occurring in natural wetlands to accomplish treatment of an impaired water such as residential, commercial or other wastewaters.

Cross-sectional area—(1) Refers to a vertical or horizontal plane view through a physical object or a part of it. (2) Refers to a vertical or horizontal plane that is perpendicular to flow in a treatment unit operation (e.g., constructed wetland).

Effective hydraulic conductivity (K_E)—In the context of a VSB or FWS constructed wetland, effective hydraulic conductivity (K_E) is a term that is associated with the ability of a constructed wetland to transmit water through it. K_E (e.g., gal/day/ft^2) is less than the K_S of the new wetland due to clogging processes that reduce the hydraulic conductivity of the wetland, particularly in the inlet zone and initial section of the treatment zone. The K_E is multiplied by the hydraulic gradient (e.g., 0.001 ft/ft) and cross-sectional area (e.g., ft^2) to determine the actual hydraulic capacity (e.g., ft^3/day or gal/day) through a zone of the wetland that has been in operation for a period of time.

Effective hydraulic retention time (HRT_E)—The average time the liquid entering a constructed wetland remains in the wetland during horizontal flow from the inlet to the outlet accounting for the presence of porous media and inactive flow zones.

Effective water volume (V_{WE})—The volume of water in a wetland that is actually involved in flow through it, accounting for the presence of porous media and inactive flow zones.

Effluent—The liquid that is discharged from a treatment unit. For example, the effluent from a biofilter is the filtrate that is discharged (not recycled) and transported to a next treatment unit or discharged to the environment. Effluent can become the influent to another treatment unit operation. For example, in the context of landscape drip dispersal (LDU) effluent is produced by an upstream treatment unit (e.g., aerobic unit) and becomes the influent to the LDU.

Evapotranspiration (ET)—The amount of water removed by evaporation and transpiration under the current soil moisture and environmental conditions.

Flow controls—Refers to physical barriers (typically vertically oriented but they could be horizontal) that are used to help enable plug-flow like conditions and avoid short-circuiting during flow through a constructed wetland from the inlet to the outlet end.

Free water surface wetland—A type of constructed wetland for treatment of wastewater or other impaired waters that has the distinguishing feature of having a major portion of the horizontal surface of the wetland that is comprised of an open water surface.

Horizontal flow—(1) Refers to a flow path that is near level from one location to another. (2) Refers to constructed wetlands in which the wastewater being treated flows in a horizontal direction from an inlet to an outlet.

Hydraulic capacity—The ability of a treatment unit operation to transmit liquid within it from the inlet to the outlet.

Hydraulic gradient—A unit-less measure of the force causing water flow to occur through a channel or bed of porous media defined as the change in elevation head over a unit length of the flow path.

Hydraulic inefficiencies—Refers to the departure from true plug flow that occurs in constructed wetlands by virtue of the inlet and outlet hydraulics, wetland geometry, and heterogeneities within the wetland. Hydraulic inefficiencies are accounted for during wetland surface area sizing using the P-k_A-C* modeling of constituent removal in a wetland.

Inactive flow zones—Generally used in the context of constructed wetlands to refer to volumetric portions within it that are not involved in advective movement of water from the inlet to the outlet.

Influent—The effluent from an upstream treatment unit becomes the influent to a downstream treatment unit. For example, septic tank effluent is often used as the influent to a porous media biofilter.

Macrophyte—A class of vegetation that is used in constructed wetlands. Examples of macrophytes include emergent plants (e.g., bulrush, common reed, cattail), floating plants (e.g., water lily, duckweed, water hyacinth), and submerged plants (e.g., pondweed, American shoreweed).

Nominal hydraulic retention time (HRT_N)—(1) The average time the liquid entering a tank, basin or compartment remains in it during flow from the inlet to the outlet not accounting for the presence of porous media and inactive flow zones. (2) In a constructed wetland the average time the liquid entering it remains in the wetland during horizontal flow from the inlet to the outlet not accounting for the presence of porous media and inactive flow zones.

Nominal volume (V_N)—(1) The volume defined by the bottom, sides, and top of a tank, basin or compartment not accounting for any objects within it. (2) In a porous media biofilter or a constructed wetland, nominal volume does not account for any porous media, vegetation, or inactive flow zones.

P-k_A-C* modeling—An approach to modeling constituent removal in a constructed wetland that is based on representing the flow regime as plug-flow like using a number of tanks in series but which accounts for a departure from plug flow due to hydraulic inefficiencies as well area-based reaction rate constants that decline with distance from the inlet to the outlet of the wetland.

Plug flow—A flow regime where the velocity of the fluid is constant across any cross-section of the tank, basin or other unit perpendicular to the axis of the inlet to the outlet flow path.

Redox zones—Refers to a condition in a constructed wetland where based on sources and sinks for dissolved oxygen there can be zones that are aerobic, anoxic, or anaerobic.

Saturated hydraulic conductivity (K_S)—A term that is associated with the ability of a porous media to transmit water through it. K_S (e.g., gal/day/ft^2) is multiplied by the hydraulic gradient (e.g., typ. 1.0 for biofilters or soil treatment units) and cross-sectional area (e.g., ft^2) to determine the hydraulic capacity (e.g., gal/day) through the porous media.

Surface area—Refers to the horizontal surface area defined by the perimeter of a constructed wetland as measured at the land surface.

Treatment wetland—See Constructed wetland.

Vertical flow subsurface bed wetland—A type of constructed wetland for treatment of wastewater or other impaired waters and sludges that includes a bed of porous media that is planted with emergent vegetation through which impaired water or drainage from sludges that are being treated flows in a vertical direction from the top to bottom or vice versa.

Wetland volumetric efficiency—Refers to the fraction of the wetland water volume that is actually involved in water movement through the wetland accounting for the presence of inactive flow zones.

Zone—Refers to a portion of a constructed wetland that is characterized by different physical conditions (e.g., a portion with vegetated porous media vs. a portion with an open water surface) and may have different functions occurring (e.g., physical such as flow entering and being distributed at the inlet end of a wetland vs. biological such as in the portion of a wetland where treatment processes occur).

10-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chapter 10 are listed below.

ALR	Areal loading rate
CSTR	Continuously stirred tank reactor
CW	Constructed wetland
DO	Dissolved oxygen
ET	Evapotranspiration
FWS	Horizontal flow free water surface wetland
HLR_D	Design hydraulic loading rate
HRT	Hydraulic retention time
L	Length
OLR	Organic loading rate
P	The number of tanks in series to represent the departure from plug flow or precipitation
$P\text{-}k_A\text{-}C^*$	Refers to an approach to mathematical modeling of constituent removal in a constructed wetland
PTIS	Acronym for "$P\text{-}k_A\text{-}C^*$ tanks in series"
Q_C	Hydraulic capacity for flow through a constructed wetland
TIS	Tanks in series
VSB	Horizontal flow vegetated subsurface bed wetland
VVSB	Vertical flow vegetated subsurface bed wetland
W	Width

A_{xc}	Cross-sectional area through which flow occurs
A_W	Total required surface area of the wetland ($L \times W$)
A_W'	Wetland surface area actually provided ($L \times W$)
A_{XC}	Cross-sectional area perpendicular to flow
A_W	Horizontal surface area of a constructed wetland
C	Concentration of a constituent at the outlet or a fractional distance from the inlet, y
C^*	Background concentration of constituents
C_E	Effluent concentration
C_I	Influent concentration
d_W	Water depth

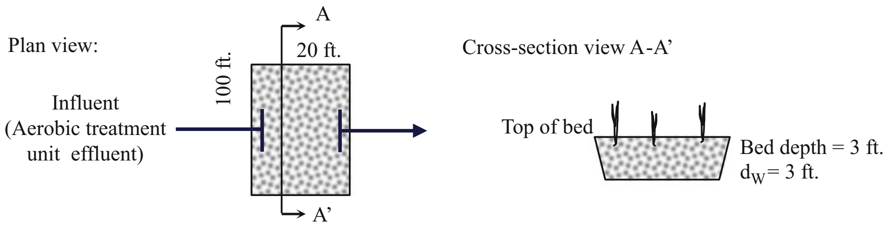
F	Conversion factor
HRT_E	Effective hydraulic retention time
HRT_N	Nominal hydraulic retention time
k_A	Area-based reaction rate constant
K_E	Effective saturated hydraulic conductivity
K_S	Saturated hydraulic conductivity
k_T	Reaction rate at temperature, T
k_{20}	Reaction rate at 20 °C
L	Length
P	Apparent no. of tanks in series for modeling that varies by constituent to account for weathering based on field data
q	Area-based design hydraulic loading rate
Q_C	Flow through the wetland
Q_D	Design daily flow rate
R_A	Aspect ratio
S	Hydraulic gradient of water surface from inlet to outlet, slope
T	Temperature
V	Volume
V_A	Volume of wetland containing water active in flow
V_B	Bulk volume of the wetland $= L \times W \times d_W$
V_N	Nominal volume
V_W	Water volume equals the open porosity in the wetland media
V_{WE}	Effective water volume accounting for porosity and inactive zones
W	Width
y	Fractional distance from the inlet to outlet
θ	Temperature activity coefficient
ε	Porosity of clean gravel in a VSB or plant-based void ratio in a FWS
e_V	Wetland volumetric efficiency accounting for inactive flow zones

10-6. Problems

10.1. What type(s) of vegetation is used in a free water surface constructed wetland?

10.2. What type(s) of vegetation is used in a vegetated subsurface bed wetland?

10.3. For the vegetated subsurface bed wetland depicted below, name two design flaws.



10.4. Which of the following characteristics apply to free water surface (FWS) wetlands: (1) the water depth is typically 10 ft or more, (2) vegetation can include floating, submerged and emergent plants, (3) internal flow controls are important to avoid short-circuiting, (4) operation requires a pump or siphon for dosing, (5) startup requires months or more of operation, (6) there is a potential concern over mosquitoes.

10.5. Given the characteristics (a) to (e) listed below, check which apply to vegetated subsurface bed (VSB) and/or free water surface (FWS) wetlands.

(a) Influent is normally secondary effluent: VSB____ FWS____ Both_____ Neither____
(b) Requires a pump or siphon for dosing: VSB____ FWS____ Both_____ Neither____
(c) Vegetation is important to treatment: VSB____ FWS____ Both_____ Neither____
(d) Startup often requires only 1–2 months: VSB____ FWS____ Both_____ Neither____
(e) Potential concern over mosquitoes: VSB____ FWS____ Both_____ Neither____

10.6. A vegetated subsurface bed wetland is being designed to treat the effluent from an aerobic treatment unit that serves a mixed-use development of homes and small businesses. The ATU effluent is expected to have a $BOD_5 = 50$ mg/L and TSS $= 75$ mg/L. Using an areal loading rate (ALR) method, will the ATU effluent BOD_5 or TSS determine the size of the surface area required for a VSB wetland to produce a BOD and TSS of 30 mg/L?

10.7. A vegetated subsurface bed wetland is being designed to handle a daily flow of 10,000 gal/day at a maximum hydraulic loading rate of $\leq$0.5 gal/day/ft^2. The landscape area available at the site would only permit the installation of a wetland with $\leq$32,300 ft^2 in surface area. Does this site have adequate area for installation of a VSB wetland to handle the daily flow?

10.8. A constructed wetland is being designed to treat a design flow of 10,800 gal/day. It was determined that a vegetated subsurface bed wetland used to polish the effluent from an aerobic treatment unit would have a horizontal surface area of 12,800 ft^2. If the VSB has porous media with a saturated hydraulic conductivity of 50,000 ft/day and a slope of 0.5 %, what is the minimum cross-sectional area (ft^2) required to process the daily flow through the treatment zone of the VSB?

10.9. A vegetated subsurface bed wetland is being designed to treat septic tank effluent from an apartment building with a design flow, Q_D, of 3000 gal/day. If the VSB has gravel media with $K_S = 50{,}000$ ft/day, what is the minimum width (ft) required to process Q_D through the VSB ($d_W = 1.6$ ft, $F = 0.1$, $S = 0.5\,\%$)?

10.10. A vegetated subsurface bed wetland is being designed to handle a maximum daily flow of 8000 gal/day. Using a PTIS sizing method, it was determined that $q = 0.4$ gal/day/ft^2. The landscape area available for installation of a VSB at the site is 100 ft wide by 330 ft long. Does this site have adequate area for installation of a VSB wetland with an aspect ratio of 3:1 that could be used to treat 8000 gal/day?

10.11. A free water surface wetland is being designed to serve a lakeshore development of homes and small businesses. Based on the information given below, what is the FWS surface area required (in ft^2) to ensure that the wetland meets its discharge permit, which is $BOD_5 \leq 20$ mg/L throughout the year?

Given information and assumed values: Maximum daily flow = 12,000 gal/day influent to the FWS is expected to have a $BOD_5 = 150$ mg/L. Monthly average temperatures during the year vary from 10 to 20 °C. FWS design details: $d_w = 2$ ft, $\varepsilon = 0.7$, and P-k_A-C* values from Table 10.7.

10.12. The Town of Silver Pass, Colorado has been using a free water surface constructed wetland to reliably treat 20,000 gal/day of influent and has routinely achieved 90 % removal of influent BOD_5, which averages 150 mg/L. The existing wetland has a surface area of 180,000 ft^2 and a water depth, $d_W = 4$ ft. A proposed commercial development is being considered for a location adjacent to the Town and the developers are requesting that the town receive and treat the wastewater from the development. It is estimated that the proposed development will generate a flow of 8000 gal/day with an anticipated BOD_5 of 240 mg/L. Will the existing FWS wetland be able to treat the increased wastewater flow and still produce an effluent with $BOD_5 \leq 30$ mg/L?

10.13. A resort village consisting of homes and small businesses is being developed and will be served by a STEG system that will convey effluent for treatment in a VSB wetland. The effluent from the VSB will be used for surface irrigation of a nearby woodland area (fenced area with little risk of human contact). The daily flow reflecting seasonal usage and the temperature of wastewater entering the VSB have been estimated as shown in the table below. The influent from the STEG system is expected to have a $BOD_5 = 200$ mg/L. The discharge permit for the wetland requires that the effluent BOD_5 must

be $\leq$20 mg/L throughout the year. What is the VSB surface area required (in ft^2) to ensure that the BOD_5 is $\leq$20 mg/L?
Given information and assumed values: VSB treatment media has a total media depth = 2 ft with an effective porosity = 0.4 and water depth = 1.5 ft

Period		Influent flow through the VSB (gal/day)	Water temperature in the VSB (°C)
1	January to March	5000	10
2	April to May	6500	15
3	June to September	10,000	20
4	October to December	7500	15

10.14. For the Mines Park housing development located on the Colorado School of Mines campus in Golden, Colorado you are tasked with preliminary design of a VSB constructed wetland. The VSB should be capable of producing a high quality effluent ($BOD_5 \leq 20$ mg/L) that would be suitable for disinfection and reuse for lawn and garden irrigation around Mines Park. Based on the information given below, answer the following design questions. (1) Using a PTIS modeling approach, calculate the VSB surface area (ft^2) required for BOD_5 removal. (2) Determine the effective hydraulic retention time for the VSB. (3) Check the BOD_5 organic loading rate (lb/day/1000 ft^2) to verify it is within generally accepted levels. (4) Determine the width and length (ft) of the VSB for an aspect ratio of 3:1. (5) For the width determined in (4), calculate the capacity for flow and verify that it is adequate for the design flow. If it is not, adjust the width to that required and check the new L:W ratio. (6) For comparison purposes, calculate the VSB surface area (ft^2) required for BOD_5 removal based on an ALR method. (7) How do the VSB surface areas compare (PTIS vs. ALR) and what area do you recommend and why?
Given information and assumed values: Average daily flow, $Q_A = 28{,}425$ gal/day. Design flow is based on the recurring maximum daily flow (PF = 2.0). A STEP collection system will convey septic tank effluent to the treatment site. The STE quality expected: pH = 6, COD = 220 mg/L, BOD_5 = 160 mg/L, TSS = 80 mg/L, TKN = 60 mg/L. Based on general practices, the VSB treatment bed will be made with 40-mm diameter gravel (n = 0.40, clean media K_S = 65,600 ft/day; F = 0.1); the VSB bed depth will be 2 ft with a water depth of 1.5 ft and a maximum VSB hydraulic gradient of 0.005 ft/ft

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[1]References cited in Chap. 10 are listed along with other references that have content relevant to the topics covered in Chap. 10.

Slides of Chapter 10

Decentralized Water Reclamation

Chapter 10: Treatment Using Constructed Wetlands

Contents

10.1

10-1. Introduction

- Constructed wetlands as a treatment unit operation
  - Constructed wetlands used as a treatment unit operation in decentralized systems are designed to exploit the water renovation characteristics of natural wetland ecosystems
  - Impaired water, wastewater, or sludge can be delivered as the influent to a constructed treatment wetland
    - Influent delivery to the wetland can be passively done using gravity or via intermittent pumping
    - Influent is distributed into the inlet of the wetland and flows through it to an outlet end
  - Treatment occurs during long hydraulic retention times (e.g., days) primarily due to biological treatment and plant-based processes
  - A constructed treatment wetland can also provide wildlife habitat and other aesthetic benefits

10.2

- Constructed treatment wetlands are among a variety of constructed land-based plant and water systems (Fig. 10.1)
  - ○ Constructed treatment wetlands can be classified by water surface position, flow direction, and type of vegetation

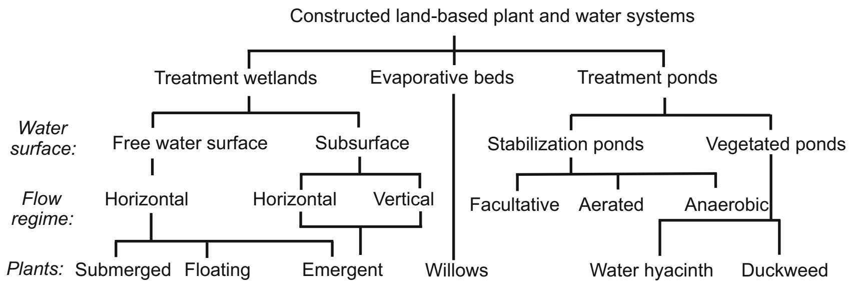


Fig. 10.1 Classification of constructed treatment wetlands within the constructed land-based plant and water systems

10.3

- There are three major types of constructed treatment wetlands
  - ○ Horizontal flow free water surface wetlands (FWS)
    - * Used for stormwaters, acid mine drainage, oil and gas co-produced waters, wastewaters
    - * For wastewater applications, FWS wetlands are typically used for tertiary polishing of secondary effluents discharged from wastewater treatment plants
  - ○ Horizontal flow subsurface vegetated bed wetlands (VSB)
    - * Used for small flows wastewater applications, VSBs typically receive primary effluent as the influent and produce a secondary effluent
  - ○ Vertical flow subsurface vegetated bed wetlands (VVSB)
    - * Used for treatment of high strength wastewaters and sludge drying and treatment
- The features of the three types of wetlands are summarized in Table 10.1 and highlighted in the following pages

10.4

Table 10.1 Summary of the key features of the major types of constructed treatment wetlands

Characteristics	Horizontal flow free water surface (FWS)	Horizontal flow subsurface bed (VSB)	Vertical flow subsurface bed (VVSB)
Operation mode	Horizontal flow, passive	Horizontal flow, passive	Vertical flow, active (downflow, upflow, fill and drain)
Common treatment applications	Stormwaters, mine waters, groundwater, leachate	Domestic wastewaters	High strength wastewaters, sludges
Typical influent	Polishing of secondary or tertiary effluent	Primary effluent	Raw wastewaters, primary effluents
Climate constraints	None, but cold climates req. design adj.	Operate under colder climates than FWS	?
Human exposure	Potential	Not likely	Potential during loading
Attraction for wildlife	Yes (Insects, mollusks, fish, amphib., reptiles, birds, mammals)	Potentially (Larger VSBs can attract wildlife due to vegetation)	Potentially (Surface is intermittently wet and dry which deters wildlife)
Ancillary benefits?	Wildlife habitats, aesthetics	Vegetation aesthetics	Vegetation aesthetics
Cost relative to alternatives?	Capital cost is high due to land area; low O&M	Capital cost is high due to media; low O&M	?

10.5

- Features of a typical FWS wetland are shown in Fig. 10.2
  - Water surface is visible over most of the wetland, with open water zones and densely planted zones
  - Wastewater enters the wetland and flow is horizontal from an inlet zone to an outlet zone, primarily through an open water zone that benefits treatment by aeration, volatilization, and photodegradation processes
  - Vegetation can be emergent, floating, and/or submerged

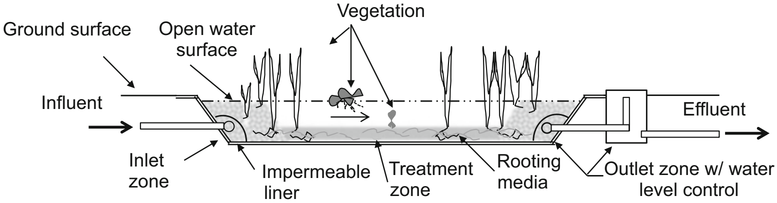


Fig. 10.2 Features of a typical horizontal flow free water surface constructed wetland

10.6

- Features of a typical VSB wetland are shown in Fig. 10.3
  - Water surface is below the top of the VSB and is not visible
    - You can walk across a subsurface flow wetland
  - Wastewater enters the inlet of the wetland and flow is horizontal through a saturated bed of planted porous media
  - Vegetation is dominantly emergent vegetation

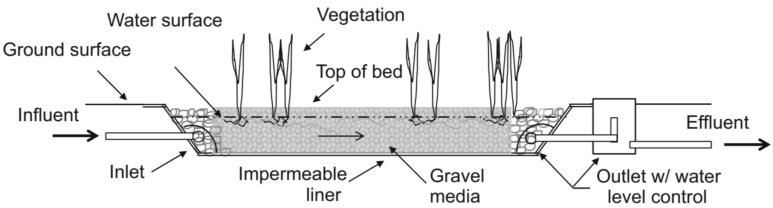


Fig. 10.3 Features of a typical horizontal flow subsurface bed constructed wetland

10.7

- Features of a typical VVSB wetland are shown in Fig. 10.4
  - A VVSB wetland is a single pass, multimedia filter with vegetation
  - Intermittent dosing of wastewater (or sludge) over the filter surface
  - Flow is intermittent and vertical from the surface to an underdrain by downflow or upflow saturation, with holding and then drainage

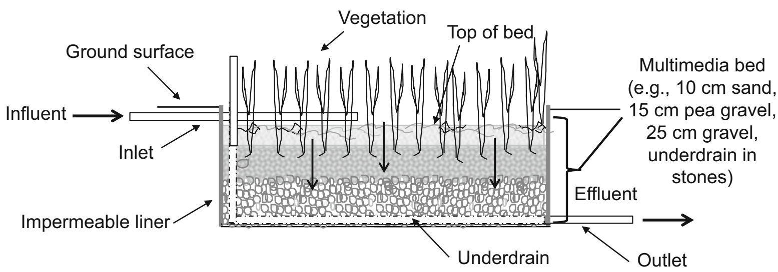


Fig. 10.4 Features of a typical vertical flow subsurface bed constructed wetland

10.8

- Where are constructed treatment wetlands used?
  - Where there is adequate land area available and a desire to utilize passive, natural treatment systems
  - FWS wetlands have been used for tertiary polishing of secondary effluents to enable surface discharge and reuse of effluent (the wetland effluent may have to be disinfected first)
  - VSB wetlands have been used to produce a secondary effluent often for discharge to a decentralized land-based treatment unit
  - VVSB wetlands have been used for sludge dewatering
- This chapter covers FWS and VSB constructed wetlands as used for treatment of wastewaters and other impaired waters

Note: From hereon in Chap. 10, constructed treatment wetlands will be referred to simply as constructed wetlands

10.9

10-2. Treatment Performance

- Constructed wetlands are normally designed to achieve secondary treatment of primary effluents or tertiary polishing of secondary effluents
  - BOD and TSS removal
    - Typically occurs through biological processes (BOD) and sedimentation and filtration (TSS)
  - N and P removal (depends on design and environment)
    - N removal can occur via biological processes and also plant uptake and volatilization
    - Phosphorus removal can occur by plant uptake[a] and sorption
  - Other pollutants and pathogens
    - Removal (to some extent) is possible by various processes
    - The open water surface in a FWS wetland enables volatilization and photodegradation processes

[a]*Note:* removal by plant uptake is often only considered true removal if the plants are harvested and removed.

10.10

- Treatment efficiency
  - Removal efficiency (R_E) as illustrated in Fig. 10.5 can be calculated for a VSB or FWS using Eq. 10.1
  - Treatment efficiencies for constituents of potential concern within a FWS or VSB wetland are presented in Table 10.2

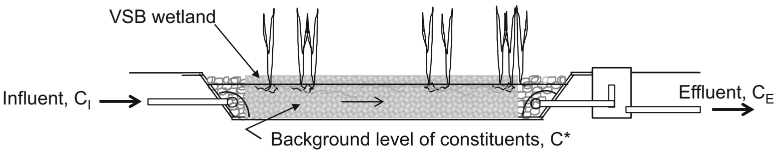


Fig. 10.5 Illustration of treatment efficiency achieved in a VSB constructed wetland

$$R_E = \left(\frac{(C_I + C^*) - C_E}{C_I + C^*}\right) \times 100\% \qquad (10.1)$$

Where:
R_E = removal efficiency for C_I (%)
C_I = influent concentration (mg/L)
C^* = background constituents (mg/L)
C_E = effluent concentration (mg/L)

10.11

Table 10.2 Representative treatment efficiency achieved within a well designed and operated constructed wetland[a]

Constituent group	Effluent mg/L or % removal[a]	Potential processes involved in treatment
BOD_5	<20–30 mg/L	Dissolved organics removal by biotransformation and particulate organics by sedimentation, filtration, biodegradation
TSS	<20–30 mg/L	Removed by sedimentation and filtration
Nitrogen[b]	Var. NH_4^+ rem. Var. total N rem.	Depending on design, potential for nitrification of NH_4^+ compounds to NO_3^- with potential removal of total N by biodenitrification; plant uptake of N and removal through plant harvesting
Phosphorus	10–20 % total P	Incorporation of P into cell mass and sorption, plant uptake and removal through harvesting; sedimentation and filtration of particulate P
Pathogens	99–99.99 %	Decay, predation, phyto-inactivation, photo-inactivation
Trace organics	0 to >90 %	Near zero removal for some compounds but up to 90 % or more removal of compounds that are susceptible to sorption, volatilization, phyto- and phyto-degradation

[a]In general, FWS wetlands produce higher quality effluents since they are normally used for tertiary polishing of secondary effluents [b]To achieve high removal efficiencies for NH_4+ and total nitrogen in a VSB, it is important to have adequate alkalinity in the wetland influent to support nitrification and a subsequent anoxic zone for biodenitrification.

10.12

- Constructed wetland effluent composition
  - Factors affecting treatment efficiency and effluent composition
    - Wetland type, surface area and volume provided to handle the design flow rate and remove constituents of concern
    - Establishing and maintaining a plug-flow like regime through the wetland and avoiding short-circuiting and inactive flow zones
    - Suitable local hydrologic and climatic conditions which are properly accounted for in system design and operation
  - Modifications for enhanced treatment purposes
    - Effluent recirculation instead of single pass flow
      - Ability to recirculate partially or fully treated effluent may offer some modest improvements to performance, particularly in VSB wetlands
      - Recirculation may add dissolve oxygen (DO) and reduce BOD and TSS concentrations in the wetland effluent

10.13

    - Active aeration instead of passive aeration
      - Aeration can be used to increase DO levels
        - Using compressed air delivery to increase DO levels and improve removal of BOD
        - Can create aerobic and anoxic zones and increase removal of nitrogen
    - Use of reactive media in place of gravel in a VSB
      - Media with surface reactivity can be used
        - e.g., aggregates with high sorption for phosphorus
    - Hybrid wetland systems
      - Hybrid wetland systems can improve overall performance
      - For example to polish BOD removal and enable improved denitrification, hybrids could include:
        - Use of a VSB followed by VVSB
        - Use of a FWS followed by a VSB

10.14

10-3. Principles and Processes

- Natural wetlands in the environment
  - Natural wetlands are located in land areas with hydric soils that are wet during part or all of the year
    - Historically, wetlands have been referred to as:
      - Swamps, Marshes, Bogs, Fens, Sloughs
    - All wetlands are 'wet' long enough each year that they exclude plant and vegetation species that cannot grow in saturated settings
  - Natural wetlands are complex ecosystems wherein a range of processes can occur, including:
    - Hydrologic and hydraulic processes
    - Purification processes: plant, microbial, physical, chemical
  - Natural wetlands have the potential to transform common pollutants to harmless or even beneficial products

10.15

- Constructed wetlands
  - Constructed wetlands are designed to exploit many of the same processes that occur in natural wetlands
  - However, system design and operation allows greater control over natural processes to help achieve a desired treatment performance from a particular type of wetland
    - FWS wetlands and VSB wetlands have different applications and similar as well as different processes and performance attributes
  - Depending on the type of wetland (FWS vs. VSB), concerns can vary regarding:
    - Access
    - Wildlife intrusion
    - Mosquito habitat
    - Operation and maintenance

10.16

- Vegetation in constructed wetlands
  - Vegetation can have multiple functions in constructed wetlands, the nature and extent of which depend on the wetland type and setting
    - Physical
      - Water transpiration, flow resistance, particulate removal
    - Chemical
      - Growth cycle and uptake and release of nutrients
      - Generation of stable residuals
      - Provision of surfaces for microbes to grow on
      - Oxygen supply effects including blocking wind, shading the water surface, photosynthesis, oxygen flux to roots
      - Provision of bioavailable carbon for microbial processes
    - Ecological
      - Wildlife habitat and aesthetics

10.17

- Different types of vegetation can be present in wetlands
  - Macrophytes (Fig. 10.6)
    - Emergent (e.g., Bullrush, Common reed, Cattail)
    - Floating (e.g., Water lily, Duckweed, Water hyacinth)
    - Submerged (e.g., Pondweed, American shoreweed)
  - Woody species
    - Shrubs and trees (e.g., willow trees)

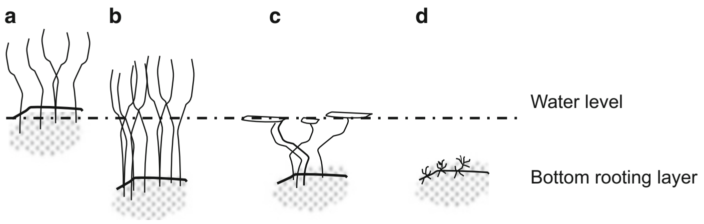


Fig. 10.6 Illustration of emergent macrophytes in a VSB (**a**) and emergent macrophytes (**b**), floating macrophytes (**c**) and submerged macrophytes (**d**) in a FWS

10.18

- Adaptation of vegetation in a constructed wetland
  - As constructed wetlands are started up, conditions evolve due to the fact that the influent is wastewater (Table 10.3)
    - Vegetation must adapt to these conditions

Table 10.3 Wetland characteristics and conditions imposed by wastewater influent

Wetland characteristics	Comparison of natural vs. treatment wetlands	
	Natural wetland	Treatment wetland
Influent BOD, N, P	Low	High
Plant growth and biomass	Limited by lack of nutrients	Not limited by nutrients and more plant biomass can be present
Plant depth and rooting	Plants can grow deep to access nutrients	Plant penetration is more limited and rooting is shallow
Water column depth in a FWS or media bed depth in a VSB	Depth is not limited by DO due to O_2 transport via plants and low demand	Depth is limited: FWS—Limited due to low O_2
DO in the FWS sediment or VSB media	Aerobic to less reducing	VSB—Lower portion of bed becomes anoxic or anaerobic

10.19

- Hydrology and hydraulics in a constructed wetland
  - Hydrology and hydraulics determine the:
    - Ability of the wetland to handle the design daily flow without backup or surface overflows
    - Achievement of flow conditions that approximate a plug-flow regime without short-circuiting
    - Hydraulic retention time and conditions for removal processes to function and produce a desired effluent quality
  - To achieve the treatment performance potential of a constructed wetland. . .
    - It is critical that design provides for, and operation maintains, the desired hydrology and hydraulic conditions

10.20

- In horizontal flow wetlands—both FWS and VSB designs—the hydraulics are generally determined by key features:
  - Geometry of the wetland
    - Surface area
    - Aspect ratio (length to width)
    - Water depth (*Note*: typically 1–5 ft (higher end in FWS))
  - Hydraulic gradient from inlet to outlet
  - Flow controls
    - Inlet and outlet design
    - Internal flow controls
- In addition, for horizontal flow in a VSB wetland
  - Hydraulics can also be determined by the saturated hydraulic conductivity (K_S) of the clean porous media used and the decline from K_S that occurs due to operation (Eq. 10.2, Fig. 10.7)

10.21

$$Q_C = (K_E)(S)(A_{xc}) \tag{10.2}$$

Where:
Q_C = flow through the wetland (gal/day)
K_E = effective saturated hydraulic conductivity of porous media (gal/day/ft^2)
K_E equals the operational conductivity which is a fraction of K_S
S = hydraulic gradient of water surface from inlet to outlet (–)
A_{xc} = cross-sectional area through which flow occurs (ft^2)

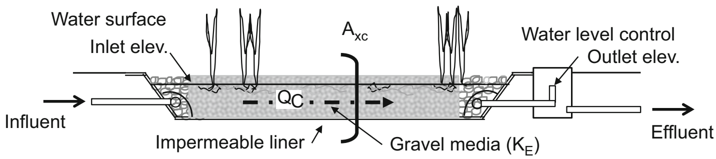


Fig. 10.7 Cross section of a vegetated subsurface bed wetland which has a capacity for flow

10.22

- The hydraulic effects of vegetation growth and pore clogging
  - Vegetation growth
    - Leads to root networks and plant parts that can occupy space and reduce hydraulic capacity
  - Pore clogging
    - Shorter-term processes
      - Establishment of plant root networks within pores
      - Filtration of TSS and development of microbial biomass
    - Longer-term processes
      - Deposition of inert (mineral) suspended solids
      - Accumulation of refractory organic materials
      - Formation of chemical precipitates
  - Hydraulic effects due to vegetation and pore clogging are most important to the function of VSB wetlands (Fig. 10.8) and much less so for FWS wetlands

10.23

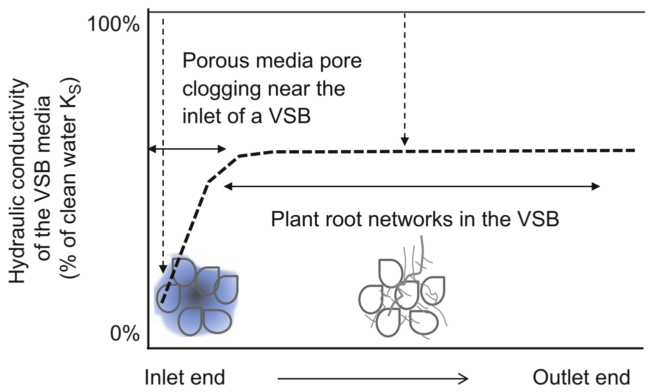


Fig. 10.8 Illustration of how the hydraulic conductivity of the media in a VSB constructed wetland can change from inlet to outlet due to clogging and vegetation (as a percentage of the pre-startup K_S)

10.24

- Effective hydraulic conductivity is used for VSB design
  * The effective hydraulic conductivity for VSB design (K_E) accounts for clogging processes
  * Clogging within the inlet zone of a VSB
    – If the hydraulic capacity declines below the daily loading rate, the inlet of a VSB can become saturated and ponding may occur
    – Influent that ponds at the inlet can re-infiltrate into the porous media within the VSB a short distance away from the inlet This may or may not be viewed as an unacceptable condition
  * Clogging within the treatment zone of a VSB
    – Clogging by vegetation growth can occur but plant decay can yield a stable condition where K_E can be about 50 % of the K_S
- Choosing a K_E for design of a VSB?
  – USEPA (2002) recommends setting $K_E = 1\%$ of K_S in the initial 30 % and $K_E = 10\%$ in the balance of the treatment zone
  – Jenssen (2015) recommends using an overall value of $K_E = 50\%$ of K_S for sizing of VSBs used for residential applications

10.25

- Redox zones within a constructed wetland
  - Constructed wetlands can have regions or zones that are aerobic, anoxic, or even anaerobic
    - Dissolved oxygen sources and sinks determine redox conditions (Table 10.4)

Table 10.4 Wetland processes and their effects on dissolved oxygen levels[a]

Wetland process	Process effects on DO
Reaeration	Adds DO
Photosynthesis	
Plant O_2 transpiration	
Plant respiration	Consumes DO
Plant decomposition	
Biodegradation of influent BOD	
Biotransformation of influent NH_4-N	

[a]The different processes can have different impacts on DO levels depending on the type of wetland and conditions present during operation.

10.26

- Water temperatures in a constructed wetland
  - Temperature is important since it can influence the rates of bioprocesses, the rate of evaporative loss, and water freezing
  - Wetland water temperatures depend on local climate conditions
    - Daily cycles can be up to 8–10 °C in warm months
    - Annual cycles
      - In mild to warm climate conditions
        - Summer maximum and winter minimum
        - Wetland water temperatures approximate the mean daily air temperatures (under moderate humidity and air temperature)
      - In cold climate conditions
        - During winter periods, ice can develop
        - Ice as well as snow and mulch can help insulate water in the wetland from freezing
        - However, under-ice water temperatures can be reduced as low as the 1–2 °C range

10.27

- Animals and insects in constructed wetlands
  - Animals and insects can inhabit or visit a constructed wetland
    - The types of animals and insects will be much the same as those that inhabit or visit a natural wetland in the same location
    - Some animals and insects are desirable (e.g., deer, birds, beetles) but others are often not (e.g., snakes, rodents, mosquitos)
    - Constructed wetland design and operation can influence to some extent, whether the animals and insects that inhabit it will be a positive or negative attribute
  - For VSB constructed wetlands
    - The biggest problems tend to occur in locations where there are undesirable animals (e.g., poisonous snakes, nasty rodents) in the natural environment that can also inhabit and thrive in a VSB wetland
      - This situation is difficult if not impossible to control through VSB design and operation

10.28

- For FWS constructed wetlands
  - In some FWS wetlands there can be undesirable animals (e.g., poisonous snakes and dangerous reptiles) and noxious insects, most notably mosquitos
  - Mosquitos will inhabit most FWS wetlands but their numbers can be controlled by design and operation
    - Control larvae generation that yields mosquitos
    - Need to foster predator access to larvae
      - Increase open and deep pool water areas
      - Avoid large monotypic stands of emergent vegetation
      - Add birdhouses
  - However, in many situations it is difficult to design and operate a FWS wetland so that it is mosquito-free
    - An achievable goal? Minimize mosquito production so it is similar to natural wetlands in the same location

10.29

10-4. Design and Implementation

- Considerations for design and implementation (D&I) of a constructed wetland to achieve secondary treatment and partial nutrient removal or tertiary polishing
  - Wetland type features and suitability for a particular project
  - Wastewater source and treatment prior to the wetland
  - Wetland surface area sizing to handle the daily flow
    - Use of loading specifications
    - Use of modeling techniques
    - Use of a combination of the two
  - Wetland geometry, porous media, flow controls, and other construction details
  - Wetland establishment and startup
  - Wetland O&M

10.30

- D&I considerations—Constructed wetland type
  - FWS and VSB constructed wetlands have different features (Table 10.1) that lend one or the other to particular project applications (Table 10.5)

Table 10.5 Applications of FWS versus VSB constructed treatment wetlands

Wetland type	Comments concerning typical applications
FWS	Used for treatment of impaired waters like stormwater, acid mine drainage, oil and gas co-produced waters, or secondary wastewater effluents. FWS wetlands require adequate land area often in a more remote location where an open water surface and wildlife habitat are acceptable. FWS wetlands may require access controls for safety reasons. Depending on the situation, effluent from the wetland may be discharged to the environment or used as reclaimed water.
VSB	Mostly used for treatment of primary wastewater effluents, often for smaller flows from residential or commercial sources. VSB wetlands can be used in populated areas and normally do not require access controls. VSB wetlands require a local source of gravel media for the wetland bed. A VSB wetland produces a secondary effluent that may need further treatment before it is discharged to the environment or used as reclaimed water.

10.31

- D&I considerations—Wastewater source and treatment prior to the wetland
  - Wastewaters from varied sources can be treated
    - e.g., graywater, residential wastewater, commercial wastewater
  - Treatment prior to a wetland
    - The influent to a FWS is typically secondary effluent
      - e.g., nitrified effluent from an aerobic treatment unit
    - The influent to a VSB is typically primary effluent
      - e.g., effluent from a septic tank or sedimentation basin
      - Secondary effluents can also be polished in a VSB wetland
  - Delivery of influent to a wetland
    - Influent can be delivered to the inlet end of a wetland by gravity flow or through pressurized delivery

10.32

- D&I considerations—Wetland surface area required
  - Wetlands need to be sized with sufficient surface area so they can achieve treatment of one or more constituents of concern
    - For example, produce an effluent with BOD and TSS $\leq$ 30 mg/L
  - Surface area sizing is primarily based on
    - Design hydraulic loading rate (HLR_D)
    - Organic loading rate (OLR) limits
  - Approaches for wetland area sizing
    - Different approaches have been used for surface area sizing of different types of wetlands
    - Two common approaches for sizing are discussed in this section
      - Area sizing using empirical data from past experiences
      - Area sizing using modeling of constituent removals

10.33

- Wetland area sizing—Areal Loading Rates (ALRs)
  - Based on experience, a maximum loading rate per unit of wetland surface area can be specified that is expected to yield a certain effluent quality
    - For example, for a VSB wetland to achieve an effluent BOD = 20 mg/L, one ALR is $\leq$0.33 lb-BOD/day/1000 ft^2
  - The ALR method is similar to the approach used for sizing oxidation ponds, lagoons, and land treatment units
  - The ALR method works reasonably well for design of wetlands to yield a given effluent BOD_5 and TSS, such as:
    - BOD and TSS = 30 mg/L
    - BOD and TSS <20 mg/L and some nutrient removal

10.34

- Example ALR values prescribed to achieve a given effluent quality with respect to BOD or TSS are shown in Table 10.6
  * ALRs are based on the entire wetland horizontal surface area

Table 10.6 Areal loading rates to achieve different effluent qualities (after USEPA 2002)

Parameter	Expected effluent quality (mg/L)	FWS wetland		VSB wetland	
		lb-BOD/day/1000 ft^2	g-BOD/day/m^2	lb-BOD/day/1000 ft^2	g-BOD/day/m^2
BOD	20	0.92	4.5	0.33	1.6
	30	1.23	6	1.23	6
TSS	20	0.62	3	–	–
	30	1.02	5	4.09	20

Note: 1 g/day/m^2 = 0.205 lb/day/1000 ft^2.

10.35

- For an ALR design the wetland surface area (Fig. 10.9) is calculated using Eq. 10.3

$$A_w = \frac{(Q_D)(C_I)(F)}{(ALR)} \tag{10.3}$$

Where:
A_W = total surface area of the wetland (L × W) (ft^2)
ALR = areal loading rate for a constituent (e.g., BOD) (lb/day/ft^2)
C_I = influent concentration (mg/L)
Q_D = design daily flow rate (gal/day)
F = 8.34×10^{-6} = conversion factor for mg/L to lb/gal

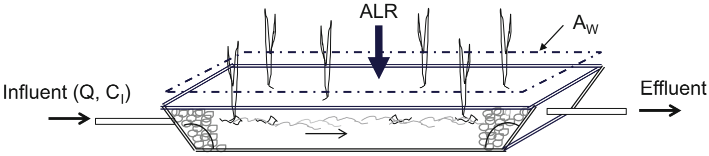


Fig. 10.9 Illustration of the wetland surface area determined by loading rate sizing

10.36

- Wetland area sizing—Modeling of constituent removals
  - Constituent removal in a wetland can be modeled assuming reactions are occurring during plug-flow like water movement
    - * A plug-flow like regime can be simulated using a number of continuously stirred tank reactors in series (Fig. 10.10)
    - * Modeling can account for hydraulic inefficiencies and changes in reaction rate constants with distance in the wetland
  - The P-k_A-C* modeling approach is discussed in this section

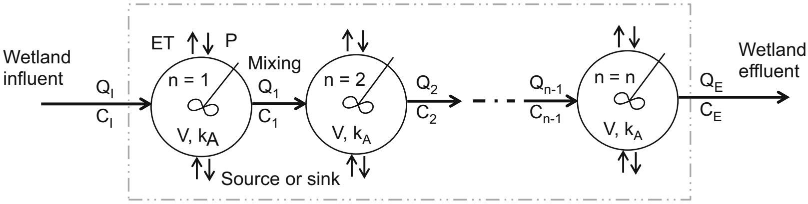


Fig. 10.10 Illustration of a mathematical representation of a pseudo plug-flow regime through a constructed wetland using multiple continuously stirred tank reactors in series

10.37

- The P-k_A-C* modeling method accounts for a departure from true plug flow due to hydraulic inefficiencies plus a declining reaction rate constant with distance from the inlet to the outlet in the wetland
  - * Steps in determining the wetland surface area required[a]
    - – Select concentration(s) in the influent to the wetland and appropriate background concentrations
    - – Choose target effluent concentrations (e.g., BOD = 20 mg/L)
    - – Analyze potential water balance additions or deletions to flow (inflow and seepage, rain and evapotranspiration)
    - – Select rate constants and consider temperature effects
    - – Select hydraulic efficiency values (e.g., P-values)
    - – Calculate the areal hydraulic loading rate and the resulting wetland surface area required
    - – Choose other sizing parameters (water depth, geometry, etc.)
    - – Iterate as necessary until effluent concentrations are met
    - – Consider constraints, if any, such as growth cycles, biogeochemical cycles, etc.

[a]Based on methods in Kadlec and Wallace (2009).

10.38

- In the P-k_A-C* modeling method, Eq. 10.4 can be used to determine an area-based loading rate, q

$$\left(\frac{C - C^*}{C_I - C^*}\right) = \left(1 + \frac{k_A y}{Pq}\right)^{-P} \tag{10.4}$$

Where:
C = concentration of a constituent at the outlet or a fractional distance from the inlet, y (mg/L)
C_I = inlet concentration of a constituent (mg/L)
C* = background concentration of a constituent (mg/L)
k_A = first-order area-based rate constant (ft/day)
P = apparent no. of tanks in series for modeling that varies by constituent to account for weathering based on field data (–)
q = area-based design hydraulic loading rate (HLR) (ft/day or ft^3/ft^2/day)
y = fractional distance from the inlet to outlet, unitless (for effluent, y = 1.0)

- Tables 10.7 and 10.8 list input parameter values for P-k_A-C* modeling of FWS and VSB wetlands, respectively

10.39

Table 10.7 P-k_A-C* values for modeling of FWS wetlands (after Kadlec and Wallace 2009)

Pollutant transformation/removal		P (–)	C* (mg/L)	Median k_A (ft/day)	Median θ (–)
BOD	Tertiary inf. (0–30 mg/L)	1	2	0.30	0.99 (T.8.4)
	Secondary inf. (30–100 mg/L)	1	5	0.37	
	Primary inf. (100–200 mg/L)	1	10	0.32	
	Super inf. (>200 mg/L)	1	20	1.70	
N	Ammonification	3	1.5	0.16	1.0 (p. 300)
	Nitrification	3	0	0.13	1.049 (T.9.29)
	Denitrification	3	0	0.24	1.102 (T.9.40)
	Kjeldahl nitrogen	3	1.5	0.09	1.036 (T.9.16)
	Total nitrogen	3	1.5	0.11	1.056 (T.9.20)
Total phosphorus		3.4	0.002	0.09	1.006 (T.10.12)
Fecal coliforms		3	40	0.75	0.963 (T.12.5)

Source: P-k_A-C* values are from Table 16.11. Kadlec and Wallace. 2009. θ values are from the table given in parenthesis or page no. in the same source.

10.40

Table 10.8 P-k_A-C* values for modeling of VSB wetlands (after Kadlec and Wallace 2009)

Pollutant transformation/removal		P (–)	C* (mg/L)	Median k_A (ft/day)	Median θ (–)
BOD	Tertiary (0–30 mg/L)	3	1	0.77	0.981 (T.8.10)
	Secondary (30–100 mg/L)	3	5	0.33	
	Primary (100–200 mg/L)	3	10	0.22	
	Super (>200 mg/L)	3	15	0.59	
N	Ammonification	6	1	0.18	1.009 (T.9.13)
	Nitrification	6	0	0.10	1.014 (T.9.32)
	Denitrification	8	0	0.38	1.102 (p.342)
	Kjeldahl nitrogen	6	1	0.08	1.001 (T.9.17)
	Total nitrogen	6	1	0.08	1.005 (T.9.21)
Total phosphorus		[a]	[a]	[a]	–
Fecal coliforms		6	0	0.93	1.002 (T.12.21)

Source: P-k_A-C* values are from Table 16.11. Kadlec and Wallace. 2009. θ values are from the table given in parenthesis or page no. in the same source.
[a]Indicates insufficient data to determine parameter.

10.41

- Equation 10.4 can be rearranged to solve for the area-based HLR to yield a certain pollutant removal (Eq. 10.5)

$$q = \frac{k_A y}{P\left[\left(\frac{1}{(C-C^*)/(C_I-C^*)}\right)^{1/P} - 1\right]} \tag{10.5}$$

- The required wetland surface area can be determined from the calculated area-based hydraulic loading rate using Eq. 10.6

$$A_W = \frac{Q_D}{q} \tag{10.6}$$

Where:
A_W = wetland surface area (wetted land area) (ft^2)
Q_D = design flow rate (ft^3/day)
q = area-based hydraulic loading rate (HLR) (ft/day)

10.42

- Other considerations in modeling for wetland area sizing
  - Water balance effects
    - * In some situations, water gains and losses will be important to consider during wetland sizing
      - – Primary water gain is due to precipitation (P)
      - – Primary water losses are due to evapotranspiration (ET)
    - * For sizing, P and ET can be factored into the calculations made using a modeling approach (e.g., by changing Q within one or more tanks in series)
  - Temperature effects
    - * Biological processes and transformation rates can be temperature dependent
      - – Rates can be adjusted for temperature using an appropriate temperature correction factor, θ
    - * For some processes, low or high temperatures can inhibit reactions
      - – For example, the nitrification process can be greatly retarded or cease at temperatures below 10 °C

10.43

* A common formulation to correct for the temperature effects on biological processes is shown in Eq. 10.7

$$k_T = k_{20}\theta^{(T-20)} \quad (10.7)$$

Where:
k_T = reaction rate at T °C ($days^{-1}$)
k_{20} = reaction rate at 20 °C ($days^{-1}$)
T = temperature (°C)
θ = temperature activity coefficient (–)

Note: In activated sludge biological systems, θ for BOD removal can be about 1.02–1.06 and some adopt values in this range for use with constructed wetlands. However, for constructed wetlands there is evidence that θ for BOD removal may be closer to 1.0. For constituents other than BOD, θ can vary widely (e.g., see Tables 10.7 and 10.8)

10.44

- Wetland area sizing—Checking the organic loading rate
  - OLR is limited to avoid anaerobic conditions and minimize odors
    - Maximum OLRs depend on wetland type and the targeted effluent quality as shown in Table 10.9
    - For A_W determined by the P-k_A-C* method, the OLR can be checked using Eq. 10.8

$$OLR = \frac{(Q_D)(BOD_5)(F)}{A_W'} \tag{10.8}$$

Where:
OLR = organic loading rate (lb-BOD_5/day/ft^2)
For guidance on maximum rates see Table 10.9
Q_D = design daily flow (gal/day)
BOD_5 = Influent BOD_5 (mg/L)
A_W' = wetland surface area actually provided based on the chosen geometry (L × W) (ft^2)
F = 8.34×10^{-6} = conversion factor for mg/L to lb/gal

10.45

Table 10.9 Organic loading rate limits (lb-BOD/day/1000 ft^2) for constructed wetlands

Reference	Type of wetland and target BOD_5 in the wetland effluent			
	FWS wetland		VSB wetland	
	20 mg/L	30 mg/L	20 mg/L	30 mg/L
Crites and Tchobanoglous (1998)	2.25	–	2.25	–
USEPA (2002)	0.90–1.19		0.33	1.23
ITRC (2003)	0.92	1.25	–	–

10.46

- D&I considerations—Water depth and rooting depth
  - In a VSB wetland (Fig. 10.11a)
    - Typical porous media depths are 1.5–2.5 ft to provide for rooting and horizontal water flow
    - The water surface is normally 0.25–0.5 ft below the bed surface
  - In a FWS wetland (Fig. 10.11b)
    - Some depth of media is required for rooting plants (e.g., 1 ft)
    - Water depths in the inlet and outlet zones are often 2–3 ft while the open water surface zone is 4–5 ft

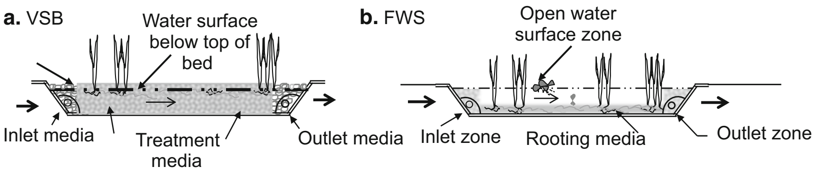


Fig. 10.11 Illustration of water depth and rooting bed depth in constructed wetlands

10.47

- Relationship of water depth to treatment efficiency
  - For a wetland design based on areal loading rates
    - * R_E is based on the areal loading rate and not directly related to HRT (Fig. 10.12)
  - For a wetland design based on modeling with area-based reaction rates where $q = Q_D/A_W$ and R_E is not affected by the HRT
    - * For a given Q_D, q does not change with increases in depth
  - Thus, increasing water depth alone would not be predicted to increase R_E

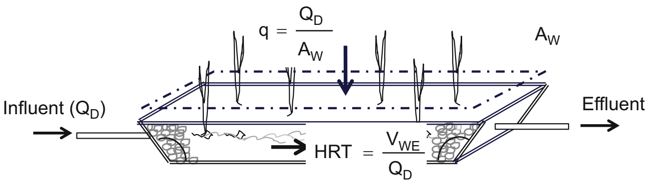


Fig. 10.12 Relationship of design hydraulic parameters in a VSB wetland

10.48

- D&I considerations—Wetland volume and HRT
  - Wetland volume
    - The nominal volume of a wetland is equal to the bulk volume of the wetland defined by the area and water depth (Eq. 10.9)

$$V_N = (A_W)(d_W) \tag{10.9}$$

    - The water-filled volume accounts for the presence of gravel or plants as given by Eq. 10.10

$$V_W = (\varepsilon)V_N \tag{10.10}$$

Where:
V_N = nominal wetland volume is the bulk volume = L × W × d_W (ft^3)
V_W = water volume equals the open porosity in the wetland media (ft^3)
A_W = wetland surface area (ft^2)
d_W = water depth (ft)
ε = porosity of clean gravel (other similar media) in a VSB (ft^3/ft^3)
= plant-based void ratio in a FWS (ft^3/ft^3)

10.49

- Some designers consider an effective water volume that accounts for the presence of gravel or plants plus inactive flow zones using Eqs. 10.11 and 10.12

$$V_{WE} = (e_V)(\varepsilon)V_N \tag{10.11}$$

$$e_V = \left(\frac{V_A}{V_B}\right) \tag{10.12}$$

Where:
V_B = bulk volume of the wetland = L × W × d_W (ft^3)
V_{WE} = effective water volume accounting for porosity and inactive zones (ft^3)
V_A = volume of wetland containing water active in flow (ft^3)
A_W = wetland surface area (ft^2)
d_W = water depth (ft)
ε = porosity of clean gravel in a VSB (ft^3/ft^3) (often assumed 0.40)
= plant-based void ratio in a FWS (reported as 0.65–0.75)
e_V = wetland volumetric efficiency accounting for inactive flow zones (–) (reported as FWS = 0.82, VSB = 0.83)[a]

[a]*Source:* Kadlec and Wallace (2009)

10.50

- Wetland hydraulic retention time
  - With the wetland volume determined, the hydraulic retention time can be calculated using Eqs. 10.13 and 10.14

$$HRT_N = \frac{V_N}{Q} \tag{10.13}$$

$$HRT_E = \frac{V_{WE}}{Q} \tag{10.14}$$

Where:
HRT_N = nominal hydraulic retention time (days)
HRT_E = effective hydraulic retention time (days)
V_N = nominal wetland volume is the bulk volume = $L \times W \times d_W$ (ft^3)
V_{WE} = effective water volume accounting for porosity plus inactive flow zones (ft^3)
Q = flow rate through the wetland—design or actual (ft^3/day)

10.51

- D&I considerations—Wetland porous media
  - A description of flow zones is presented in Table 10.10 and a summary of the media size ranges used is given in Table 10.11

Table 10.10 Features of key flow zones within a VSB constructed wetland

Zone	Function	Media
Inlet	Designed to distribute flow and help establish plug-flow like conditions through the VSB. The TSS loading to the X-C area of the inlet zone and initial portion of the treatment zone is sufficient to avoid excessive clogging and hydraulic failure (e.g., TSS <0.008 lb/day/ft^2)	Clean coarse gravel (e.g., 40 mm) or similar solids in the initial 3–6 ft
Treatment	Designed to support plant root development and biofilm growth	Clean, fine gravels that are uniform in size are typically used (e.g., 10–20 mm diam. pebbles and fine gravels)
Outlet	Designed to capture treated water and direct it out of the VSB	Clean coarse gravel (e.g., 40 mm) or similar in the final 3 ft

10.52

Table 10.11 Characteristics of porous media used in VSB wetlands (after Knowles et al. 2011)

Country or organization	Size (mm)	Source
Austria	0–4 (graywater) 4–8 (primary trt.) 1–4 (tertiary trt.)	ÖNORM (2005)
Czech Republic	<20	Vymazal (1996)
Germany	0.1–1.0 (sand)	GFA (1998)
United Kingdom	10–12	Griffin et al. (2008)
United States	3–6	TVA (1993)
	20–30	USEPA (2000)
European Design Guidelines	3–6	EC/EWPCA (1990)
	6–12	
Intern. Water Assn.	8–16	IWA (2000)

10.53

- D&I considerations—Flow rate capacity of a VSB
  - Flow rate capacity is controlled by the cross-sectional area of the wetland (A_{XC}) and K_E of the porous media after accounting for hydraulic conductivity loss during operation (Fig. 10.13)
  - A minimum A_{XC} is required so the VSB can handle the daily flow without surfacing of water flowing through it

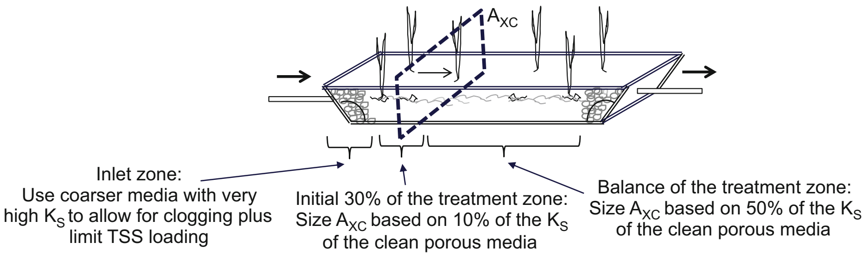


Fig. 10.13 Example approach to cross-sectional area sizing within a VSB wetland

10.54

- The minimum cross-sectional area required can be calculated using Eq. 10.15

$$A_{XC} = (d_W)(W) = \frac{Q_D}{(K_S)(F)(S)} \qquad (10.15)$$

Where:
A_{XC} = cross-sectional area of the wetland required for Q_D (ft^2)
d_W = depth of water in the wetland (ft)
W = width of the wetland (ft)
Q_D = design flow rate (ft^3/day)
K_S = saturated hydraulic conductivity of the clean VSB media (ft/day)
F = factor to account for loss in K_S due to clogging (*Note:* $(K_S)(F) = K_E$) (USEPA (2002) recommends F = 0.01 for the initial 30 % of the treatment zone and 0.10 for the balance of that zone; Jenssen (2015) recommends F = 0.50 for the overall treatment zone)
S = hydraulic gradient from the inlet to outlet (slope) (ft/ft) (If the VSB bottom is sloped, can use that as S (typ. $\leq$0.01) For flat bottoms with outlet control use $S \approx 0.001$)

10.55

- Hydraulic conductivity properties of porous media used within the treatment zone of a VSB wetland
  - Table 10.12 shows example media with hydraulic properties and the flow rate capacities per unit of A_{XC} based on K_E = 10 or 50 % of K_S

Table 10.12 Hydraulic conductivity properties of porous media used in VSB wetlands

Media size (mm)	Media porosity (–)	Saturated hydraulic conductivity of clean uniform media[a], K_S (ft/day)	Effective K_E in the treatment zone after accounting for clogging[b] (ft/day)	Example flow capacities per unit of A_{XC}[c] (gal/day/ft^2)
2	0.32	3200	320–1600	2.45–12.2
8	0.35	16,400	1640–8200	12.2–61
12	0.37	23,000	2300–11,500	17.2–86
32	0.40	32,800	3280–16,400	24.5–122.5

[a]The K_S shown is an approximate value for the media size listed assuming uniform media (e.g., $d_{60}/d_{10} < 4$) [b]The effective K_E is based on 90–50 % loss of K_S due to clogging processes [c]The capacity is based on Eq. 10.15 with F = 0.10 or 0.50 and S = 0.001. 1 L/m^2 days = 0.0245 gal/day/ft^2.

10.56

- D&I considerations—Wetland surface area geometry
  - The length (L) and width (W) can be selected to help achieve plug-flow like conditions and avoid short-circuiting
    - L:W ratios of 1:1 to 4:1 are generally desirable
      - * L:W ratios apply to the entire wetland or individual cells within it if flow control boundaries are used (Fig. 10.14)
    - Length and width can yield a desired aspect ratio (R_A) (Eq. 10.16)

$$R_A = \frac{L}{W} \tag{10.16}$$

Where:

$W =$ width of the wetland or a cell within it (ft)

W has to be sufficient so the cross-sectional area ($A_{XC} = W \times d_w$) is $\geq$the size needed to handle Q_D (Eq. 10.15); but limited to avoid short circuiting (e.g., $W \leq 200$ ft)

$L =$ length of the wetland or a cell within it (ft)

$R_A =$ aspect ratio, length to width of the wetland or cell within it (e.g., 1:1 to 4:1)

10.57

- Flow controls can be used to improve wetland hydraulics
  - Vertical flow controls can be used to divide the wetland into cells (Fig. 10.14)
    - * Horizontal flow controls can be used if density driven flow is of concern

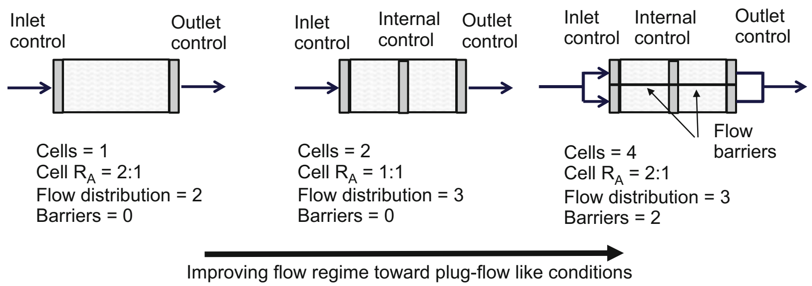


Fig. 10.14 Illustration of geometries, cell aspect ratios, and internal flow control structures to enhance plug-flow conditions (*Note*: The wetland A_W is the same for all three configurations)

10.58

- Illustration of a potential solution for a wetland area that has a high A_{XC} needed to handle Q_D (as calculated using Eq. 10.15)
  - Dividing the wetland into four parallel that provide the same total A_{XC} but have better L:W ratios (Fig. 10.15)

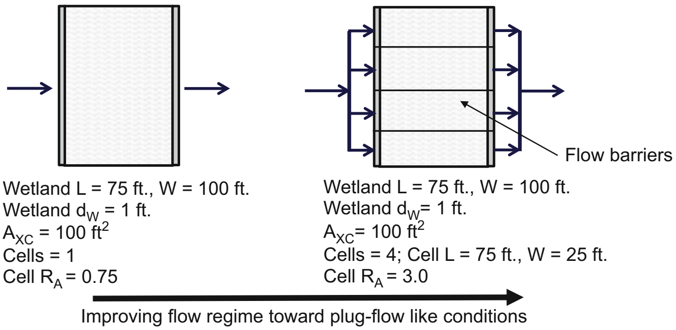


Fig. 10.15 Illustration of an approach to provide the same wetland A_W and A_{XC} but with improved flow regime characteristics

10.59

- D&I considerations—Installation at the site
  - Establishment of a constructed wetland includes the physical design, construction, and vegetation planting
  - Wetland location on a site must account for constraints such as:
    - Ensuring construction equipment access
    - Choosing sites with gentle slopes (1–3 % are easiest)
    - Avoiding damage to existing utilities
    - Floodplains vs. floodways—avoid
    - Compliance with applicable regulations and permitting processes
  - Layout and configuration of the wetland
    - Number of independent wetland flow paths (e.g., $\geq$2 for larger design flows and wetland sizes)
    - Wetland area geometry and flow controls for cell configurations

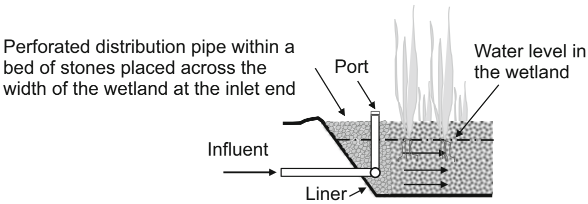
10.60

- Bed containment and bed depth
  - Containment
    - Synthetic liners
      - For small projects (e.g., <1000 ft^2), one-piece factory seamed PVC liners (0.76 mm thick) can be used
      - For larger projects, high density polyethylene (1.1–1.5 mm thick) can be used with in-field welded seams after placement
    - Natural liners
      - In some applications, compacted soil (with high clay content) might be used
  - Bed depth
    - FWS require bed depth for rooting (e.g., 1 ft) and additional water column depth for flow (e.g., 2–3 ft in the inlet and outlet zones and 4–5 ft in the open water zone)
    - VSB wetlands need depth for rooting and flow (e.g., 2 ft)

10.61

- Water flow controls
  - Inlet structures (Fig. 10.16)
    - Designed to achieve distribution of the influent across the entire A_{XC} of the wetland inlet zone
    - Uniform distribution across the inlet Is very important to achieving plug-flow like hydraulics, which are needed for high treatment efficiency
    - Inlets can be made with piping, channels, chambers, and coarse rock beds

Fig. 10.16 Cross section of an example inlet configuration



10.62

- Outlet structures and water level controls (Fig. 10.17)
  - Outlet design is very important to wetland hydraulics to:
    - Enhance plug-flow regime through the wetland
    - Avoid short-circuiting and inactive flow zones
    - Enable control of the water level in the wetland
  - Outlets can be made with piping, rock channels, chambers, and combinations thereof
  - Water level controls can be implemented in the body of the wetland or in a separate basin near the outlet end

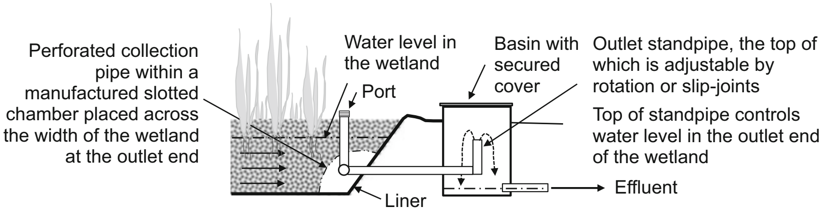


Fig. 10.17 Cross section of an example of an outlet and water level control configuration

10.63

- Photographs of a VSB inlet and outlet are shown in Fig. 10.18

Fig. 10.18 Photographs of a VSB wetland during construction (*Source*: www.goshen.edu/merrylea/collegiate/riethvillage.html)

10.64

- Vegetation establishment
  - Plant selection
    - Select plants that are locally available and are noninvasive species
    - Plants must be suited to the climate and wetland environment with consideration of aesthetic values
  - Planting
    - Cuttings (e.g., 4-in. long) or potted plants are placed about 2 in. into the porous media
    - For most common species, spacing is on the order of 20–60 plants per 100 ft^2
  - Initiating growth
    - The wetland zone is flooded with water to initiate growth

10.65

- Figure 10.19 presents a photograph of a newly planted subsurface vegetated bed wetland.

Fig. 10.19 Photograph of a newly planted VSB wetland

10.66

- D&I considerations—Wetland startup
  - Once a wetland is established and started up it can take time to achieve stable performance with respect to treatment efficiency
  - Startup can occur relatively quickly with VSBs
    - Biological processes that occur in biofilms that develop during flow through porous media that are not dependent on plants
  - Startup for FWS wetlands can take longer
    - For FWS wetlands, stable performance can take up to 1–2 growing seasons

10.67

- D&I considerations—Operation and maintenance
  - After startup, routine operation of a constructed wetland is typically minimal, and might include:
    - Flow monitoring and adjustments as needed
    - Inspection of inlets and outlets as well as berms
    - Need for monitoring of effluent quality varies widely
      - Depends on discharge plans and regulatory requirements
  - Maintenance functions in a properly designed and implemented wetland can include:
    - Harvesting of plants (required for true removal of constituent uptake)
    - Solids removal in FWS wetlands (possibly only every 10–15 years)
    - Recovering from clogging in VSB wetlands (possibly only every 20 years±)

10.68

10-5. Summary

- Constructed wetlands exploit natural wetland processes to treat wastewaters and other impaired waters
  - FWS wetlands utilize horizontal flow through a shallow pond planted with floating, submerged or emergent vegetation
    - Often used for tertiary polishing of secondary effluents
  - VSB wetlands utilize subsurface flow through a bed of porous media that is planted with emergent vegetation
    - Often used for secondary treatment of primary effluents
- Constructed wetland design and implementation
  - Constructed wetlands can produce a high quality effluent and potentially provide wildlife habitat and aesthetic benefits
  - Wetlands can be passive and not require power or chemicals
  - Constructed wetlands can be implemented in a wide range of conditions, including very cold and very hot climates
  - Wetlands do require land area and time for startup

10.69

10-6. Example Problems

- 10EP-1. Adjusting a total nitrogen removal rate constant for temperature
  - Given information
    - The wetland type is a free water surface wetland that is being designed using a P-k_A-C* modeling approach
    - A removal rate constant is needed for the wetland that will be located in a climate with an average temperature of 12 °C
  - Determine
    - Calculate the k_A for a wetland that is operated at 12 °C

10.70

- Solution
  - Based on literature data, select a k_A value for total N removal = 0.11 ft/day at 20 °C with a $\theta = 0.156$ (e.g., see Table 10.7)
  - Using Eq. 10.7, adjust the k_A at 20 °C to k_A at 12 °C

$$k_T = k_{20}\theta^{(T-20)} = k_{20}1.056^{(12-20)} \tag{10.7}$$

$$k_{12} = k_{20}(0.647)$$

$$k_{12} = 0.11(0.647) = 0.071\,\mathrm{ft/d}$$

10.71

■ 10EP-2. Sizing a VSB constructed wetland

- Given information
  - Design daily flow = 10,000 gal/day from an apartment complex
  - Treatment prior to the wetland is by a septic tank unit w/ screen (STE BOD_5 = 150 mg/L, TSS = 40 mg/L, NH_4^+-N = 40 mg-N/L)
  - Wetland temperature is relatively constant at 10 °C
  - Treatment goal: reduce the STE BOD_5 to 30 mg/L (after accounting for internally produced BOD)
  - VSB treatment zone = 20–30 mm diameter gravel ($\varepsilon = 0.40$, K_S = 32,800 ft/day, F = 0.1). VSB bed depth = 1.5 ft with a water depth = 1 ft and a VSB hydraulic gradient = 0.5 %
- Determine
  - Using ALR and P-k_A-C* methods, determine the wetland surface area (ft^2), hydraulic retention time (days), and wetland volume (ft^3)

10.72

- Solution
  - VSB wetland area based on ALR loading specifications

$$A_W = \frac{(Q_D)(C_I)(F)}{(ALR)} \tag{10.3}$$

$$A_W = \frac{(10{,}000\ \text{gal/d})(150\,\text{mg/L})\left(8.34 \times 10^{-6}\right)}{1.23\ \text{lb/day}/1000\text{ft}^2}$$

$$\text{ALR } A_W = 10{,}170\ \text{ft}^2$$

10.73

- Wetland area based on P-k_A-C* modeling
  - Calculate the areal hydraulic loading rate, q

$$q = \frac{k_A y}{P\left[\left(\frac{1}{(C-C^*)/(C_I-C^*)}\right)^{1/P} - 1\right]} \tag{10.5}$$

$$q = \frac{(0.22)(1)}{3\left[\left(\frac{1}{(30-10)/(150-10)}\right)^{1/3} - 1\right]}$$

$$q = \frac{0.22}{2.74} = 0.08\ \text{ft/day}$$

10.74

* Calculate the wetland surface area (A_w) using the calculated hydraulic loading rate (q) and the design daily flow rate (Q_D)

$$A_W = \frac{Q_D}{q} \tag{10.6}$$

$$A_W = \frac{1337 \ \text{ft}^3/\text{day}}{0.08 \ \text{ft/day}}$$

$$\text{PTIS } A_W = 16{,}710 \ \text{ft}^2$$

10.75

* Check to see if the organic loading rate for the A_W determined by P-k_A-C* modeling is acceptable

$$OLR = \frac{(Q_D)(BOD_5)(F)}{A_W'} \tag{10.8}$$

$$OLR = \frac{(Q_D)(BOD_5)(8.34 \times 10^{-6})}{A_W'}$$

$$OLR = \frac{(10{,}000 \ \text{gal})(150 \ \text{mg/L})(8.34 \times 10^{-6})}{16{,}710 \text{ft}^2}$$

$$OLR = 0.00075 \ \text{lb} \ BOD_5/\text{ft}^2$$

$$OLR = 0.75 \ \text{lb} \ BOD_5/1{,}000 \ \text{ft}^2$$

✓ This OLR is okay based on a guidance value of 1.23 lb-BOD_5/1000 ft^2 (refer to Table 10.9)

10.76

- Determine the wetland volume
  - Nominal volume (V_N) depends on A_W and selected d_W
  - Effective volume (V_{WE}) accounts for porosity and inactive flow zones

$$V_N = (A_W)(d_W) \qquad (10.9)$$

$$V_{WE} = (e_V)(\varepsilon)V_N \qquad (10.11)$$

$$\underline{V: \ \text{ALR based } A_W}$$
$$V_N = (A_w)(d_W)$$
$$V_N = (10170 \ \text{ft}^2)(1 \ \text{ft})$$
$$V_N = 10{,}170 \ \text{ft}^3$$
$$V_{WE} = (0.83)(0.4)(10{,}170 \ \text{ft}^3)$$
$$V_{WE} = 3{,}376 \ \text{ft}^3$$

$$\underline{V: \ P - k_A - C^* \ \text{based } A_W}$$
$$V_N = (A_w)(d_W)$$
$$V_N = (16710 \ \text{ft}^2)(1 \ \text{ft})$$
$$V_N = 16{,}710 \ \text{ft}^3$$
$$V_{WE} = (0.83)(0.4)(16710 \ \text{ft}^3)$$
$$V_{WE} = 5{,}540 \ \text{ft}^3$$

10.77

- Determine the effective wetland hydraulic retention time (HRT_E)
  - Nominal HRT_N depends on calculated A_W and selected d_W
  - Effective HRT_E accounts for porosity and volumetric efficiency
  - Note Q can be chosen to equal Q_D or the actual daily flow rate

$$HRT_N = \frac{V_N}{Q} \qquad (10.13)$$

$$HRT_E = \frac{V_{WE}}{Q} \qquad (10.14)$$

$$\underline{HRT_E: \ \text{ALR based } A_W}$$
$$HRT_E = \frac{V_{WE}}{Q_D}$$
$$HRT_E = \frac{3{,}376 \ \text{ft}^3}{1{,}337 \ \text{ft}^3/\text{day}} = 2.5 \ \text{days}$$

$$\underline{HRT_E: \ P - k_A - C^* \ \text{based } A_W}$$
$$HRT_E = \frac{V_{WE}}{Q_D}$$
$$HRT_E = \frac{5{,}540 \ \text{ft}^3}{1{,}337 \ \text{ft}^3/\text{day}} = 4.1 \ \text{days}$$

10.78

- A comparison of results is shown in Table 10EP.1

Table 10EP.1 Comparison of a wetland designed using different approaches

Parameter	VSB sizing method used	
	ALR	P-k_A-C*
Effluent concentration	Presumed:	Target set:
	BOD = 30 mg/L	BOD = 30 mg/L
Internally produced BOD	Not used	Assumed = 10 mg/L
Wetland water-filled depth (ft)	Given = 1	Given = 1
BOD removal rate constant	Not used	Selected 25 m/year $\theta = 1.0$, so no adj.
Apparent tanks in series used in PTIS modeling	Not used	3
Wetland area (ft^2)	10,170	16,710
BOD loading rate (lb/day/ft^2)	0.00123	0.00075
Conversion of V_N to V_A	$e_V = 0.83$; $\varepsilon = 0.4$	$e_V = 0.83$; $\varepsilon = 0.4$
Wetland volume—effective (ft^3)	3376	5540
Hydraulic retention time—effective (day)	2.5	4.1

10.79

- Some derived values based on the given values and sizing calculations are shown in Table 10EP.2

Table 10EP.2 Comparison of some derived values based on the results for a wetland designed using different approaches

Parameter	VSB sizing method used	
	ALR	P-k_A-C*
Daily flow rate (gal/day) given	10,000	
Population equivalents (assume 1 PE = 60 gal/day)	167	
Areal hydraulic loading rate (gal/day/ft^2)	1.0	0.6
Wetland surface area provided per population equivalent (ft^2/PE)	61	97

10.80

- 10EP-3. Determining the geometry for a VSB constructed wetland
  - Given information
    - Design daily flow = 10,000 gal/day (1337 ft^3/day)
    - VSB surface area = 13,140 ft^2
    - VSB treatment zone = 30 mm diameter gravel ($\varepsilon = 0.40$, clean media $K_S = 32{,}800$ ft/day, F = 0.1). VSB bed depth = 2.0 ft with a water-filled depth = 1.5 ft and VSB hydraulic gradient = 0.5 %
  - Determine
    - The minimum cross-sectional area required to handle the design flow rate and the length and width of the wetland surface area

10.81

- Solution
  - Calculation of A_{XC} using Eq. 10.15

$$A_{XC} = (d_W)(W) = \frac{Q_D}{(K_S)(F)(S)} \tag{10.15}$$

$$A_{XC} = \frac{1{,}337\,\text{ft}^3/\text{day}}{(32{,}800\,\text{ft}/\text{day})(0.1)(0.005)}$$

$$A_{XC} = 81.5\ \text{ft}^2$$

  - Calculation of the minimum width needed to handle Q_D

$$A_{XC} = (d_W)(W) = 81.5\ \text{ft}^2 \tag{10.15}$$

$$W = \frac{81.5\ \text{ft}^2}{1.5\,\text{ft}} = 54.3\ \text{ft}$$

10.82

- Calculation of the length based on the minimum width required

$$A_W = L \times W$$
$$L = \frac{A_W}{W} = \frac{13,140\,\mathrm{ft}^2}{54.3\,\mathrm{ft}} = 242 \;\; \mathrm{ft}$$

- Calculate the aspect ratio

$$R_A = \frac{L}{W} = \frac{242}{54.3} = 4.5 \tag{10.16}$$

  * The wetland aspect ratio of 4.5 is fine and within a reasonable range based on engineering practice limits
  * If the landscape area available required a different wetland geometry the length and width could be adjusted as long as the resulting R_A (with one or more cells) would be conducive to plug-flow like conditions in the wetland

10.83

Chapter 11

Treatment Using Subsurface Soil Infiltration

11-1. Scope

Subsurface soil infiltration is a form of land-based treatment where effluent from a tank-based wastewater treatment unit is infiltrated into soil below the ground surface and treatment occurs during percolation through a soil profile and assimilation into the subsurface soil and groundwater environment. Land-based systems have been used for more than 100 years, initially for simple waste disposal but later for effective treatment purposes also. A modern version of a land-based system is a soil treatment unit that is designed to achieve tertiary treatment and natural disinfection. This Chapter presents the principles and processes involved in wastewater treatment in soil and the considerations important to design and implementation of soil treatment units that rely on subsurface soil infiltration and recharge of local groundwater.

11-2. Key Concepts

- A soil treatment unit (STU) is designed to treat primary (e.g., septic tank effluent) or secondary effluent (e.g., sand filter effluent) when it is applied as an influent to the STU and infiltration and percolation occurs through an unsaturated, aerobic soil profile followed by recharge of groundwater under a site. A soil treatment unit is a 21st century version of older 20th century systems that were referred to as leachfields, drainfields, or soil absorption systems.
- Treatment processes in a STU include physical (e.g., filtration), chemical (e.g., sorption), and biological (e.g., aerobic biotransformation).

R.L. Siegrist, *Decentralized Water Reclamation Engineering*,
DOI 10.1007/978-3-319-40472-1_11

- With domestic septic tank effluent, after unsaturated flow through as little at 3 ft of an aerobic soil profile, the percolating soil pore water can have BOD_5 and TSS <5 mg/L, near complete nitrification, 20–60 % removal of total N, near complete removal of total P, and 99.99 % removal of pathogens.
- Attenuation processes can further treat the percolate in a deeper vadose zone (if present) and in groundwater under a site as reclaimed water moves away from the site and is assimilated into the environment.
- The treatment efficiency achieved in a STU and the receiving soil and groundwater environment can be comparable to a tank-based treatment operation that produces tertiary effluent with disinfection.

■ Soil is fundamental to the design and performance of a soil treatment unit for wastewater treatment and water reclamation. As defined by the National Resources Conservation Service, "Soil is a natural body comprised of solids (minerals and organic matter), liquid, and gases that occurs on the land surface, occupies space, and is characterized by one or both of the following: horizons, or layers, that are distinguishable from the initial material as a result of additions, losses, transfers, and transformations of energy and matter or the ability to support rooted plants in a natural environment."

■ Treatment of a wastewater effluent (e.g., STE) dispersed into a native soil profile (i.e., undisturbed and in place in the landscape where it was formed during soil genesis) involves effluent water movement and constituent removal reactions and the process interactions are diverse and dynamic over time. Key processes are associated with:

- Wastewater infiltration and percolation in a soil profile.
- Wastewater pollutant and pathogen transformation and removal reactions.
- Wastewater attenuation and assimilation within subsurface soil and groundwater.

■ Infiltration of wastewater into a soil profile depends generally on the same factors that govern infiltration of clean water but there are three important differences:

- Potential presence of expandable clay minerals—If a soil profile has expandable clay minerals (e.g., montmorillonite and bentonite), addition of wastewater can cause swelling, loss in permeability, and a reduced saturated hydraulic conductivity (K_S). Water chemistry interactions (e.g., Na^+ cation exchange) can also cause dispersion of clays and reduce K_S. Both effects can reduce the infiltration capacity of the soil. Sites with expandable clay minerals in the soil profile should be avoided.

- Potential damage caused during installation—If construction is done poorly, soil compaction and smearing can cause soil pores to be blocked or sealed and this can greatly reduce the infiltration capacity compared to that of undisturbed native soil. Damage to the soil infiltrative surface can be avoided by careful construction practices and this is especially important for installations in fine-grained soils (e.g., silty clays, clay loams, clays, etc.).
- Wastewater-induced changes and effects—Infiltration of wastewater (even secondary or higher quality effluents) into soil causes three types of wastewater-induced changes at and just below the soil infiltrative surface: (1) Biofilms develop within the initial depth of soil below the soil infiltrative surface due to water, nutrients, and microorganisms entering the soil pore network; (2) A biomat can form at and above the soil infiltrative surface if there are suspended solids in the wastewater applied that can be filtered out and retained on the surface; and (3) Pore-filling occurs immediately beneath the soil infiltrative surface as humic substance like materials evolve over time. The zone in which changes occur has been referred to as a 'clogging zone' or 'biozone'. Wastewater-induced changes are unavoidable but the nature and extent of the effects depends on soil properties and site conditions in combination with the wastewater composition, hydraulic loading rate, and method of delivery and distribution.

■ Infiltrability (the infiltration rate when water is made freely available at the soil infiltrative surface) declines during longer-term operation due to soil clogging that results from the development of a biomat and the pore filling that can occur. The decline can be substantial and can lead to a long-term acceptance rate (LTAR) for wastewater infiltration through a soil infiltrative surface.

- The LTAR is the pseudo steady-state infiltrability that occurs after a period of operation but before wastewater is continuously ponded on top of the soil infiltrative surface.
- The LTAR is much less than the infiltrability for clean water that exists prior to the addition of wastewater (e.g., 5–10 % or less of the saturated hydraulic conductivity (K_S)).
- For long-term operation, the design hydraulic loading rate (HLR_D) used to calculate the soil infiltrative surface area required is typically set at or near the LTAR.
- With long-term operation and under some conditions, the LTAR can eventually decline toward zero. Depending on the actual hydraulic loading rate applied during routine operation of a STU, ponding of wastewater above the soil infiltrative surface can become intermittent or continuous and the resulting hydraulic head can help drive infiltration so the STU can continue to process the daily flow.

- If the LTAR does decline to a value well below the actual hydraulic loading rate and if there is insufficient height for ponding above the infiltrative surface, hydraulic failure can result (e.g., backup of wastewater into a building or seepage of partially treated wastewater to the ground surface). If this occurs rehabilitation or replacement of the STU is required.
- Key factors that control the infiltrability decline and affect the LTAR applicable to a particular STU include soil properties and site conditions and design and operation attributes (i.e., infiltrative surface architecture, wastewater application rate and wastewater composition with respect to the concentrations of biochemical oxygen demand, total Kjeldahl nitrogen, and total suspended solids, and the method of wastewater delivery and distribution).

■ Wastewater that infiltrates into the soil below a soil infiltrative surface percolates downward by gravity forces. Percolating wastewater is gradually transformed to a reclaimed water as constituents in the wastewater are transformed and removed. The reclaimed water typically recharges local groundwater under the site where wastewater infiltration occurs. The reclaimed water can undergo further treatment by attenuation processes that can occur in the deeper subsurface and in the groundwater as it migrates away from the site.

■ Many of the key transformation and removal processes in a soil treatment unit are somewhat analogous to those in a single-pass sand filter (SPSF) but the processes in a STU can be more complex and dynamic than those in a SPSF because:

- Soil has more dynamic properties and heterogeneities that affect water movement and treatment.
- A STU 'lives' in the natural environment and can be connected to a deeper vadose zone and groundwater and surface waters that can help attenuate constituents of concern and assimilate reclaimed water.
- Conceptually, removal processes in a STU occur in several zones encompassing wastewater infiltration, soil percolation, groundwater recharge and transport, and linkage with a surface water.
- Key factors that control the purification of wastewater within the soil and subsurface include: hydraulic loading rate and wastewater composition, method of wastewater delivery and application, and infiltration depth and unsaturated soil properties. Treatment can be impacted (both positively or negatively) by the occurrence of layers within the soil profile and the presence of rock fragments.

■ Groundwater recharge by reclaimed water generated within a STU can cause a plume in the groundwater where there are elevated concentrations of one or more constituents derived from the wastewater infiltrated.

This plume can migrate for a short (10s of ft) or long (100 s of ft or more) distance away from the location of the STU. At some location the plume dissipates sufficiently that it is no longer distinguishable from local groundwater. The extent of an identifiable plume depends on aquifer thickness, flow velocity, and biogeochemical conditions. Depending on the context, the plume may or may not be a concern with respect to presenting an unacceptable level of contamination and risk.

- Design of a STU requires integrated analysis of the site conditions and soil properties, the wastewater source and options for wastewater treatment prior to wastewater application to the soil, along with the treatment goals and the method of assessment.
  - STUs need to be properly located at a site where landscape features are appropriate and where soil and site conditions are suitable for the size and type of STU to be installed.
  - Many STUs are often designed to treat primary effluent (e.g., septic tank effluent) and for this purpose the natural soil profile must have adequate permeability (e.g., $K_S > 1$–2 gal/day/ft^2), adequate depth to a limiting condition (e.g., 2–4 ft of unsaturated aerobic soil to groundwater or bedrock), and conditions suitable for aerobic biological treatment.
  - For higher quality effluents (e.g., packaged biofilter or membrane bioreactor), site suitability requirements can be relaxed (e.g., less unsaturated soil depth).

- For a particular site, design of a STU involves a series of engineering steps including choices concerning: the type of treatment prior to application to the soil; the architecture of the soil infiltrative surface, the wastewater application rate for infiltration area sizing, the geometry and landscape placement of the soil treatment unit, the depth of soil required beneath the infiltrative surface, the method of wastewater application and distribution, and the options for resting and cyclic operation. Engineering design of a STU is an interactive and iterative process since design choices made in one step may affect the choices made in another.

- The soil infiltrative surface area needed is a primary element in the design of a STU.
  - The soil infiltrative surface area is normally established as the horizontal surface area (bottom area) within one or more excavated trenches or beds. There is also vertical surface area (sidewall area) within a trench or bed, but infiltration through a vertical soil infiltrative surface is less predictable and only utilized when the bottom area is ponded with wastewater. Sidewall area is normally considered to be a reserve area that can be used intermittently as needed (e.g., during high flow periods or wet seasons of the year).

- The amount of horizontal soil infiltrative surface area needed is determined by a design hydraulic loading rate (HLR_D) that depends on factors controlling infiltrability decline. The HLR_D is normally limited to $<$5–10 % of the clean water K_S with an organic loading rate limited to 0.0002–0.001 lb-BOD_5/day per ft^2. For domestic septic tank effluents, the HLR_D typically ranges from 0.24 to 1.2 gal/day/ft^2 depending on soil profile properties. The HLR_D for higher quality effluents (e.g., packaged biofilter or membrane bioreactor) can be much higher (e.g., 10$\times$ or more) without compromising service life or treatment efficiency.

■ Wastewater delivery and distribution is very important to the function and performance of a STU and can be accomplished in several ways.
- Achieving the most uniform application to the soil infiltrative surface from startup through long-term operation can be accomplished using intermittent dosing with a pressurized delivery and distribution network. In contrast, highly non-uniform application can result from the semi-continuous trickle loading that occurs with gravity-based delivery and distribution.
- With most systems receiving primary effluents (e.g., septic tank effluent) soil clogging will develop and eventually lead to progressive utilization of the entire soil infiltrative surface area that the STU was designed to have. This can take months to years depending on the design features of the STU (e.g., configuration and geometry, wastewater delivery and distribution method, the HLR_D) and the actual conditions experienced during operation (e.g., actual HLR and wastewater composition).
- With high quality effluents (e.g., packaged biofilter, membrane bioreactor) soil clogging is retarded or may be nearly absent and intermittent dosing using pressurized delivery and distribution is essential to help achieve more uniform distribution and infiltration, which are necessary for a desired treatment efficiency.

■ All STUs need to be carefully installed to avoid damage to the soil infiltrative surface during construction. This is particularly true in fine-grained soils (e.g., silty clays, clays) that are susceptible to compaction and smearing and especially so if construction occurs when the soil has a high water content. Special attention is also required when the STU design includes soil infiltrative surface area that is provided in bed geometries with larger width and length dimensions.

■ STUs are generally designed and implemented to be relatively passive with limited requirements for power, chemicals, and labor. For STUs serving larger developments in particular, there can be electrical and mechanical components (e.g., pumps, valves, timers, controls) and a greater need for routine and reliable operation and maintenance.

- Monitoring of the operation and performance of a STU depends on the application and need to assure a certain performance is being achieved.
  - All STU should have a method to reliably measure and record daily flow (e.g., indoor water meter, dosing counter) and a means for inspection (and maintenance) of the soil infiltrative surface (e.g., observation ports).
  - What may or may not be required and/or feasible includes monitoring of the wastewater to be applied to the STU since this can be costly if done properly, and it is normally not needed, except for some commercial or institutional applications.
  - Sampling and analysis of soil and groundwater under and around a STU is difficult and costly, and should only be considered for special cases, such as larger systems (e.g., $\geq$ 25,000 gal/day) and those located in sensitive environmental areas.
- For sites with certain limitations (e.g., too little unsaturated soil due to high groundwater or shallow bedrock or too low a K_S due to low permeability soils) design modifications can be made to enable subsurface soil infiltration (e.g., STU can be installed at-grade or in mounded sand fill).
- Modeling tools have recently become available that can facilitate design and assessment of soil-based treatment operations including soil treatment units. For example, STUMOD is a spreadsheet-based analytical model that was developed to simulate the fate of nitrogen during subsurface soil infiltration. It has been extended to include transport and fate in groundwater under and away from a STU. Several hydrologic models can simulate the potential for mounding of groundwater that is important to consider for soil treatment units that are designed to handle larger design flows. WARMF is a watershed-scale model that can be used to simulate the cumulative effects of large numbers of decentralized systems on water quality in a basin.

11-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 11 are presented in the Slides section.

11-4. Terminology

Terminology introduced and used in Chap. 11 is defined below.

Aggregate—Refers to stones (typically 0.5–1.5 in. diameter) that are used to establish a storage volume within a subsurface infiltration trench or bed.

Assimilation—Refers to the ability of subsurface soil and groundwater to accept and integrate water reclaimed from wastewater treated in a soil-based treatment operation into the hydrologic cycle.

Attenuation—Refers to a set of soil and groundwater processes (e.g., biological and chemical reactions along with dilution and dispersion) that can reduce the concentrations of constituents of potential concern in water as it moves from a depth below a soil-based treatment operation (e.g., subsurface soil treatment unit or landscape drip dispersal unit) and recharges groundwater and moves away from the recharge location.

Biozone—Term that refers to the region at and around the soil infiltrative surface where wastewater-induced changes occur involving biofilms, a biomat, and pore-filling agents.

Bottom area—The horizontal soil infiltrative surface that is used for infiltration of wastewater until wastewater ponding causes infiltration to occur through vertical sidewall infiltrative surfaces. Bottom area may also be referred to as horizontal area.

Carbonaceous BOD_5 ($cBOD_5$)—Oxygen demand exerted over five days due to biological degradation of organic matter.

Cesspool—An older form of land-based waste disposal that was used for direct release of untreated wastewater into the subsurface to keep wastewater away from direct contact with humans. Cesspools have caused soil and groundwater contamination and are no longer used in most locations.

Clay—A naturally occurring granular material composed of finely divided mineral particles. Clay can be further defined as particles with a diameter of <0.002 mm. Clay is also a textural class of soil along with silt and clay.

Clogging—In the context of a soil treatment unit, clogging refers to the blocking and filling of soil pores at and near the soil infiltrative surface that is caused by a set of physicochemical and biological processes that occur during infiltration of wastewater into soil.

Failure—A term that is used to describe the inability of a product, process, operation, or system to achieve the performance expected based on the specifications or engineering design and implementation. For example, a porous media biofilter that was designed to process 5 gal/day/ft^2 but can no longer process 1 gal/day/ft^2 could be considered to have suffered a hydraulic failure. Failure normally implies that rejuvenation would be required to restore the performance to that expected during the design and implementation. Dysfunction is a measure of the degree of failure, but which does not normally require rejuvenation. For example, a porous media biofilter that was designed to process 5 gal/day/ft^2 but can only process 4 gal/day/ft^2 could be considered dysfunctional but not yet a failure.

Footprint area—Refers to the landscape area encompassed within the perimeter surrounding the area occupied by the entire treatment unit (e.g., membrane bioreactor, constructed wetland, soil treatment unit or landscape drip dispersal unit).

Gravity distribution—Refers to a method of distributing wastewater into different parts or zones of a soil treatment unit (e.g., different trenches or different portions of a bed).

Horizon—A term used to describe a layer in a soil profile that has developed through a set of soil-forming processes and is distinguishable from a layer above and below it.

Hydraulic loading rate for design (HLR_D)—The areal loading rate applied to the surface area of a treatment unit such as a porous media biofilter or soil treatment unit that is used for design of the surface area required for a design daily flow rate.

Infiltrability—The infiltration rate when water is made freely available at a soil infiltrative surface (at the ground surface or within the subsurface).

Infiltration rate (IR)—The rate at which water passes through the infiltrative surface area of a bed of porous media.

Infiltrative surface (IS)—The horizontal surface area that comprises (1) the top of a biofilter to which influent is distributed during a dose or (2) the location in a soil profile to which wastewater is distributed and becomes the influent to a soil treatment unit.

Infiltrative surface architecture (ISA)—Refers to the physical characteristics at and around the soil infiltrative surface encompassing the geometry of the infiltration unit (e.g., narrow trench or bed) and the characteristics of the space through which wastewater moves once it is released from the delivery piping and moves over and infiltrates into the pore network of the native soil (e.g., gravel filled vs. chamber outfitted). ISA can be difficult to grasp but it is analogous to the architecture of a building.

Infiltrative surface utilization (ISU)—Refers to the fraction of the infiltrative surface area determined during design that is actually used during startup and as operation continues.

Infiltration unit—Refers to an individual physical unit (e.g., a single trench, a narrow bed, a chamber within a larger bed) to which wastewater is applied within a soil treatment unit.

Kjeldahl nitrogen (TKN)—A laboratory method of measurement that determines the concentrations of reduced forms of nitrogen. Total Kjeldahl nitrogen (TKN) includes organic N and ammonia N.

Land-based treatment system—Refers to unit operations and systems that are used to treat wastewater or other impaired waters by exploiting processes that naturally occur on the land surface and in the soil profile underlying it. Groundwater is often involved as the receiving environment where reclaimed water is ultimately assimilated. Land-based treatment systems can also be referred to as soil-based treatment systems. Examples of land-based treatment operations covered in this book include subsurface soil treatment units and landscape drip dispersal units.

Layer—Refers to a thickness of soil, rock, water, or other matter that may or may not be affected by soil-forming processes.

Leachfield—A term that was used in the 20th century for a soil-based wastewater system that primarily involved a subsurface means of infiltration that was used for disposal of septic tank effluent.

Limiting condition—Refers to a characteristic of the subsurface that can interfere with proper function and performance of a soil-based treatment system. Three common types of limiting conditions include: (1) a shallow perched zone of saturation or shallow groundwater table, (2) a layer of soil materials that has low permeability and low hydraulic conductivity, and (3) a layer of bedrock.

Linear loading rate (LLR)—Refers to the rate of flow of wastewater that is applied as influent to a cross-section of a soil treatment unit that is perpendicular to the landscape contour and flows downgradient in the subsurface. The landscape linear loading rate has dimensions of gal/day per ft of soil treatment unit length along a slope.

Long-term acceptance rate (LTAR)—The pseudo steady-state rate at which wastewater is transmitted through a soil infiltrative surface after a long period of operation in the absence of continuous ponding of wastewater on top of the soil infiltrative surface.

Natural Resources Conservation Service (NRCS)—An organization of the U.S. Department of Agriculture concerned with the description, mapping, and preservation of natural soils, waters, and other resources associated with the landscape.

Orifice—A perforation (typ. 1/8-in. diameter +/−) in the wall of a lateral pipe in a pressure distribution system through which the pressurized influent is discharged onto a porous media in a treatment unit.

Organic loading rate (OLR)—The mass of organic matter (typically measured as lb-BOD_5/day/ft^2) that is applied to the surface of a treatment unit (e.g., porous media biofilter, constructed wetland, soil treatment unit).

Percolation—Refers to water movement that occurs in a downward direction from below a soil infiltrative surface through a soil profile under unsaturated flow conditions. Can also refer to unsaturated water movement through the media in a porous media biofilter.

Percolation test—Refers to a crude test procedure to measure a soil's infiltration capacity for clean water that is based on ponding clean water in a 6–12 in. diameter borehole for a period of time and measuring the rate of decline in the water level to determine the 'perc rate' in min./in. Perc rates have been used in the past (and still are in some locations) to judge site suitability and guide selection of a hydraulic loading rate for design but the test procedure is crude and the rate measured is dependent on test conditions and operator behaviors. Use of perc rates is generally not recommended for siting and design of a STU.

Plume—A term that refers to the extent of measurable concentrations of one or more constituents contained in groundwater that are derived from wastewater that recharges groundwater under a site used for land-based treatment. The nature and extent of the plume is different for reactive constituents (e.g., BOD, NH_4^+), which are retarded compared to nonreactive constituents (e.g., Cl^-), which are not.

Point of compliance—Refers to the location in space associated with a decentralized system(s) where a water quality criteria must be satisfied (e.g., NO_3–N concentration $\leq$10 mg-N/L). An example point of compliance is the effluent discharged from a confined unit operation such as a textile biofilter. Another example for a soil-based treatment operation is the groundwater quality measured in a groundwater observation well placed at the downgradient property line or in the groundwater as it reaches the edge of a local stream.

Pressure distribution—Refers to a method of distributing wastewater over the horizontal infiltrative surface within a treatment unit operation such as a porous media biofilter or a soil treatment unit. Pressure distribution is often used to help achieve more uniform application of wastewater to all portions of the infiltrative surface from startup through longer-term operation.

Sand—A naturally occurring granular material composed of finely divided rock and mineral particles. Sand can be further defined as particles with a diameter of 0.05–2.0 mm. Sand is also a textural class of soil along with silt and clay.

Sidewall area—The vertical sides of an infiltration unit (e.g., trench or bed) through which wastewater can infiltrate in a horizontal direction. Sidewall area is only used for infiltration when the trench or bed it is part of is has intermittent or continuous ponding of the bottom area soil infiltrative surface.

Silt—A naturally occurring granular material composed of finely divided mineral particles. Silt can be further defined as particles with a diameter of 0.002–0.05 mm. Silt is also a textural class of soil along with sand and clay.

Soil—Soil is a natural body comprised of solids (minerals and organic matter), liquid, and gases that occurs on the land surface, occupies space, and is characterized by one or both of the following: horizons, or layers, that are distinguishable from the initial material as a result of additions, losses, transfers, and transformations of energy and matter or the ability to support rooted plants in a natural environment.

Soil absorption system—A term that was used in the latter part of the 20th century for a soil-based wastewater system that primarily involved a subsurface means of infiltration that was used for disposal and treatment of domestic septic tank effluent.

Soil-based treatment operation—Refers to a treatment unit operation or system that involves the use of soil as a treatment medium. Soil-based treatment systems may also be referred to as land-based treatment systems.

Soil treatment area (STA)—See Soil treatment unit.

Soil treatment unit (STU)—A term that was coined in the early part of the 21st century in the United States to refer to a land-based wastewater treatment system that primarily involved a subsurface means of infiltration that was used for dispersal and treatment of wastewater including primary and secondary effluents. This definition is also used for a soil treatment area (STA).

STUMOD—Acronym for a spreadsheet-based analytical flow and transport model that was developed to simulate the treatment of wastewater during subsurface soil infiltration and percolation in a soil treatment unit.

Vadose zone—A depth interval in the subsurface characterized by unsaturated conditions where pores in a porous media are filled with some volume of air as well as water. The vadose zone is also referred to as the unsaturated zone.

Web Soil Survey—Refers to an online tool developed and maintained by the National Resources Conservation Service that enables the user to obtain descriptive and assessment information for specific parcels of land. http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm

11-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 11 are listed below.

AF	Adjustment factor
AOI	Area of interest
bgs	Below ground surface
BOD	Biochemical oxygen demand
cBOD	Carbonaceous BOD
C_E	Concentration in the effluent
C_I	Concentration in the influent
COD	Chemical oxygen demand
EF	Efficiency factor
GW	Groundwater
HLR	Hydraulic loading rate
HLR_A	Hydraulic loading rate that is actually applied
HLR_D	Hydraulic loading rate for design
i	Infiltrability
IR	Infiltration rate
IR_o	Infiltration rate at startup
IR_t	Infiltration rate at time, t
IS	Infiltrative surface
ISA	Infiltrative surface architecture
ISU	Infiltrative surface area utilization

LLR	Linear loading rate
LTAR	Long-term acceptance rate
MBR	Membrane bioreactor
nBOD	BOD caused by biological nitrification of ammonia
O&M	Operation and maintenance
PW	Soil pore water
STA	Soil treatment area
STU	Soil treatment unit
STUMOD	Soil treatment unit model
tBOD	Total BOD includes long-term carbonaceous BOD plus nitrogenous BOD
TFU	Textile filter unit
TKN	Total Kjeldahl nitrogen
TSS	Total suspended solids
W	Width
WARMF	Watershed analysis risk management framework model
WSS	Web Soil Survey

A'_{IS}	Area of horizontal soil infiltrative surface provided based on the STU layout
A_F	Landscape footprint area
A_{IS}	Area of the soil infiltrative surface
C	Concentration
C_{GW}	Concentration in groundwater
C_{PW}	Concentration in pore water
dh/dz	Hydraulic gradient
d_u	Depth of unsaturated soil
F	Factor, conversion factor
H	Height of ponding above a soil infiltrative surface
k	Reaction rate constant
k_{20}	Reaction rate at 20°C
K_C	Saturated hydraulic conductivity of a crust
K_S	Saturated hydraulic conductivity
k_T	Reaction rate at temperature, T
K_U	Unsaturated hydraulic conductivity
L	Length
L_I	Length of the trench or narrow bed or other infiltration unit
L_U	Length of the undisturbed land between adjacent zones
M_d	Mass discharge rate
n	Number or trenches or narrow beds
n_e	Effective porosity contributing to flow
q	Rate of water movement through a cross-sectional area
q_C	Rate of water movement through a cross-sectional area of a crust-topped soil

q_U	Rate of water movement through a cross-sectional area of an unsaturated soil profile
R_C	Hydraulic resistance to flow through a crust
t	Time
T	Temperature
W_I	Width of an individual trench or narrow bed
W_U	Width of the undisturbed land between each trench or bed or chamber
θ	Temperature activity coefficient
Ψ_u	Suction force due to capillary action of the soil pores

11-6. Problems

11.1. A site evaluation helps determine if soil and site conditions are suitable for subsurface soil infiltration using a soil treatment unit. Name two soil properties or site conditions that are typically assessed during a site evaluation.

11.2. Review of available resource information (e.g., Web Soil Survey) can be very helpful to site evaluation and design of a soil treatment unit. However, it is not good practice to rely only on available information for final system design and implementation—True or false?

11.3. Which of the following site conditions might represent a limiting condition for application of a soil treatment unit to serve a 4-unit apartment building (select all that apply)?

(a) The lot is located in the floodplain of a nearby river.
(b) The land surface on the lot has a northerly aspect with a slope of approximately 9 %.
(c) The soil profile at the site is characterized as having permeable sandy loam soils.
(d) The depth below ground surface to the ground water table is 3 ft.
(e) There is creviced bedrock at the site encountered at $>$6 ft below the ground surface.

11.4. Which of the following are reasons why trenches ($\leq$3 ft wide) or narrow beds ($>$3 to $<$12 ft wide) that are installed just below the ground surface ($<$3 ft) are often preferred for a soil treatment unit? Select all that apply.

(a) The soil profile close to the ground surface typically has a higher hydraulic conductivity.
(b) Narrow, shallow infiltration geometries facilitate O_2 diffusion into, and below, a STU.
(c) Compared to a wide bed (e.g., 20 ft wide), the hydraulic capacity of the soil infiltrative surface in a narrow trench is less likely to be damaged by construction.

(d) A STU established with trenches requires less footprint area compared to a one with beds.

11.5. Which of the following describe the effects that wastewater can have on soil properties at and within about 1 ft of the horizontal infiltrative surface of a soil treatment unit? Select all that apply.

(a) Biofilms form in the soil pores as wastewater containing water, nutrients and microorganisms enters the soil.

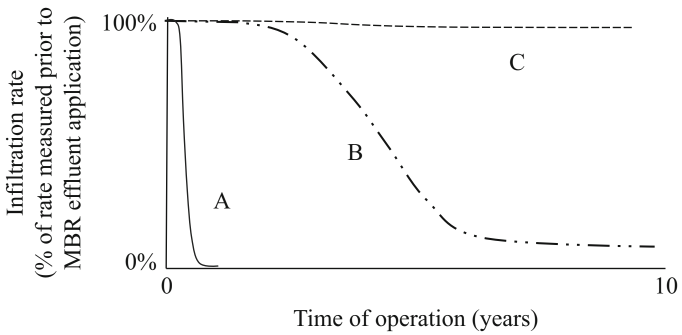
(b) TSS can be filtered out and accumulate as a biomat on top of the soil infiltrative surface.

(c) Soil pores can get filled with humic substance like materials, which develop over time.

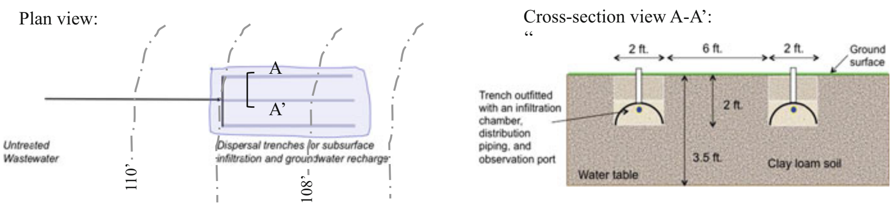
11.6. Which three of the following wastewater composition characteristics are known to be primarily responsible for the loss in infiltration capacity during operation of a soil treatment unit: BOD_5, pH, suspended solids, NO_3–N, Kjeldahl N?

11.7. In addition to the wastewater composition characteristics chosen in Problem 11.6 what other design parameter has a major impact on the loss in infiltration capacity during operation of a soil treatment unit?

11.8. In a soil treatment unit the infiltration rate (IR) at the soil infiltrative surface declines with time during routine operation. Which one of the three curves shown below best represents the shape of the decline in infiltration capacity for a soil treatment unit used to treat membrane bioreactor effluent applied to sandy soil at a HLR_D of 5 gal/day/ft^2?



11.9. For the soil treatment unit shown below, name two design flaws.



11.10. Several design factors affect purification of wastewater in a soil treatment unit. These factors are related to the types and rates of reactions that occur plus the hydraulic retention time in the soil profile. Name two of the factors that are very important to effective purification in a soil treatment unit.

11.11. A soil treatment unit was installed to handle the septic tank effluent generated at a restaurant near Denver. The STU design included a set of 3-ft wide by 50-ft long chamber equipped trenches. Within the first few months of operation, the STU hydraulically failed with partially treated wastewater seeping to the ground surface above the STU. Assuming the soil profile was sandy loam soil and there were no limiting conditions present, which two of the following are the most plausible explanations for the hydraulic failure?

(a) During the first month of operation in April, it was overcast and colder than normal.
(b) The HLR_D used in system design was 1.2 gal/day per ft^2.
(c) The wastewater temperature was about 15 °C.
(d) The actual daily flow (gal/day) was 50 % higher than the flow used for system design.

11.12. A soil treatment unit is being designed to handle the daily flow from a group of 4 houses ($Q_D = 1800$ gal/day). The site conditions and soil profile have no limiting conditions and the HLR_D for the soil type and effluent quality $= 0.8$ gal/day/ft^2. The STU design will include pressurized dosing into 2-ft wide trenches ($AF = 0.8$). What is the required infiltrative surface area (in ft^2)?

11.13. What is the greatest design daily flow (gal/day) that can be handled by a soil treatment unit designed for the site conditions listed below given an applicable design hydraulic loading rate?

Given information and assumed values: Landscape slope on the property = 0 %. Available footprint area with suitable conditions is 48 ft wide by 98 ft long. Soil profile in the available area is sandy loam soil and the depth to high groundwater = 7 ft. Treatment prior to the STU will be done using a textile media biofilter. The textile media biofilter effluent will be pressure dosed into trenches outfitted with infiltration chambers ($EF = 1$). Each trench will be 2-ft wide and 40-ft to 80-ft in length and the undisturbed soil separation between adjacent trenches = 6 ft.

11.14. A soil treatment unit is being used to treat septic tank effluent from a fast-food restaurant. The STE is expected to have a total nitrogen concentration of 150 mg-N/L. If the concentration of total nitrogen in the soil water that is percolating into the groundwater located at 4 ft depth below the soil infiltrative surface has to be reduced to $\leq$10 mg-N/L, what percent removal of total N is required? Is achievement of this N removal efficiency likely in a typical STU?

11.15. What design daily flow volume (gal/day) can be handled by a soil treatment unit in the available footprint area under the conditions outlined in the given information?

Given information and assumed values: Soil profile at the site is sandy loam soil with a $HLR_D = 0.4$ gal/day/ft^2. The available footprint area is about 80 ft by 122 ft. Pressure dosing of septic tank effluent into a set of trenches, each of which is 80-ft long and 2-ft wide and outfitted with infiltration chambers ($EF = 1$). The undisturbed soil separation between adjacent trenches = 6 ft.

11.16. For the Mines Park housing development you are tasked to do a preliminary design of a soil treatment unit to handle the design daily flow. Based on the given information given below, answer the following design questions. (1) Based on the site investigation data, at what depth (in inches below ground surface) would you install the horizontal infiltrative surface? (2) What design hydraulic loading rate (HLR_D in gal/day/ft^2) would you select for sizing the infiltrative surface area required for a STU to treat the PBF effluent (based on the design approach where wastewater application rates are based on HLR_D)? (3) What total infiltrative surface area (bottom area, ft^2) would be required to treat the design flow? (4) The STU will be equipped with a pressurized distribution network and intermittent dosing will be used. How many doses per day would you recommend be used? (5) Assuming PBF effluent will be delivered to narrow beds, which are 5-ft wide and outfitted with open bottom chambers, how many narrow beds of what specific length are required? Sketch a layout for the narrow bed system. (6) What is the total landscape footprint area required for installation of this STU (ft^2) (assume the separation distance between adjacent narrow beds is 6 ft).

Given information and assumed values: The average daily flow, $Q_A = 28{,}425$ gal/day. The design flow, Q_D, is based on the recurring maximum daily flow ($PF = 2.0$). A STEP collection system will convey septic tank effluent to the treatment site. The STE quality expected: $pH = 6$, $COD = 220$ mg/L, $BOD_5 = 160$ mg/L, $TSS = 80$ mg/L, $TKN = 60$ mg/L. At the location of the STU, a textile media packaged biofilter (PBF) will be used to produce a high quality, aerobic effluent. The PBF quality expected: $cBOD_5 = 5$ mg/L, $TSS = 5$ mg/L, $TKN = 5$ mg-N/L. The site investigation revealed an area that was ~1000 ft long (parallel to landscape topographic contours) and 400 ft wide where the following conditions were present:

Parameter	Result
Soil series (Map unit 7 and 8)	Ascalon sandy loam, 5–9 or 9–15 % slopes
Landscape slope and elevation	5–15 % at 5200–6500 ft elev.
Typical soil profile (depth interval below ground surface)	0–7 in.: sandy loam 7–18 in.: sandy loam, sandy clay loam 18–23 in.: sandy loam, loam, sandy clay loam 23–60 in.: sandy loam, loamy sand, fine sandy loam
Frequency of flooding/ponding	None/none
Depth to restrictive feature	More than 80 in.
Drainage class	Well drained
Capacity of the most limiting layer to transmit water	Moderately high to high (0.60–2.0 in./h)
Depth to water table	More than 80 in.

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Slides of Chapter 11

Decentralized Water Reclamation

Chapter 11: Treatment using Subsurface Soil Infiltration

Contents

11.1

11-1. Introduction

- Treatment using land-based systems
  - Treatment of wastewaters (and other impaired waters) can be accomplished in land-based systems by exploiting specific characteristics of landscape and soil ecosystems
  - In these systems wastewaters (primary or secondary effluents) are treated and assimilated into the local hydrologic system
    - Wastewater effluent is released to the land, either above ground or below ground, and migrates in one or more directions
      - Downward and laterally through the soil profile to groundwater
      - Upward via evapotranspiration to the atmosphere, and/or
      - Laterally across a vegetated landscape surface
    - Treatment occurs during long hydraulic retention times by a dynamic set of processes that include physical, chemical, and biological reactions

11.2

- Land-based treatment systems can be classified by the method of wastewater dispersal and the receiving environment (Fig. 11.1)

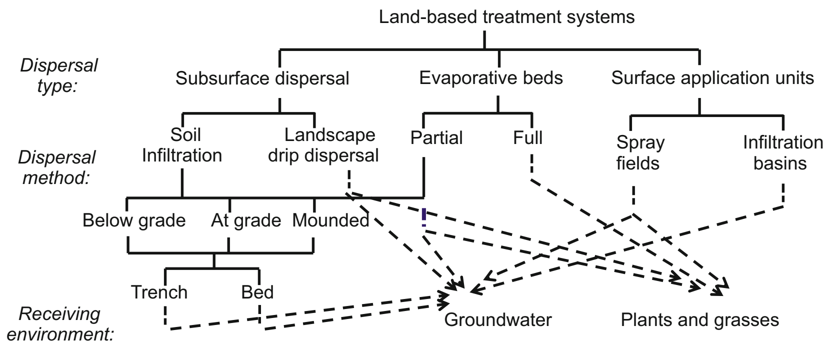


Fig. 11.1 Classification of land-based treatment systems including subsurface soil infiltration and landscape drip dispersal, which are most commonly used in decentralized applications

11.3

- Land-based systems have been used in decentralized applications for more than 100 year
  - In the United States during much of the 20th century, land-based systems were used in various forms for waste disposal
    - e.g., in a pit privy, cesspool, seepage pit, or leachfield
  - Advances in understanding during the 1970s led to improvements directed at effective treatment plus disposal
    - e.g., in a soil absorption system
  - Millions of these land-based systems exist in the United States
    - Some are poorly designed, improperly located, and not performing well with respect to treatment efficiency
    - Some are more properly designed and located and are providing an acceptable treatment efficiency

11.4

- Evolution of modern soil-based treatment systems
  - Advancements were made in soil-based systems to enable long-term wastewater treatment in decentralized applications
    - Research projects and field experiences occurred through the late 1990s and early 2000s
    - Advancements in system science, engineering and modeling helped enable more rational design and implementation of a soil treatment unit (a.k.a. soil treatment area) that would reliably achieve tertiary treatment with natural disinfection during subsurface infiltration, percolation, and groundwater recharge
  - Chapter 11 is focused on soil treatment units (STU) and the word “modern” is used to distinguish a contemporary STU from older land-based systems that were designed primarily for disposal

Note: Chap. 12 is focused on landscape drip dispersal.

11.5

- Basic features of a modern STU
  - Effluent from a confined unit operation (e.g., septic tank or aerobic unit) is delivered as the influent to a set of infiltration units (e.g., trenches or beds) that are constructed below ground surface
  - The influent to the STU infiltrates into the native soil profile and percolates downward and transitions to a reclaimed water that can be further attenuated and assimilated in a local groundwater
  - Treatment occurs during long hydraulic retention times in the subsurface (e.g., months to years) due to physico-chemical and attached growth biological processes as well as by attenuation through dilution and dispersion
  - An example of a STU for a particular application and set of site conditions is illustrated in Figs. 11.2 and 11.3.

11.6

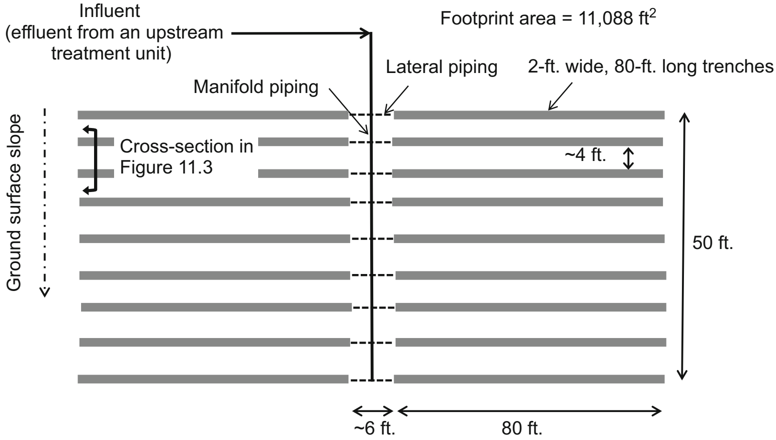


Fig. 11.2 Plan view of a modern STU (network of 18 trenches for 2000 gal/day)

11.7

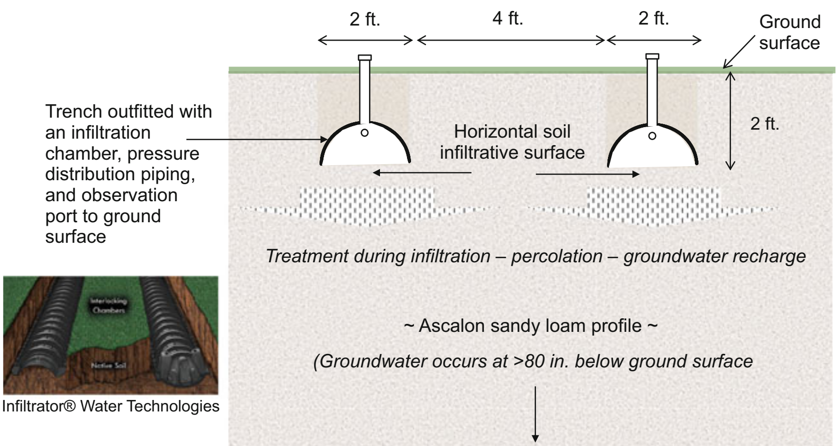


Fig. 11.3 Cross-section view of a set of subsurface infiltration trenches used in a STU for a particular application and set of site conditions

11.8

- Placement of a soil infiltrative surface
  - The horizontal soil infiltrative surface can be placed so there is a depth interval of unsaturated aerobic soil before a limiting condition is reached (Fig. 11.4)
    - * Limiting conditions include a low permeability layer, fractured bedrock, perched zone of saturation or groundwater table

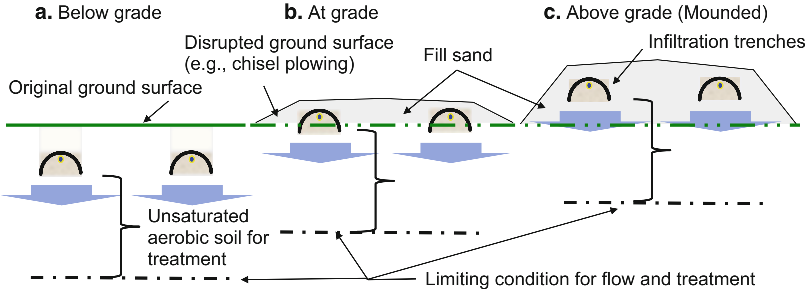


Fig. 11.4 Optional placement of the soil infiltrative surface to provide an adequate depth of unsaturated aerobic soil before a limiting condition occurs

11.9

■ Where are STUs used?

- Where site conditions and soil properties are suitable and land area is available
  - For treatment of primary (e.g., septic tanks) or secondary treated wastewater (e.g., aerobic treatment or porous media biofilter effluent) and hydrologic assimilation of reclaimed water
  - For discharge and hydrologic assimilation of tertiary treated wastewater (e.g., membrane bioreactor effluent)
- A STU can be used in decentralized systems to treat wastewaters, graywaters, and other impaired waters generated in buildings and developments including:
  - Isolated homes and businesses in rural areas
  - Clusters of buildings including mixed-use development centers
  - Suburban developments and small towns

11.10

11-2. Treatment Performance

- STUs are commonly designed to receive primary or secondary treated wastewaters and achieve tertiary treatment with natural disinfection
  - Treatment occurs within two or more zones in the subsurface
    - Treatment within the STU—boundaries are variably defined
    - Attenuation and assimilation within the subsurface under and away from the STU
  - Treatment performance depends on site conditions and soil properties as well as STU design and implementation
- STUs can also be designed to receive tertiary quality effluents (e.g., from a membrane bioreactor)
  - Primary goal is to enable discharge via high rate infiltration into the subsurface and polishing of any residual constituents of concern

11.11

- Treatment efficiency
  - Assessing treatment efficiency for a STU is more complicated than for a confined unit like a packaged biofilter since in a STU there is not an outlet *per se* for a treated effluent
    - Concentrations in percolate (soil pore water, C_{PW}) at a depth below the infiltrative surface (e.g., 3 ft) can be compared to the effluent applied (C_I) to determine removal efficiency (R_E) using Eq. 11.1 (Fig. 11.5)
    - In most situations it is very hard to assess R_E in this manner due to the difficulties and costs of monitoring soil pore water
  - Oftentimes what is most important is that the treatment efficiency is sufficient to reduce the concentrations of constituents of concern to a certain level at a certain location
    - e.g., $C_{GW2} = 10$-mgN/L in groundwater at the property boundary (Fig. 11.5)

11.12

$$R_E = \left[\frac{C_I - C_{PW}}{C_I}\right] \times 100\% \quad (11.1)$$

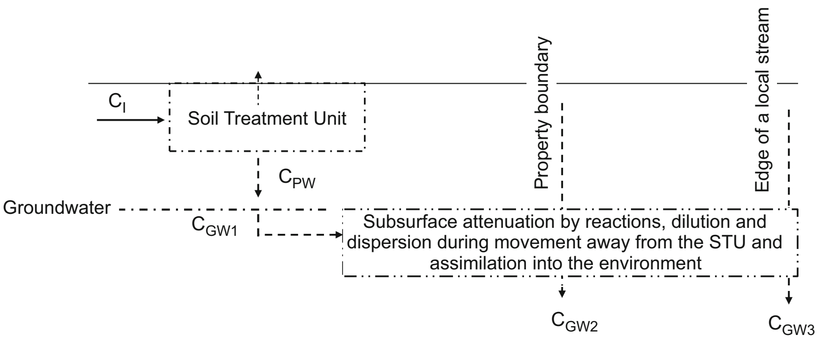


Fig. 11.5 Definition schematic for components and compliance points that can be used to define the treatment efficiency in a STU

11.13

- Treatment efficiencies achievable in a STU are shown in Table 11.1

Table 11.1 Treatment efficiency achieved within a well designed and operated STU[a]

Constituent group	Soil solution mg/L (C_{PW}) or % removal by 3-ft depth	Potential processes involved in treatment
BOD_5	<5 mg/L	Dissolved, colloidal and particulate organics are converted to cell mass and CO_2 which can be separated from the effluent
TSS	<5 mg/L	TSS in the form of colloidal and particulate solids are filtered out and separated from the effluent
Nitrogen	<5 mgN/L NH_4^+	Biological nitrification of NH_4^+ compounds to NO_3^- with 20–60 % removal of total N by biodenitrification in the soil profile[b]
	20–60 % total N	
Phosphorus	90–99 % total P	Sorption of P to mineral surfaces and precipitation
Pathogens	99.99 %	Filtration, die-off and inactivation
Trace organics	Up to >99 %	Near zero removal for some compounds but up to 90 % or more removal of compounds that are susceptible to sorption and aerobic biodegradation

[a]Based on application of 0.24–1.2 gal/day/ft^2 of domestic STE to an unsaturated, aerobic soil profile with conditions conducive to treatment.
[b]Total N removal depends on soil profile attributes combined with system design and operation.

11.14

- Treatment efficiency for nitrogen can be complicated
  - Total N removal involves nitrification and denitrification, which depend on soil profile attributes combined with the wastewater loading rate and composition (Tables 11.2 and 11.3)
  - Results of model simulations and studies illustrate varied interactions affecting the removal of NH_4-N and total N, e.g.:
    - Complete conversion of NH_4^+ by 2 ft depth is common, except at higher HLRs applied to finer-grained soils
    - By 2 ft depth total N removal is often 30–50 % or more except at higher HLRs (e.g., 1.2 gal/day/ft^2) applied to coarse grained soils (Table 11.2)
    - By 3 ft depth or more, total N removal can be 70–100 % at low HLRs (e.g., 0.5 gal/day/ft^2 or less) (Table 11.3)
  - If the soil infiltrative surface is continuously ponded with influent, the soil below it may be anoxic and nitrification will be hindered
    - Application of nitrified effluent under this condition could yield high total N removals by denitrification processes

11.15

Table 11.2 STUMOD model simulations of nitrogen fate by two-foot depth below the soil infiltrative surface for septic tank effluent infiltration at two rates in different soils[a]

	NH_4—N removal (% of NH_4—N in STE infiltrated)		Total nitrogen removal (% of total N in effluent infiltrated)	
USDA soil texture	0.5 gal/day/ft^2 (%)	1.2 gal/day/ft^2 (%)	0.5 gal/day/ft^2	1.2 gal/day/ft^2
Sand	100	100	25	10
Loamy sand	100	100	32	13
Sandy loam	100	100	37	15
Sandy clay loam	100	99	36	15
Clay	100	38	45	14
Clay loam	100	26	44	12
Loam	100	49	40	13
Sandy clay	40	13	31	7

[a]*Source*: presentation by Mengistu Geza to an expert panel for Chesapeake Bay concerning Soil Attenuation of Nutrients during Onsite Wastewater Treatment, September 17, 2014.

11.16

Table 11.3 STUMOD model simulations of total nitrogen removal during subsurface soil infiltration compared to measured data from laboratory or field studies (Geza et al. 2014)

USDA soil texture	HLR (gal /day/ft^2)	Influent C_I (mg/L)	Depth below infiltrative surface (ft)	Measured lab or field data (% Removal)	Model simulations	
					STUMOD (%Removal)	Hydrus 2D (%Removal)
Sand[a]	2	57	3 (2)	5 (6)	5 (4)	4 (3)
	1.2	57	3 (2)	11 (3)	8 (4)	5 (4)
Sandy loam[b]	0.5	61	2	36	31	37
Sandy loam[c]	1.0	82	2	43	38	35
Sandy loam[d]	0.5	14[f]	2 (4)	88 (99)	87 (100)	83 (100)
	2	14[f]	2	70	69	65
Loamy sand[e]	0.3	44	5.6	97	98	98
Sandy clay loam[e]	0.7	48	5.6	98	100	100
Clay[e]	0.1	44	5.6	97	100	100
Clay[e]	0.25	44	5.6	98	99	100

As reported in Geza et al. (2013): [a]Lab data of VanCuyk et al. (2001). [b]Field data of Andreoli et al. (1979). [c]Field data of Tackett et al. (2004). [d]Field data of Conn et al. (2010). [e]Field data of Cogger et al. (1988). [f]nitrified effluent from a packaged biofilter.

11.17

- STU effluent (soil pore water) composition
  - Factors affecting treatment efficiency and water quality
    - Site conditions and soil profile properties and their suitability for wastewater infiltration and migration in the subsurface
    - A design hydraulic loading rate that is appropriate for the influent wastewater composition and the site conditions and soil properties
    - A method of wastewater influent delivery and distribution that results in utilization of the soil infiltrative surface per the design
    - During operation, unsaturated aerobic conditions are present in the soil profile for a minimum depth below the soil infiltrative surface before a limiting condition is encountered
    - Groundwater conditions under the site help provide attenuation and assimilation of reclaimed water

11.18

11-3. Principles and Processes

- Natural soil in the environment
  - Soil has been defined by the National Resources Conservation Service (NRCS) as follows:

    "Soil is a natural body comprised of solids (minerals and organic matter), liquid, and gases that occurs on the land surface, occupies space, and is characterized by one or both of the following: horizons, or layers, that are distinguishable from the initial material as a result of additions, losses, transfers, and transformations of energy and matter or the ability to support rooted plants in a natural environment."—NRCS[a]
  - A basic understanding of soil science is needed
    - To understand the principles and processes important to wastewater treatment in a native soil profile
    - Several key concepts and considerations are highlighted in Fig. 11.6

[a] *Source:* http://www.nrcs.usda.gov/wps/portal/nrcs/detail/soils/edu/?cid=nrcs142p2_054280

11.19

- Soil profile with horizons or layers that evolve from soil forming processes and have properties such as:
  - Soil texture (wt.% sand (2 – 0.05 mm), silt (0.05-0.002 mm), clay (<0.002 mm)), soil structure (e.g., blocky, prismatic, platy), color, temperature
  - Soil water regime (e.g., dry, moist, wet), saturated hydraulic conductivity (K_S), drainage (e.g., well-drained vs. poorly drained), aeration status (e.g., oxic vs. anoxic)
- Occurrence of rock fragments (vol.% of particles > 2mm) or cemented layers (e.g., fragipan)
- Occurrence of a low permeability layer, high groundwater, or bedrock
- Depth below ground surface and thickness of a horizon or layer or other feature

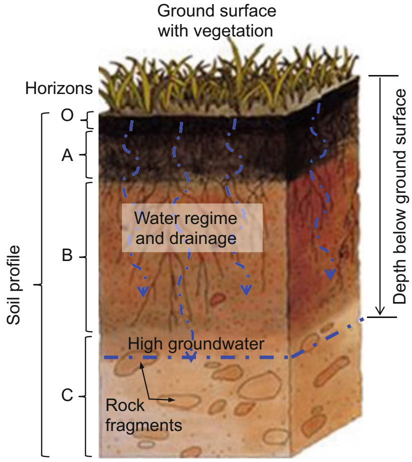


Fig. 11.6 Basic illustration of several soil science concepts and considerations related to wastewater treatment and water reclamation in a native soil profile

11.20

- Processes affecting wastewater treatment in soil[a]
  - Major processes affecting wastewater applied to subsurface soil
    - Wastewater infiltration and percolation in a soil profile
      - Soil clogging genesis and infiltrability loss
    - Wastewater pollutant and pathogen removal reactions
      - Kinetic reactions (e.g., biodegradation)
      - Capacity-based reactions (e.g., filtration, sorption)
      - Plant-based reactions[b] (e.g., nutrient uptake)
    - Wastewater attenuation and assimilation in the subsurface
      - Evapotranspiration[b]—transport up and out of the soil
      - Groundwater recharge—movement into groundwater
      - Groundwater—movement away from the STU

These processes can interact in a diverse and dynamic manner over time

[a]In the context of subsurface soil infiltration, wastewater applied is almost always an effluent from a confined unit operation like a septic tank, aerobic treatment unit, porous media biofilter, etc.

[b]ET contributions to a water balance in a STU (e.g., $<15\,\%$) and plant-based reaction contributions to treatment are normally very low based on STU design features (e.g., depth and absence of rooting). ET and plant-based reactions are important during treatment using landscape drip dispersal (refer to Chap. 12).

11.21

- Wastewater infiltration and percolation
  - Infiltration of clean water into a soil profile
    - Definition

      "Infiltration is the term applied to the process of water entry into the soil, generally by downward flow through all or part of the soil surface."—Hillel 1998
    - Terminology
      - Infiltration rate
        - IR or $q =$ the volume flux of water per unit surface area per time (e.g., gal/day/ft^2, cm^3/cm^2/day or cm/day)
      - Infiltrability
        - $i =$ the infiltration rate when water is made freely available at a soil infiltrative surface (at the ground surface or within the subsurface)

11.22

- Infiltration of clean water into soil
  - Infiltration is affected by the rate of water addition and the hydraulic properties of the native soil profile
  - If the water delivery rate $<$ the soil infiltrability
    * IR is supply-controlled (a.k.a. flux controlled)
  - If the water delivery rate $>$ the soil infiltrability
    * IR is soil-controlled
      - Surface controlled IR
        e.g., by a surface crust
      - Profile controlled IR
        e.g., by layers with low permeability
  - Soil-controlled IR situations are illustrated in Fig. 11.7

11.23

- Infiltration of clean water into soil where there is a crust
  - For a crust-topped soil, and steady infiltration, the IR can be soil-surface controlled but affected by a subcrust 'soil suction'
  - A crust has a resistance to flow (R_C) which determines the hydraulic conductivity of the crust (K_C) (Fig. 11.8)
  - Crust resistance can yield unsaturated water content in the soil under the crust
    * Unsaturated soils have an unsaturated hydraulic conductivity (K_U) which depends on the soil properties and the water content
    * At a given water content, there is a 'suction' (Ψ_u) due to capillary action of the soil pores
  - With water ponding on top of the crust (H), flow through the crust (q_C) is equal to flow in the unsaturated soil (q_U) which reaches an equilibrium based on R_C and Ψ_U (Eqs. 11.2 and 11.3)

11.24

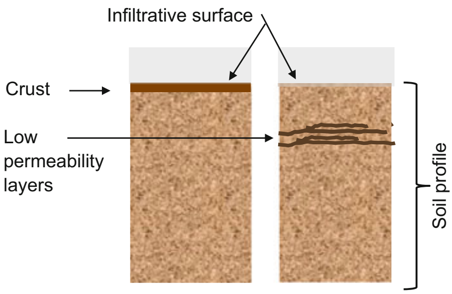


Fig. 11.7 Illustration of two types of controls on infiltration rates into a soil profile

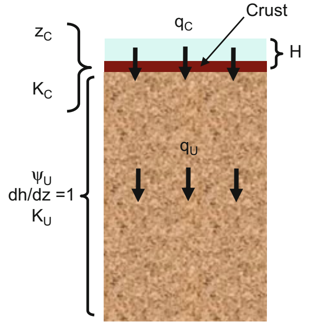


Fig. 11.8 Definition schematic for infiltration into a crust-topped soil profile

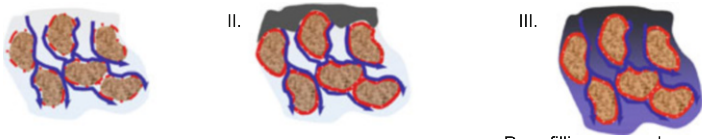
$$q_C = K_C \left(\frac{dh}{dz}\right)_C \approx q_U = K_U \left(\frac{dh}{dz}\right)_U \qquad (11.2, 11.3)$$

11.25

- Infiltration of wastewater into a soil profile depends generally on the same factors that govern infiltration of clean water, but there are important differences in three areas
  - Potential presence of expandable clay minerals
    - If a soil profile has expandable clay minerals (e.g., montmorillonite and bentonite) addition of water can cause swelling, loss in permeability, and a reduced K_S
    - Water chemistry interactions (e.g., Na^+ cation exchange) can also cause dispersion of clays and reduce K_S
    - Both effects can reduce the infiltration capacity of the soil
    - Sites with expandable clay minerals can be avoided
  - Potential damage caused during construction
    - If construction is done poorly, soil compaction and smearing can cause soil pores to be blocked or sealed
    - This can greatly reduce the infiltration capacity compared to that of undisturbed native soil
    - Careful construction practices can mitigate this damage

11.26

- Wastewater can induce changes as illustrated in Fig. 11.9



I. Biofilms develop within the soil pores

- Infiltrative surface to 6 to 12 in. depth below it
- Biofilms form as water, nutrients & microbes enter the soil pore network
- Biofilm biomass may die off and be degraded

II. A biomat can develop on the infiltrative surface

- May be near zero or up to several in. above the infiltrative surface
- Suspended solids can be filtered out and form a biomat on top of the soil infiltrative surface
- Some of the filtered solids may be biodegradable and slowly decay

III. Pore-filling occurs beneath the infiltrative surface

- Infiltrative surface to 1 to 2 in. depth below it
- Humic substance like materials can evolve over time
- Yield a 'glue' that retains water and matter in soil pores at and just below the effluent infiltration location

Development of a biozone (a.k.a. clogging zone) during operation

Fig. 11.9 Illustration of effluent-induced effects on properties important to infiltration that result in formation of a biozone (a.k.a. clogging zone). *Note*: the nature and extent of the changes and effects illustrated depends on soil properties and site conditions as well as effluent composition and loading rate

11.27

- Infiltrability declines due to wastewater-induced effects
  - Concept of three phases and infiltrability declining over time to a low infiltration rate is illustrated in Fig. 11.10
    - Infiltrability declines due to soil clogging are caused primarily by a biomat (II) and pore-filling agents (III)
  - Infiltrability declines to a pseudo steady-state infiltrability that occurs after a period of operation when wastewater ponding just begins intermittently on the soil infiltrative surface
    - This is defined as the long-term acceptance rate (LTAR)
  - With long-term operation, the infiltrability can decline to a LTAR that is less than the actual hydraulic loading rate (HLR_A) being applied to the soil infiltrative surface
    - Ponding of wastewater above the soil infiltrative surface can help drive infiltration and enable processing of the daily flow
    - Continuous ponding on the soil infiltrative surface is not inherently a bad condition as long as the soil beneath it continues to be unsaturated and aerobic

11.28

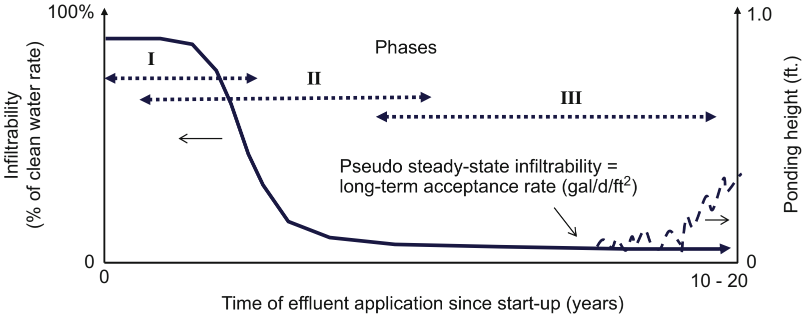


Fig. 11.10 Illustration of how soil infiltrability declines during three phases due to soil clogging caused by application of domestic septic tank effluent to a soil infiltrative surface (*Note*: for other higher quality effluents the nature and extent of infiltrability decline can be less pronounced and take longer than shown in this figure.)

11.29

- Key factors that affect infiltrability decline and the LTAR
  - Soil properties and site conditions—*these are generally given*
    - Soil texture and structure at/near the soil infiltrative surface
    - Soil profile lithology and hydrogeology
    - Site climatic and hydrologic conditions
  - Design and operation—*these are generally chosen*
    - Infiltrative surface architecture
      - Infiltrative surface geometry and depth
      - Infiltrative surface features
    - Effluent application rate and method
      - Hydraulic loading rate for design (HLR_D) and hydraulic loading rate actually experienced during operation
      - Frequency, uniformity, and continuity of wastewater application
    - Effluent quality
      - Concentrations of BOD, Kjeldahl N, and TSS

11.30

- Effects of STU attributes on the LTAR are summarized in Table 11.4

Table 11.4 Relative effects of STU attributes on the LTAR for a given system[a]

STU attribute	Relative effect(s)	Reference(s)
Initial clean water K_S of the natural soil	Minor ~ for permeable well-drained soils with K_S of about 1–1000 gal/day/ft^2, the long term acceptance rate for domestic septic tank effluent will approach 0.5 gal/day/ft^2	Jenssen (1986), Jenssen and Siegrist (1990) and Beal et al. (2005)
Subsurface soil conditions during operation	Moderate ~ higher temperatures, lower soil water contents, and higher aeration levels tend to enable relatively higher LTARs	Siegrist et al. (2001)
Infiltrative surface architecture	Moderate ~ horizontal infiltrative surfaces that are aggregate-free in low-height, narrow trenches characterized by sidewall-to-bottom area ratios of 0.5–1.0 and placed shallow in the subsurface	VanCuyk et al. (2001) and Siegrist et al. (2004, 2005)
Wastewater effluent quality	Major ~ At a given hydraulic loading rate, wastewater quality exerts a major effect based on the mass loadings of total BOD and TSS, which are key determinants of soil clogging and infiltration rate decline	Siegrist and Boyle (1987), Tyler and Converse (1989), Siegrist et al. (2001) and VanCuyk et al. (2005)

(continued)

11.31

Table 11.4 (continued)

STU attribute	Relative effect(s)	Reference(s)
Actual HLR	Major ~ For a given effluent quality, the actual HLR exerts a major effect by determining the mass loadings of total BOD and TSS which are key determinants in soil clogging and infiltration rate loss (*Note*: for clean water, the effects of HLR could be negligible if the HLR ranges below about 10 % of the soil K_S)	Siegrist (1987), Siegrist and Boyle (1987), Jenssen and Siegrist (1990), Siegrist et al. (2001, 2005) and VanCuyk et al. (2005)
Wastewater effluent application method	Minor to Major ~ for systems in continuous daily use, soil infiltrability effects are uncertain and variable depending on the features considered	Siegrist et al. (2001, 2002)
Continuity of use	Major ~ infrequent or intermittent use with long periods of resting (e.g., ≥1 year) can sustain higher soil infiltrability and LTARs	Siegrist et al. (2001)

[a]The descriptors used were developed by Siegrist (2006 et al.) and have the following meanings: “minor” indicates a relative effect of ~ +/−20 % or less, “moderate” indicates an effect on the order of +/−50 %, and “major” indicates an effect on the order of +/−100 % or more.
Source: after Siegrist 2006.

11.32

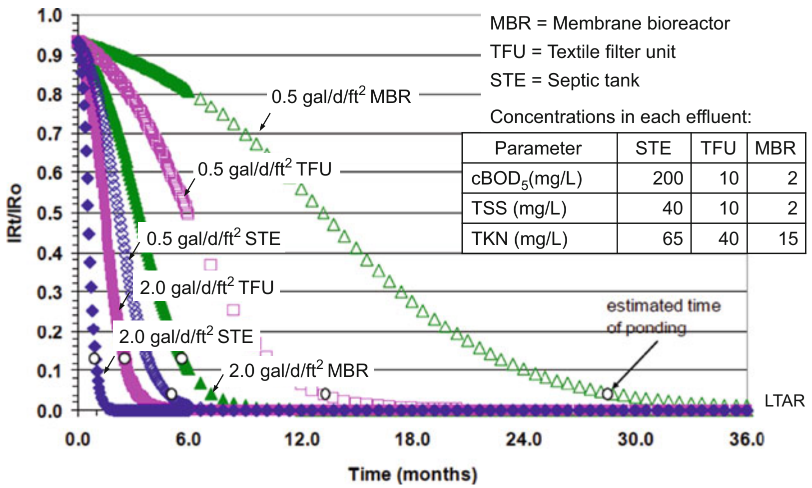
- Effects on infiltrability—Wastewater composition
  - Infiltrability decline is most strongly impacted by wastewater HLR and composition
  - Equation 11.4 describes IR decline and Fig. 11.11 presents simulations showing the effects of HLR and composition

$$\frac{IR_t}{IR_o} = \frac{exp[2.63 - 5.70(tBOD) + 41.08(TSS) - 0.048(tBOD \times TSS)]}{1 + exp[2.63 - 5.70(tBOD) + 41.08(TSS) - 0.048(tBOD \times TSS)]} \quad (11.4)$$

Where:
IR_t = infiltration rate after a period of operation (cm/day)
IR_o = infiltration rate at startup (cm/day)
tBOD = cumulative mass loading of tBOD applied to the infiltrative surface after a period of operation (kg/m^2)
= ultimate cBOD plus nBOD
TSS = cumulative mass loading of TSS applied to the infiltrative surface after a period of operation (kg/m^2)

Source: Siegrist and Boyle 1987.

11.33



Parameter	STE	TFU	MBR
$cBOD_5$(mg/L)	200	10	2
TSS (mg/L)	40	10	2
TKN (mg/L)	65	40	15

Fig. 11.11 Simulated infiltration rate decline as affected by effluent quality and loading rate in a sandy loam soil in Colorado (VanCuyk et al. 2005) (*Note*: simulations were based on Eq. 11.4)

11.34

- Effects on infiltrability—Infiltrative surface architecture
  - Infiltrative surface architecture (ISA) can be a difficult concept to grasp but think of it as analogous to the architecture of a building
  - ISA applied to a STU includes physical attributes, primarily:
    - Geometry of the infiltration trench or bed
      - Infiltration through horizontal vs. vertical surfaces (e.g., bottom and sidewall in narrow trenches vs. bottom area in wider beds)
      - Infiltrative surface depth below ground surface (e.g., shallow vs. deep trenches)
    - Features of how the infiltrative surface is established
      - Physical characteristics of the space through which wastewater influent moves once it is released from the delivery piping network and infiltrates into the native soil (e.g., gravel filled vs. chamber outfitted trenches)

11.35

- ISA choices can affect infiltrability and the LTAR within a STU
  - Narrow, low-profile trenches placed shallow in the soil profile benefit from higher porosity, organic matter, and subsurface aeration
  - Open infiltrative surface areas (e.g., chamber-equipped and similar designs without buried stones or similar media (referred to as object laden surfaces)):
    - Avoid compaction and fines from dirty gravel media
    - Avoid pore entry blockage and embedment
    - Enable inspection and maintenance as needed
  - LTARs for open infiltrative surface areas:
    - For STE and similar primary quality effluents
      LTAR for open surface $>$ object laden surface
    - For PMB effluent and similar higher quality effluents
      LTARs for open surface $\cong$ object laden surface

11.36

- Percolation of wastewater through an unsaturated soil profile
  - Wastewater that infiltrates into the soil below a soil infiltrative surface percolates downward by gravity forces
    - Due to soil clogging, unsaturated flow typically occurs and the rate of vertical water movement can be described by Eq. 11.3

$$q_u = K_u \left(\frac{dh}{dz}\right)_u \tag{11.3}$$

Where:
q_u = rate of water movement in unsaturated soil (ft/day)
K_u = unsaturated soil hydraulic conductivity (ft/day)
(K_u depends on the soil texture and water content)
dh/dz = hydraulic gradient (–) (typ. 1)

  - Depending on subsurface conditions, a portion of the infiltrated wastewater may move laterally and upward based on differences in water content and capillary forces in the soil

11.37

- Wastewater transitions to a reclaimed water
  - Wastewater influent that percolates downward is gradually transformed to a reclaimed water as constituents in the wastewater are transformed and removed within the soil profile
  - The reclaimed water typically recharges local groundwater under the site where wastewater infiltration occurs
  - The reclaimed water can undergo further treatment by attenuation processes that can occur in the deeper vadose zone and in the groundwater as it migrates away from the site

11.38

- Pollutant and pathogen transformation and removal
  - Many of the key processes in a subsurface STU are analogous to those in a single-pass sand filter (SPSF)
  - However, the processes in a STU can be more complex and dynamic than those in a SPSF because:
    - Soil has more dynamic properties and heterogeneities that affect water movement and treatment
    - A STU "lives" in the natural environment and is connected to groundwater and surface waters which can help attenuate constituents of concern and assimilate reclaimed water
  - Conceptually, removal processes in a STU occur in several zones encompassing wastewater infiltration, soil percolation, groundwater recharge and transport, and discharge to a surface water (Fig. 11.12)

11.39

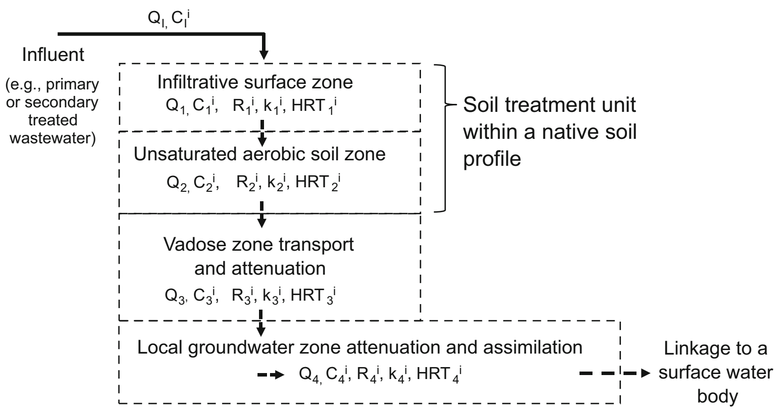


Fig. 11.12 Illustration of pollutant and pathogen removal occurring through processes in several zones within and around a STU (after Siegrist 2014) (*Note*: Q_I = influent flow rate to the STU, C_I^i = influent concentration, R = reaction in a zone, k = reaction rate, HRT = retention time, Q = flow rate and C = concentration leaving the zone, where the subscript (e.g., 1) designates a zone and the superscript (i.e., i) designates a constituent.)

11.40

- General attributes of a native soil profile important to treatment
  - Suitable saturated hydraulic conductivity (K_S) for water movement
    - Not too slow: e.g., $K_S > 1–2$ gal/day/ft^2
    - Not too fast: e.g., $K_S < 100–200$ gal/day/ft^2
  - Adequate unsaturated soil profile depth
    - Depending on wastewater HLR and composition, 2–4 ft of unsaturated soil is typically sufficient
  - Conditions conducive to removal of pollutants and pathogens
    - Unsaturated aerobic soil with film flow over soil grains and long travel times for kinetic processes (e.g., BOD and NH_4^+ removal, virus inactivation)
    - Adequate volume of soil to provide grain surface area for biofilms and sorption reactions (e.g., P removal)
    - Properties conducive to treatment (e.g., circumneutral pH, high Eh, moderate temperatures, no biotoxins)

11.41

- At a site with generally suitable soil profile conditions, design factors affect treatment by determining the types and rates of reactions (k) plus the hydraulic retention time (HRT)
  - Wastewater HLR and composition
    - HLR and composition can affect infiltrability, which can affect uniformity of infiltration and the HRT in a soil profile
  - Method of wastewater delivery and uniformity of application
    - The application method can affect uniformity of infiltration, unsaturated flow conditions, and the HRT
  - Infiltration depth and unsaturated soil properties
    - Depth affects aeration and plant-based processes
    - Unsaturated soil thickness affects aeration and HRT
    - Soil properties can affect wastewater movement and reaction types and rates (e.g., pH, Eh, mineralogy, natural organic matter content)

13.42

- Treatment of constituents often occurs by kinetic reactions
  - A very simplified estimate of removal for BOD as an example can be made assuming uniform unsaturated flow and 1st-order kinetics (Eqs. 11.5 and 11.6) (Fig. 11.13)
  - For some constituents and conditions a more complex model is required to properly capture the flow and transport processes involved in treatment (e.g., STUMOD or Hydrus 2D)

$$R_E = \left(1 - e^{-kt}\right) \times 100\% \quad (11.5) \qquad t = HRT = \frac{(d_u)(n_e)}{HLR} \quad (11.6)$$

Where:
R_E = removal efficiency (%)
k = 1st-order reaction rate (h^{-1})
k: BOD = 0.04–0.09; NH_4^+ = 0.4–0.9; Fecal coli. = 0.1–0.3
HRT = hydraulic retention time for removal reaction (h)
d_u = unsaturated soil depth (ft)
n_e = effective porosity (v/v)
HLR = hydraulic loading rate (ft/h)

Source: after Siegrist 2007.

11.43

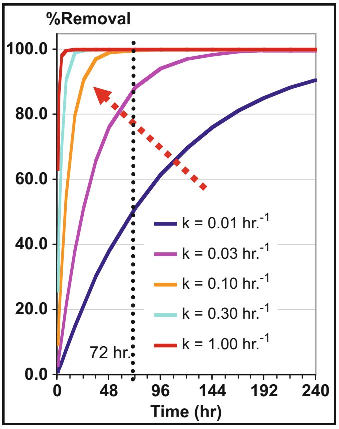


Fig. 11.13 Illustration of removal efficiencies as a function of 1st-order rate constants under example conditions during subsurface soil infiltration (HLR = 1 gal/day/ft^2, soil profile travel distance = 2 ft with n_e = 0.2, HRT = 72 h) (after Siegrist 2007)

11.44

- Effects of temperature on reaction rates
  - Temperature affects the rates of processes during biological treatment
  - Temperature effects on biological reaction rate constants can be significant and are expressed by Eq. 11.7

$$k_T = k_{20}\theta^{(T-20)} \tag{11.7}$$

Where:
k_T = reaction rate at temperature, T (°C)
k_{20} = reaction rate at 20 °C
T = temperature (°C)
θ = temperature activity coefficient (–)
In activated sludge biological systems, θ for BOD removal can be about 1.02–1.06 and some adopt values in this range for use with subsurface soil infiltration

11.45

- Treatment effects—Uniformity of infiltration
  - Infiltration—by engineered distribution or due to soil clogging—affects the IR, HRT and treatment efficiency (Fig. 11.14)

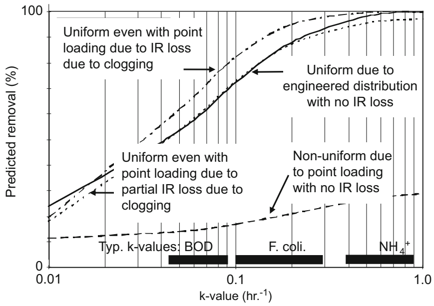


Fig. 11.14 Illustration of the purification effects of uniformity of infiltration which impacts the HRT in the soil (after Ausland 1998) (*Note*: removal predicted using Eq. 11.5 with HRT values determined by tracer testing during application of 2.4 gal/day/ft^2 through 3 ft of sand (d_{50} = 0.86 mm))

11.46

- Treatment effects—Unsaturated soil thickness
  - The thickness of the unsaturated soil beneath the soil infiltrative surface (d_U) impacts HRT, which affects the removal efficiency achieved by kinetic reactions (e.g., BOD removal)
    - d_U also impacts the extent of wastewater contact with soil grain surface areas, which impacts sorption (e.g., P removal)
  - Wastewater infiltration can change the unsaturated soil thickness by causing:
    - A perched zone of saturation above a low permeability layer in the soil profile or mounding of a local groundwater table
  - For STUs handling larger flows, the hydrologic effects of wastewater application on d_U must be carefully considered
  - Unsaturated soil thickness is normally assessed through a site investigation and during design (discussion to follow)

11.47

- Treatment effects—Layering in the soil profile
  - If there are layers in a soil profile with different grain size properties, contrasting K_S values and capillary forces can cause saturation at the boundary, e.g.:
    - Sandy loam over a coarse sand—saturation can occur in the finer grained sandy loam (lower K_S and higher Ψ_U)
    - Coarse sand over sandy loam—saturation can occur in the coarse sand (higher K_S compared to sandy loam)
  - Depending on conditions, layering can cause a sequence of aerobic and anaerobic zones in a soil profile
    - This can be important to removal of pollutants that benefit from aerobic conditions followed by anaerobic conditions
    - For example, nitrification can occur in an aerobic zone and denitrification can occur in an underlying anaerobic zone

11.48

- Treatment effects—Presence of rock fragments
  - Rock fragments (>2-mm diam.) within a soil profile occupy volume and can have varied effects on water movement and treatment processes (Fig. 11.15)
    - A reduction in the bulk soil porosity can lead to an increased water-filled porosity and reduced aeration
    - A reduced porosity can increase the rate of water movement, which can reduce the HRT and treatment achieved in a given depth interval
  - Reduction in treatment capacity due to rock fragments?
    - For treatment of domestic STE, it is reasonable to limit the volume occupied by stones (>2-mm diameter) to < 35 % of the bulk volume
    - For profiles with 35–60 % by volume, it is advised to use a buried sand filter design or provide a higher degree of treatment prior to discharge to the soil (e.g., secondary rather than primary)

11.49

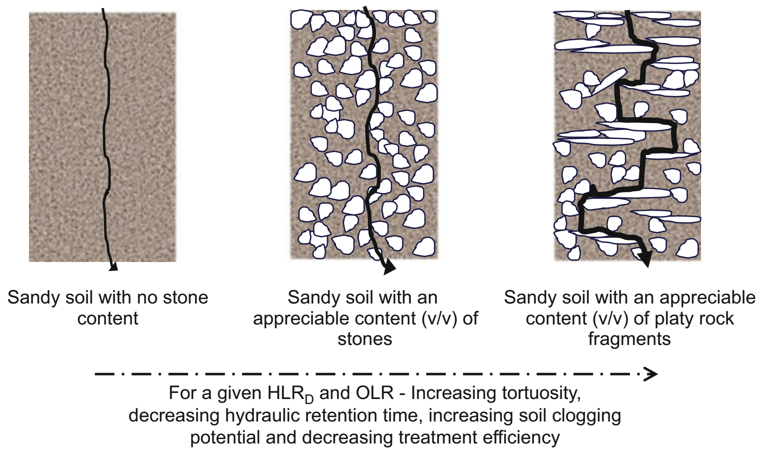


Fig. 11.15 Illustration of rock fragments and potential effects on tortuosity and water movement, hydraulic retention time, soil clogging potential, and treatment efficiency

11.50

- Attenuation and assimilation in the subsurface
  - Percolate from a STU commonly recharges groundwater
    - Groundwater is typically flowing in some direction toward a surface water (e.g., stream, lake, estuary)
  - Recharge can cause a plume of affected groundwater
    - This plume can migrate for a short (10 s of ft) or long (100 s of ft or more) distance away from the location of the STU
      - At some location the plume dissipates sufficiently that it is no longer distinguishable from unaffected local groundwater
      - The extent of an identifiable plume depends on aquifer thickness, flow velocity, and biogeochemical conditions
    - The plume may or may not be a concern re: water quality deterioration and risk
  - A schematic illustration of attenuation and assimilation under and away from a STU is shown in Fig. 11.16

11.51

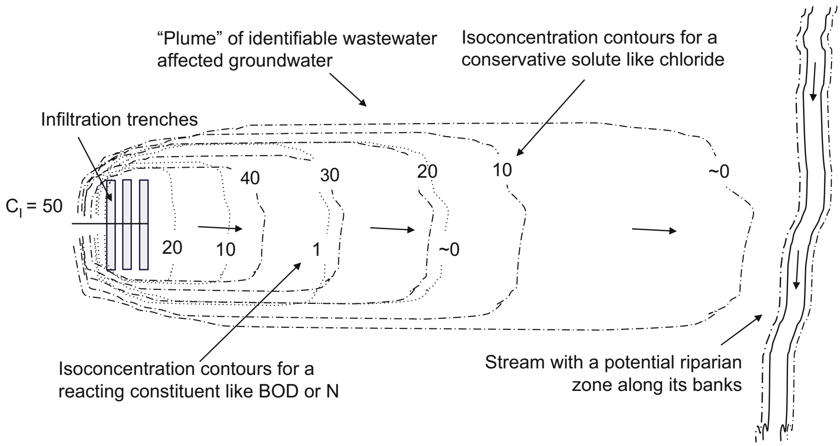


Fig. 11.16 Plan view illustration of a STU and a constituent plume, the nature and extent of which depends on subsurface conditions and the constituent attenuation and assimilation under and away from the location of infiltration

11.52

11-4. Design and Implementation

- Considerations for design and implementation (D&I) of a soil treatment unit to achieve tertiary treatment and natural disinfection
  - Treatment goals and method of assessment
  - Site evaluation and suitability assessment
  - Treatment options prior to wastewater application to the soil
  - Infiltrative surface architecture
  - Wastewater application rates for infiltration area sizing
  - Depth of soil required beneath the infiltrative surface
  - Landscape placement and layout
  - Effluent delivery and distribution, resting and cyclic loading
  - Design for long-term service
  - Installation and startup, operation and maintenance, monitoring
  - Overcoming site limitations and use of design variants
  - Modeling tools for system design and assessment

11.53

- D&I considerations—Treatment goals and assessment
  - For a STU, treatment goals can be set in different ways
    - It depends on how and where boundaries and points of compliance are set or required
    - Treatment goals can include the performance of the STU alone (based on boundaries defined) or include the attenuation within the deeper vadose zone and receiving groundwater, e.g.:
      - NO_3—N concentrations will be <10 mg-N/L when the reclaimed water reaches the groundwater table, or
      - NO_3—N concentrations in groundwater at the property boundary will be <10 mg-N/L
  - Assessments can be done *a priori* or after operation startup
    - Predictions can be made using modeling tools (e.g., STUMOD)
    - Monitoring can be carried out after system startup

11.54

- Assessment of treatment efficiency and achievement of treatment goals requires boundaries and points of compliance as illustrated in Figs. 11.17 and 11.18

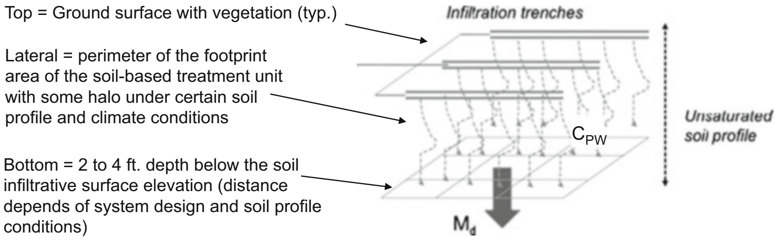


Fig. 11.17 Illustration of one approach to setting boundaries for the top, bottom, and lateral planes associated with treatment within a STU

11.55

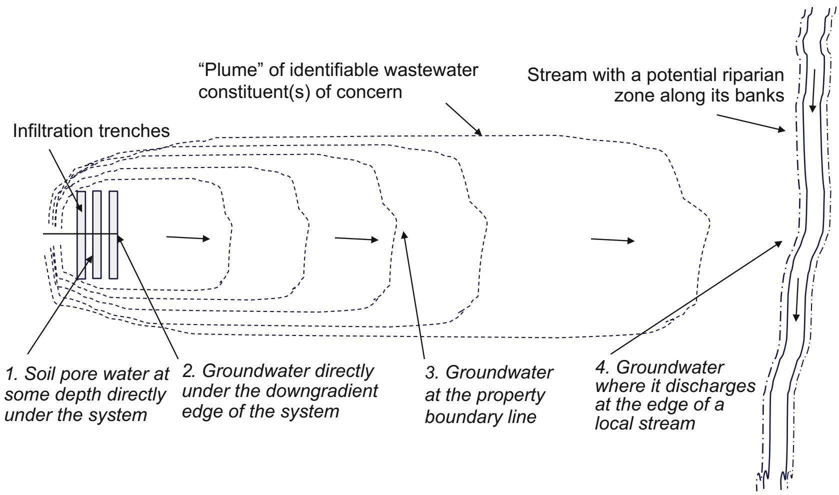


Fig. 11.18 Plan view illustration of a STU and four potential points of compliance that might be used to assess achievement of a treatment goal

11.56

- Assessment of treatment efficiency requires data for concentration (C_{PW} or C_{GW}) or mass discharge (M_d) as illustrated in Fig. 11.19

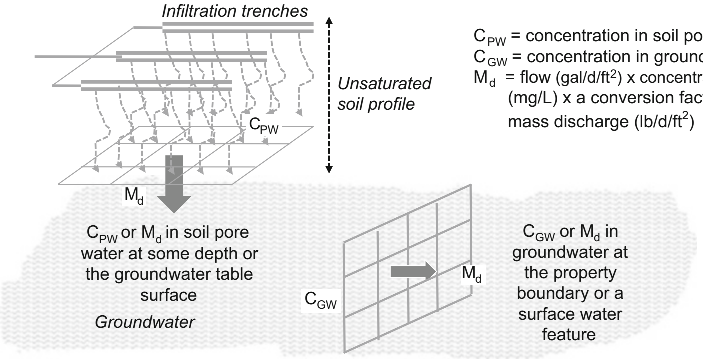


Fig. 11.19 Treatment performance can be assessed using a mass discharge approach or one that assesses changes in concentrations (*Note*: you can model and predict C_{PW}, C_{GW} or M_d or you can use monitoring data to estimate these parameters)

11.57

- Mathematical models can help assess treatment efficiency for different STU designs and environmental settings
  - STUMOD is a spreadsheet-based analytical model that was initially developed to simulate nitrogen concentrations or mass flux with depth under a soil infiltrative surface (Geza et al. 2009, 2014)
    - Figure 11.20 presents simulation results for a current version of STUMOD and an example set of conditions
    - Recent refinements to STUMOD have focused on including evapotranspiration and plant uptake and nitrogen transport and fate in an underlying groundwater system
    - Future plans are to extend STUMOD so it can simulate transport and fate of other wastewater pollutants and pathogens and to generate generalized export coefficients for catchments that can be used in watershed scale assessment and modeling

11.58

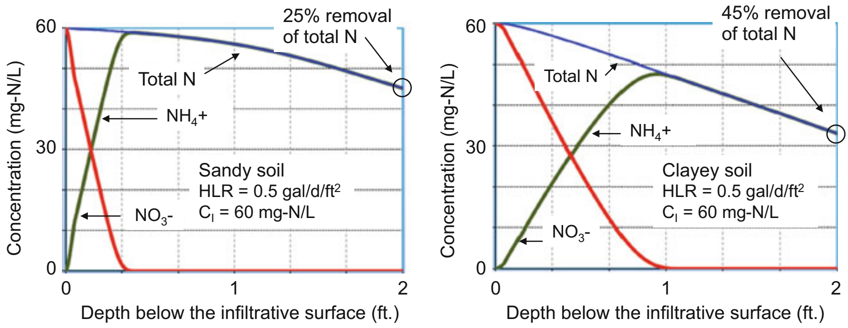


Fig. 11.20 Example outputs from simulations completed with STUMOD to predict nitrogen concentrations with depth below the infiltrative surface for a STU installed in sandy versus clayey soils (Geza et al. 2014) (*Note*: the removal of total nitrogen is predicted by 2 ft depth to be 45 % in clayey soilds compared to 25 % in sandy soils due to a higher denitrification rate in the higher water content clayey soils)

11.59

- D&I considerations—Evaluation of site suitability
  - Components of a typical site evaluation appear in Table 11.5

Table 11.5 Typical components of a site evaluation used for assessing suitability for a STU

Evaluation component	Conditions that support implementation of a STU
Land surface features	Area for the STU can not be in a floodplain or under a structure; well-drained areas are desirable but the land surface slope should not be too excessive
Saturated hydraulic conductivity of the soil profile	Not too high since this could lead to poor treatment or too low since this could lead to hydraulic capacity problems and anaerobic soil conditions
Depth to a limiting condition	Need adequate depth of unsaturated soil for treatment before reaching a low permeability layer, bedrock surface, or groundwater table
Sufficient land area	Need minimum setback distances to property lines and buildings, drinking water wells, cut banks, and surface waters to allow for treatment and assimilation
Assimilative capacity	The site must be able to assimilate the daily flow discharged and the pollutants and pathogens of concern; this is a particular concern for larger STUs needed to handle high daily flows

11.60

- Key methods used during a site evaluation commonly include:
  - Review of existing information
    - e.g., land use, topographic maps, environmental reports, Web Soil Survey, etc.
  - Site characterization field observations along with sampling and lab analyses to determine:
    - Available landscape area, topography, drainage features, etc.
    - Soil profile morphology and soil properties
      - Hydraulic properties (e.g., layering, texture, structure, color, etc.)
      - Treatment properties (e.g., mineralogy, aeration status, etc.)
  - Larger systems require very careful evaluation of a site's overall "assimilative capacity" for water, pollutants and pathogens

11.61

- Site suitability is judged based on comparison of the site evaluation results against a set of requirements (e.g., Table 11.6)

Table 11.6 Example requirements for judging the suitability of a site for a STU

Site suitability requirements for the State of Colorado	Range of values[a]
Soil depth below an infiltrative surface to a limiting condition	2–4 ft
Saturated hydraulic conductivity as evidenced by a percolation rate (*Note*: A percolation test is a crude measure that is still used in some locations but it is not recommended as a major determinant of site suitability)	<5–120 min/in.
Maximum slope of the land surface (for common designs)	30 %
Setback distance from groundwater wells	75–100 ft
Setback distance from cut bank or dry gulch	10–25 ft
Setback distance from nearby surface waters	25–50 ft

[a]Examples shown are based on requirements in Colorado Regulation 43 (CDPHE 2013). Note the values for these parameters can vary depending on the design flow being handled, the effluent quality being treated in the STU, and the system design attributes.

11.62

- D&I considerations—Treatment prior to the STU
  - At a minimum, primary or advanced primary treatment of wastewater using a septic tank or similar unit is needed
  - Secondary treatment (e.g., using an aerobic unit or porous media biofilter) can be warranted to enable soil-based treatment in otherwise challenging sites, such as where:
    - Subsurface soils have a low K_S and reduced aeration
    - Subsurface soils have a high K_S and a low HRT
    - Limited land area requires a higher HLR_D
  - Tertiary treatment using a membrane bioreactor or similar technology may be warranted in some areas, for example:
    - To enable a very high HLR_D to a small footprint STU in a high density development
    - To enable use of a STU for commercial applications in locations with sensitive water quality conditions

11.63

- Classification of wastewater effluents applied to STUs
  - Classification can be based on composition characteristics that control infiltrability and are also important to treatment efficiency
  - An effluent classification scheme, first proposed by Siegrist in 2006, is given in Table 11.7

Table 11.7 Classification scheme for the effluents applied to a STU (Siegrist 2014)

Types of effluents applied to a STU	Effluent composition			Example unit operation to achieve an effluent type[a]
	$cBOD_5$ (mg/L)	TKN (mg-N/L)	TSS (mg/L)	
Type I	150	60	75	Septic tank with effluent screen
Type II	30	5	30	Aerobic treatment unit
Type III	5	5	5	Porous media biofilter, Membrane bioreactor

[a]Note that other treatment systems could provide the composition shown for Types I to III effluents.

11.64

- An effluent classification scheme published in regulations adopted in Colorado in 2013 is shown in Table 11.8

Table 11.8 Example treatment level classification scheme prescribed in Colorado for effluents applied to a STU (CDPHE 2013)

Treatment level	$cBOD_5$[a] (mg/L)	TSS (mg/L)	Total Nitrogen (mg/L)
TL 1[b]	145	80	60–80 mg/L
TL 2	25	30	60–80 mg/L
TL 2 N	25	30	>50 % reduction[c]
TL 3	10	10	40–60 mg/L
TL 3 N	10	10	20 mg/L

[a] $cBOD_5$ can be estimated as 0.85 × total BOD_5.
[b] Values for TL 1 are typical but design must account for site-specific information.
[c] NSF/ANSI Standard 245—Wastewater Treatment Systems—Nitrogen Reduction requires. reduction of 50 % rather than achieving a specific value.

11.65

- D&I considerations—Infiltrative surface architecture
  - A soil infiltrative surface can be established in the subsurface with a horizontal or vertical orientation within one or more excavated trenches or beds
  - In general, narrow trenches with short sidewall heights that are placed just below the ground surface are preferred for several reasons
    - Shallow depth below ground surface—the infiltrative surface is located in a more biogeochemically active depth zone
    - Narrow width and short height—the path length for oxygen transport from the atmosphere to the infiltrative surface is shortened leading to better re-aeration of the soil beneath and around the STU (Fig. 11.21)
    - Narrow width—The risk of damage caused during construction is lessoned since heavy equipment does not have to operate on the soil infiltrative surface and there is typically more side-wall as a reserve area for infiltration

11.66

- Soil infiltrative surface orientation (Fig. 11.21)
  - At startup, only the horizontal infiltrative surface is used
  - With operation, the HLR can exceed the infiltrability of the horizontal infiltrative surface and ponding may develop
    - With ponding, infiltration also occurs through the vertical sidewall area
    - As biomat development and pore-filling evolves, ponding depth can increase and more vertical sidewall area is used
  - Sizing based on bottom area alone leaves the sidewall area to provide extra infiltrative surface area as needed

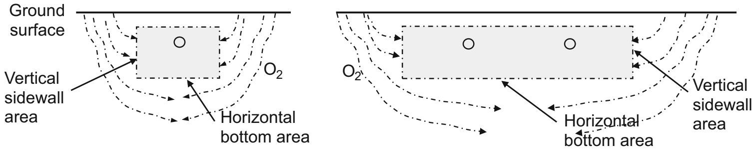


Fig. 11.21 Cross-section view of a trench (*left*) versus bed (*right*) geometry illustrating horizontal bottom area versus vertical sidewall area for infiltration

11.67

- Geometry of trenches and beds
  - Suggested trench dimensions
    - Width $\leq$ 3 ft, Height (sidewall) $<$ 2 ft, Length $<$ 100 ft.
  - Suggested bed dimensions
    - Width $\leq$ 12 ft, Height (sidewall) $<$ 2 ft, Length $<$ 100 ft.
  - Avoid wide beds, especially for primary treated wastewaters which have high O_2 demands due to high BOD and NH_4^+ levels
- Placement of the soil infiltrative surface
  - Suggested placement of the horizontal infiltrative surface at 1–3 ft bgs
    - Deeper placement can be considered for some sites
      - e.g., profile layering might be conducive to infiltration below a shallow low permeability layer
  - Placement needs to provide adequate unsaturated soil depth below the infiltrative surface to a limiting condition
    - If site conditions warrant it, a STU can be established at grade or in a mound of fill sand (discussed later)

11.68

- Use of bigger beds under special circumstances
  - Trenches are generally preferred but they can have practical disadvantages or be infeasible due to land area constraints
    - Trenches require more landscape footprint area for a given horizontal infiltrative surface area
    - Trenches can take more time for installation
  - Bed geometries that have widths greater than 12 ft can be appropriate for some applications
    - With secondary and tertiary effluents where O_2 demands are very low and soil clogging is very limited
    - Where only part of the horizontal surface within the bed is used to provide a soil infiltrative surface—e.g., with chambers placed on the bottom of the bed with unused area as spacing between adjacent chambers
  - Installation of bigger wider beds must be done very carefully to avoid construction damage to the soil infiltrative surface

11.69

- Infiltrative surface features
  - Various options have been used to establish a soil infiltrative surface to which treated wastewater effluent can be applied
  - Gravel was widely used in the past since it was locally available and relatively inexpensive
  - But in the late 1900s, research findings revealed several potential negative aspects of using gravel for establishing an infiltrative surface, including:
    - Gravel can cause soil compaction when dumped onto a soil infiltrative surface
    - Fines can be washed off gravel and cause soil clogging
    - Gravel can mask soil pore entries and get embedded into a gravel-soil zone with a lower K_S and higher clogging potential
    - There can be life-cycle costs due to excavation, processing, and hauling gravel from a quarry to a project site

11.70

- Alternative aggregate materials emerged
  - Alternatives included lightweight expanded clay aggregate, glass fragments, shredded tire chips, or bundled plastic beads
  - These media were lighter than gravel, did not cause compaction, did not have fines and in some cases were made from recycled materials
- Manufactured products were also developed
  - Chambers were developed to use in place of aggregate
  - Chambers eliminate the issue with gravel compaction and fines, but also avoid pore entry blockage and embedment, and enable inspection and maintenance of the infiltrative surface
- Table 11.9 highlights the features of two contrasting options—gravel-filled versus chamber-equipped

11.71

Table 11.9 Examples of two contrasting methods to enable access to a below-ground soil infiltrative surface[a]

	Examples of two contrasting methods and description	Comments
Gravel	Gravel (e.g., 0.75–1.5 in. diameter) is dumped onto an excavated soil infiltrative surface within a trench or bed and piled up to a depth of 0.5 of 1.0 ft or more. Effluent distribution piping and observation ports (extending to the infiltrative surface) can be placed in the gravel. The gravel layer is covered with a geotextile fabric and then backfilled with native soil up to ground surface. Effluent distributed into the gravel can infiltrate into the undisturbed soil and if needed due to soil clogging, effluent can intermittently or continuously pond up in the gravel layer	Gravel needs to be locally available and needs to be washed to remove fines; gravel can cause compaction, masking and embedment of the soil infiltrative surface
Chambers	Manufactured products made in the form of a half-cylinder serve as a chamber that is placed on top of the soil infiltrative surface in a trench or bed. Effluent distribution piping can be suspended in the crown of the chamber and observation ports can be placed into it. The chamber is covered with native soil on the sides and top up to ground surface. Effluent distributed into the chamber can infiltrate into the open undisturbed soil and if needed due to soil clogging, effluent can intermittently or continuously pond up within the chamber	Chambers are lightweight and can be shipped to a project site; chambers avoid fines and embedment and maintain a completely open soil infiltrative surface

[a]*Note*: There are other optional methods that have been used including the use of substitutes for gravel such as light expanded clay aggregates, shredded tire fragments, glass fragments, and bundled plastic beads or alternatives to chambers such as fabric wrapped large diameter corrugated piping.

11.72

- D&I considerations—Design hydraulic loading rate
  - Design hydraulic loading rates are set to account for:
    - Hydraulic considerations—Need for long-term infiltration and processing of the wastewater effluent applied to the soil
    - Treatment considerations—Need for treatment to remove pollutants and pathogens to an acceptable level
  - Design hydraulic loading rates are normally controlled by hydraulic considerations and the need to manage soil clogging
    - Hydraulic loading rate for design
      - HLR_D = design hydraulic loading rate based on soil classification and effluent quality with adjustments for design attributes and soil clogging processes
    - Long-term acceptance rate
      - LTAR = infiltrability that exists after a period of operation but prior to the development of continuous ponding

11.73

- Selection of a HLR_D for a particular site and STU design
  - The HLR_D is not just an inherent property of a soil profile, but is "system conditional" and depends on soil and site conditions but also wastewater composition, STU design features, operation, etc.
  - HLR_D is normally set based on soil clogging and infiltrability considerations
    - Higher HLR_D can be used for higher quality effluents
    - But, even for high quality effluents, the HLR_D should be $\leq$ 5–10 % of the K_S in the depth interval of the infiltrative surface to help maintain unsaturated aerobic conditions
  - In addition, the HLR_D must not exceed the hydraulic and treatment capacity of the entire soil profile and site
    - The HLR_D can not cause excessive groundwater mounding on potential low K_S zones or a shallow groundwater table
    - The HLR_D needs to provide an adequate HRT for treatment

11.74

- The value of HLR_D is typically limited to a fraction of the native soil K_S
  * Benefits of setting $HLR_D << K_S$ (Eq. 11.7)
    – Provides unsaturated flow in the soil profile which aids treatment: soil profile aeration can occur, wastewater flows in films over biofilm coated soil particles, and wastewater remains in the soil profile for a long HRT
    – Accounts for the loss in infiltrability during operation
    – Ensures that the HLR_D can be processed over the design lifetime

$$HLR_D \leq F \times K_S \tag{11.7}$$

Where:
HLR_D = hydraulic loading rate used in design (gal/day/ft^2)
F = factor to account for long-term wastewater infiltration
e.g., F = 0.05 for STE and up to F = 0.10 for MBR effluent
K_S = saturated hydraulic conductivity of the native soil (gal/day/ft^2)

11.75

- Setting a HLR_D—a simplified approach
  - Soil profile conditions can be classified as shown in Table 11.10
    * Based on hydraulic and purification processes, consider three major soil classes with different representative K_S
    * Exclude soil profiles with K_S values that are too high or too low unless design modifications are made

Table 11.10 Example classification scheme for soil profiles (Siegrist 2006)

Soil class	Representative soil textures in the soil profile	Representative clean water K_S (gal/day/ft^2)	Maximum daily HLR for clean water infiltration
Class I	Sand, loamy sand	245	Maximum HLR_D for clean water should not exceed 5 % to 10 % of the K_S of the profile in the infiltration zone
Class II	Sandy loam, loam, silt loam	24.5	
Class III	Silty clay loam, clay loam	2.5	

Source: After Siegrist (2006, 2007, 2014).
Colorado Reg. 43 uses an approach based on the approach given here (see Table 11.12).

11.76

- Selection of a baseline HLR
  - Examples schemes based on wastewater effluent type and soil profile classification appear in Tables 11.11 and 11.12

Table 11.11 Example of a classification scheme proposed for selecting a HLR_D (Siegrist 2006)

Effluent type	Effluent composition (mg/L)	Example treatment to achieve an effluent type	Baseline HLR[a] (gal/day/ft^2)		
			Class I (Sand, loamy sand)	Class II (Sandy loam, silt loam)	Class III (Silty clay loam, clay loam)
Type I	$cBOD_5 = 150$ $TKN = 60$ $TSS = 75$	Septic tank with effluent screen	1	0.5	0.12
Type II	$cBOD_5 = 30$ $TKN = 5$ $TSS = 30$	Fixed film aerobic treatment unit	2.5	1	0.12
Type III	$cBOD_5 = 5$ $TKN = 5$ $TSS = 5$	Packed bed recirculating biofilter	5	2	0.25

[a]For an open horizontal infiltrative surface with continuous usage during a normal service life (e.g., 20 year).

11.77

Table 11.12 Example of a regulatory approach for setting HLR_D (CDPHE 2013)

Soil Type, Texture, Structure, Percolation Rate Range					Long-term Acceptance Rate (LTAR) (gal/d/ft^2)				
Soil type	USDA soil texture	USDA soil structure-shape	USDA soil structure-grade	Percolation rate (MPI)	Treatment Level 1[2]	Treatment Level 2[2]	Treatment Level 2N[2]	Treatment Level 3[2]	Treatment Level 3N[2,3]
0	Soil Type 1 with more than 35% Rock (>2mm); Soil Types 2-5 with more than 50% rock (>2mm)	-	0	<5	Min. 3-ft. deep unlined sand filter required	Minimum 2-foot deep unlined sand filter required[2]			
1	Sand, Loamy Sand	-	0	5-15	0.80	1.25	1.25	1.40	1.40
2	Sandy Loam, Loam, Silt Loam	PR (prismatic) BK (blocky) GR (granular)	2 (Moderate) 3 (Strong)	16-25	0.60	0.90	0.90	1.00	1.00
2A	Sandy Loam, Loam, Silt Loam	PR, BK, GR 0 (none)	1 (Weak) Massive	26-40	0.50	0.70	0.70	0.80	0.80
3	Sandy Clay Loam, Clay Loam, Silty Clay Loam	PR, BK, GR	2, 3	41-60	0.35	0.50	0.50	0.60	0.60
3A	Sandy Clay Loam, Clay Loam, Silty Clay Loam	PR, BK, GR 0	1 Massive	61-75	0.30	0.40	0.40	0.50	0.50
4	Sandy Clay, Clay, Silty Clay	PR, BK, GR	2, 3	76-90	0.20	0.30	0.30	0.30	0.30
4A	Sandy Clay, Clay, Silty Clay	PR, BK, GR 0	1 Massive	91-120	0.15	0.20	0.20	0.20	0.20
5	Soil Types 2 – 4A	Platy	1, 2, 3	121+	0.10	0.15	0.15	0.15	0.15

Source: Table 10.1, Co. Reg. 43, 2013. 11.78

- D&I considerations—Infiltrative surface area required
  - Once the HLR_D is chosen for a given STU design, the infiltrative surface area can be determined using Eqs. 11.8 or 11.9

$$A_{IS} = \left[\frac{Q_D}{HLR_D}\right]\left(\frac{1}{EF}\right) \tag{11.8}$$

$$A_{IS} = \left[\frac{Q_D}{HLR_D}\right](AF) \tag{11.9}$$

Where:
A_{IS} = area of the soil infiltrative surface (ft^2)
Q_D = design daily flow (gal/day)
HLR_D = design hydraulic loading rate (gal/day/ft^2) (Tables 11.11 or 11.12)
EF = infiltration efficiency factor = ƒ (design, construction, operation) (–)
AF = area adjustment factor = ƒ (design, construction, operation) (–)

11.79

- Efficiency Factors (EF) can be used to account for the infiltration effects of design, construction, and operation (Table 11.13)

Table 11.13 Examples of efficiency factors used to adjust the area of the soil infiltrative surface (Siegrist 2006, 2007, 2014)

Construction or operation feature	Efficiency Factor	Efficiency factor accounts for:
Construction impacts	0.1 or less	Account for the loss in clean-water K_S due to compaction and smearing during installation
Infiltrative surface architecture	0.50–0.75	Account for loss in LTAR due to solid objects including effects of fines and embedment and greater difficulty for monitoring and rehabilitation
	1.0	Open infiltrative surface established with a chamber or similar technology
Discontinuous operation during normal 20-year life	1.5–2.0	Account for elevated hydraulic and treatment capacity due to extended rest periods during cyclic operation; e.g., 1 year online and 3 years offline
Relatively shorter design service life	2.0–4.0	Account for higher capacity even at higher HLR_D during only a short (1–5-year) design life

11.80

- Area Adjustment Factors (AF) can be used instead of EF (Table 11.14)

Table 11.14 Examples of Area Adjustment Factors used to adjust the area of the soil infiltrative surface required (CDPHE 2013)

Type of soil treatment area	Method of effluent application to soil treatment area					
	Gravity		Dosed (siphon or pump)		Pressure dosed	
	AF	EF[a]	AF	EF[a]	AF	EF = 1/AF
Trench	1.0	1.0	0.9	1.11	0.8	1.25
Bed	1.2	0.83	1.1	0.91	1.0	1.0
Type of soil treatment area	Type of storage/distribution media used with treatment level 1					
	Rock or tire chips		Manufactured media other than chambers		Chambers	
Trench or Bed	1.0	1.0	0.9	1.11	0.7	1.42

- Example infiltrative surface areas for different site conditions and design choices are shown in Table 11.15

11.81

Table 11.15 Infiltrative surface areas required to handle a design flow of 1000 gal/day for different example site conditions and design choices

Site and design conditions[a, b]				Infiltrative surface area at the given HLR_D and EF or AF		
Soil	Effluent	Geometry	Infiltrative Surface	HLR_D (gal/day/ft^2)	EF or AF (–)	A_{IS} (ft^2)
Type 1[a]: Loamy sand	Trt. level 2 N[a]: Sec. effluent w/N removal	2 ft wide trenches	Chamber	1.25	AF = 0.8	640
Type 3[a]: Sandy clay loam	Trt. Level 1[a]: Septic tank effluent	10 ft wide bed	Gravel	0.35	AF = 1.2	3429
Class III[b]: Silt loam	Type I[b]: Septic tank effluent	10 ft wide bed	Gravel	0.5	EF = 0.5	4000
		2 ft wide trenches	Chamber	0.5	EF = 1.0	2000
Class I[b]: Sand	Type III[b]: Sand filter effluent	2 ft wide trenches	Chamber	4.9	EF = 1.0	204

[a]Based on requirements in Colorado Reg. 43 (CDPHE 2013). [b]Based on the scheme of Siegrist (2006, 2007, 2014).

11.82

- With the area determined based on the HLR_D, the organic loading rate needs to be checked using Eq. 11.10
  - If the OLR is too high, anoxic or anaerobic conditions may develop in the soil profile under the soil infiltrative surface
  - Suggested OLR limits are shown in Table 11.16
    - If the OLR is too high, additional A_{IS} is required or treatment must reduce the BOD_5 applied to the STU

$$OLR = \frac{(Q_D)(BOD_5)(F)}{A'_{IS}} \tag{11.10}$$

Where:
A'_{IS} = area of horizontal soil infiltrative surface provided based on the STU layout (ft^2) (see Eq. 11.11) (*Note*: A'_{IS} will be $\cong A_{IS}$)
Q_D = design daily flow (gal/day) (e.g., typ. $Q_A \times PF$, with PF = 1.5)
HLR_D = design hydraulic loading rate (gal/day/ft^2)
OLR = organic loading rate (lb-BOD_5/day per ft^2); suggested OLR limits: Class I soil = 0.001; Class II soil = 0.0005, Class III soil = 0.0002
BOD_5 = Influent BOD_5 (mg/L)
$F = 8.34 \times 10^{-6}$ = conversion factor for mg/L to lb/gal

11.83

Table 11.16 Organic loading rates to a soil infiltrative surface

	Calculated organic loading rate (lb-BOD_5/day per ft^2)				
HLR_D (gal/day/ft^2)	BOD_5 = 5 mg/L	BOD_5 = 30 mg/L	BOD_5 = 150 mg/L	BOD_5 = 200 mg/L	BOD_5 = 300 mg/L
0.2		0.00005	0.00025	0.00033	0.00050
0.4		0.00010	0.00050	0.00066	0.00100
0.6		0.00015	0.00075	0.00100	
0.8		0.00020	0.00100[a]	Suggested OLR limits: Class I soil = 0.001 Class II soil = 0.0005 Class III = 0.0002	
1.0	0.00004	0.00025			
2.0	0.00008	0.00050			
4.0	0.00017	0.00100			
25.0	0.00104				

[a]*Note:* USEPA (2002) recommended maximum OLR for domestic STE with BOD_5 = 150 mg/L.

11.84

- D&I considerations—Unsaturated soil thickness needed
  - Minimum depth to a limiting condition (e.g., groundwater, bedrock) accounting for capillary rise and preferential flow
    - Type I to III effluents in coarse grained Class I soils: > 2 ft.
    - Type I to III effluents in structured Class II and III soils: > 3 ft.
  - Depth requirements for special conditions
    - For larger systems and sites with low assimilative capacity for reclaimed water recharge into the local groundwater
      - Need to consider and evaluate water mounding potential
    - For higher strength wastewater effluents where Type I effluent may not be reliably achieved, or where certain pollutants may still be at unusually high levels (e.g., N)
      - Need to evaluate need for higher treatment prior to the STU and/or a greater thickness of unsaturated soil

11.85

- D&I considerations—Landscape placement and layout
  - A STU should be placed in well-drained, upslope locations within reasonable proximity to the pre-STU treatment units
  - Infiltration trenches or beds should be oriented along landscape contours (Fig. 11.22)
    - Enables the soil infiltrative surface to be placed shallow
    - Minimizes linear-loading rates (LLR) and reduces groundwater perching or mounding effects on unsaturated soil thickness
      - LLR = gal/day per ft of STU length along the slope

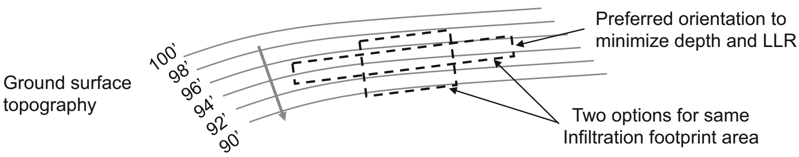


Fig. 11.22 Illustration of the preferred orientation of a STU on a sloping site

11.86

- Layout of trenches or narrow beds
  - Based on the A_{IS} required and the landscape features, the details of the layout for the STU need to be determined
    - A layout needs to be designed with consideration of effluent delivery and distribution options
    - The width of a trench (e.g., 2 ft) or narrow bed (e.g., 10 ft) and the length (e.g., 50 ft) need to be selected
      – Figure 11.23 shows 2 layout options for the same A_{IS}

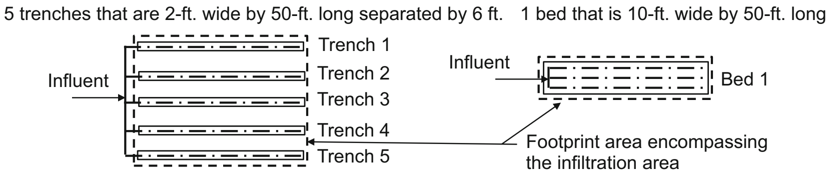


Fig. 11.23 Illustration of a trench layout versus a bed layout, both of which provide the same horizontal infiltrative surface area

11.87

- Based on the geometry and general layout chosen, the length and number of infiltration units can be determined using Eqs. 11.11 and 11.12
  - Then a specific layout with dimensions can be determined and the infiltrative surface can be calculated (Eq. 11.13)

$$L_T = \left(\frac{A_{IS}}{W_I}\right) \quad (11.11) \qquad n = \left(\frac{L_T}{L_I}\right) \quad (11.12)$$

$$A'_{IS} = n(L_I \times W_I) \quad (11.13)$$

Where:
L_T = total length of trenches or narrow beds required at the selected width (ft)
A_{IS} = infiltrative surface area required within the infiltration unit (ft^2)
A'_{IS} = infiltrative surface area provided based on the final layout (ft^2)
W_I = width of an individual trench or narrow bed (ft)
L_I = length of an individual trench or bed (ft)
n = number of narrow trenches or beds (–) (to be assembled in a layout)

11.88

- The landscape footprint area (A_F)
  - The A_F required includes A_{IS} plus any land between adjacent infiltration units (Fig. 11.24)
    - Undisturbed land enables construction without driving heavy equipment on the infiltrative surface and also provides for soil aeration[d]
    - Typically, undisturbed land between adjacent trenches or narrow beds is equal to a % of the trench or bed width

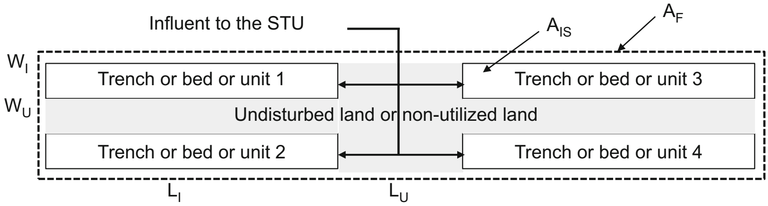


Fig. 11.24 Illustration of the footprint area required for a set of trenches or narrow beds or a set of separated chambers placed within a larger bed

[d]*Note:* If a larger bed is used with multiple infiltration chambers or units placed within it, separation between adjacent chambers or units (i.e., non-utilized land) can provide for soil re-aeration.

11.89

- For simple layouts with one or more zones along the contour (e.g. Fig. 11.22), the landscape footprint area required can be calculated using Eq. 11.14
- For more complicated layouts, geometry computations for the specific layout are required

$$A_F = (y)[(L_I)(W_I)(n) + (L_I)(W_U)(n-1)] + (y-1)\left[(L_U)(W)_I(n) + (L_U)(W_U)(n-1)\right] \quad (11.14)$$

Where:

A_F = landscape footprint area of the entire STU (ft^2)
L_I = length of the trench or narrow bed or other infiltration unit (ft)
W_I = width of the trench or narrow bed or other infiltration unit (ft)
n = number of trenches or narrow beds or other infiltration units (-)
y = number of zones with trenches or narrow beds or other infiltration units (–)
L_U = length of the undisturbed land between adjacent zones (ft)
W_U = width of the undisturbed land between each trench or bed or chamber (ft)

11.90

- D&I considerations—Influent delivery and distribution
  - Delivery and distribution of wastewater (e.g., STE) to a STU is very important to the performance achieved in the STU
  - Delivery methods transport the effluent from a pre-STU treatment unit (e.g., ST, PMB, etc.) to the site of the STU (Table 11.17)
    - Delivery methods can include:
      - Semi-continuous trickle flow under gravity
      - Intermittent dosing under pressurized flow
        - Based on a timer (timed dosing)
        - Based on wastewater generation (demand dosing)
    - Distribution methods that disperse the influent to the STU over the design soil infiltrative surface area can include:
      - Gravity flow in larger diameter perforated piping
      - Pressurized flow in small diameter piping networks

11.91

Table 11.17 Example approaches for wastewater delivery and distribution to a STU[a]

Method description	Delivery mode[b]	Typical application	Uniformity of distribution at startup and early operation
Gravity delivery of effluent into a distribution box connected to non-pressurized distribution piping	Semi-continuous trickle flow as wastewater is generated	To handle small Q_D and primary effluents in STUs located on level landscapes	Poor—Distribution is normally highly uneven (until progressive soil clogging evolves)
Gravity delivery and distribution using drop boxes connected to non-pressurized distribution piping	Semi-continuous trickle flow as wastewater is generated	To handle small Q_D and primary effluents in trench-geometry STUs located on sloping landscapes	Poor—By design, flow overloads the upslope trenches until soil clogging leads to overflow to the downslope trenches
Dosing with a pump or siphon connected to non-pressurized distribution piping	Intermittent dosing (e.g., 2–>4 per day)	To handle small to larger Q_D and primary effluents in STUs where there are multiple zones or it is located at a higher elevation than the source	Moderate—By design, this method can help achieve even delivery to different portions or zones of a STU but relatively poor distribution occurs within each of them (until progressive soil clogging evolves)

(continued)

11.92

Table 11.17 (continued)

Method description	Delivery mode[b]	Typical application	Uniformity of distribution at startup and early operation
Dosing with a pump that discharges into pressurized delivery piping connected to a manifold(s) and laterals with orifices	Intermittent dosing	For small to large Q_D with primary or higher quality effluents and any infiltration unit layout on level or sloping sites	Very good—Reasonably uniform distribution is possible if design is done properly
Dosing with a pump that discharges into pressurized delivery piping connected to a network of drip dispersal tubing	Intermittent dosing	Typically needed for higher quality effluents where soil clogging is retarded and uniform distribution is more dependent on engineering design	Very good—Near uniform distribution can be achieved if design is done properly
Dosing with a pump that discharges into pressurized delivery piping connected to laterals outfitted with spray nozzles within a chamber	Intermittent dosing		Excellent—Near uniform distribution can be achieved if design is done properly

[a]There are other delivery and distribution methods that can be used as well as hybrids of those listed in this table.
[b]Dosing can be limited to a certain number per day based on distribution system design requirements. Dosing can be triggered using a set of floats or pressure transducers for demand dosing of a few random doses per day (e.g., 0–4) or a timer for timed dosing of several doses more uniformly throughout each day (e.g., 2–> 4/day).

11.93

- Infiltrative surface utilization during startup and early operation
  - Based on the methods used (Table 11.17), during startup and early operation, the infiltrative surface utilization (ISU) in a STU is less than the entire surface area provided by design
  - Localized overloading occurs to some degree
    - For domestic STE, actual HLRs to localized areas in some STU designs can be up to 30× the HLR_D
    - Localized overloading leads to accelerated soil clogging
    - As soil clogging develops, wastewater influent spreads laterally until sufficient infiltrative area can process the daily loading
  - Progressive soil clogging leads to an ISU equal to design
    - Months to years may pass before achieving uniform distribution to the design soil infiltrative surface area
    - Non-uniformity can be prolonged when higher quality effluents are applied using delivery and distribution designs that do not achieve uniform application

11.94

- In general, dosing with pressure distribution is preferred
  - An example distribution layout is illustrated in Fig. 11.25
  - Dosing can be beneficial to treatment in all STUs by allowing alternating loading and resting
    - Enables drainage and re-aeration of the soil profile
    - Suggested dosing frequencies depend on soil properties
      - Coarse grained, fast-draining soils (e.g., Class I): >4/day
      - Fine grained, slowly-draining soil (e.g., Class III): <2/day
  - Pressure distribution can help achieve more uniform distribution to the design soil infiltrative surface area
    - Benefits overall performance with respect to hydraulic and treatment processes
  - For secondary and higher quality effluents (e.g., Type II, III), dosing and pressure distribution is essential to achieve a high ISU and a desired treatment efficiency in a STU since soil clogging may be retarded or nearly absent

11.95

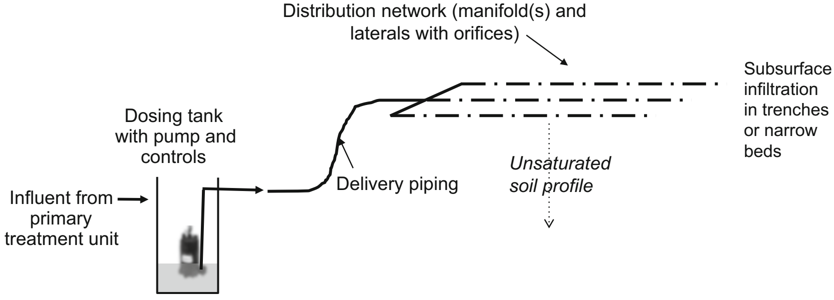


Fig. 11.25 Illustration of the layout and components of a dosing and pressurized distribution network for wastewater application to a set of subsurface infiltration trenches

11.96

- Micro-dosing and pressure distribution approaches
  - Some sites have treatment limitations
    * For example, sites where there are soils with very high K_S and inadequate unsaturated soil thickness, or shallow groundwater with nearby drinking water wells
  - Pressurized delivery and micro-dosing can help achieve highly uniform distribution and unsaturated flow conditions
    * Pressurized distribution networks can use spray nozzles (Fig. 11.26) or drip dispersal tubing (see Chap. 12)
    * Micro-dosing can deliver several to many timed doses distributed uniformly during the day (e.g., 4–12/day)

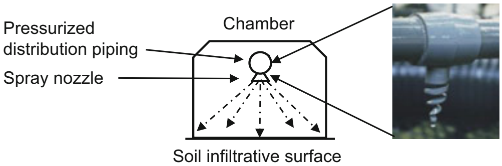


Fig. 11.26 Illustration of wastewater delivery and distribution using spray nozzles on pressurized distribution piping within a chamber-equipped trench

11.97

- Pressurized delivery can also be used for dosing and distribution to zones within a larger STU
  - Figure 11.27 illustrates the use of distributor valves
    * In this example, distributor valves are used to distribute flow between 6 zones of a STU by directing each sequential dose to 1 of the 6 zones
    * Each zone uses pressurized delivery and distribution within it

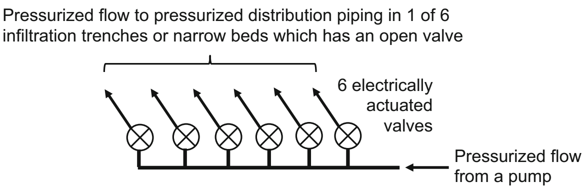


Fig. 11.27 Pressurized delivery and uniform distribution sequentially to different trenches or beds can be accomplished with electric or hydraulically actuated distributor valves

11.98

- Figure 11.28 illustrates the use of a pressurized hydrosplitter
  - Hydraulic design to distribute flow during a dose between 6 zones of a STU with gravity delivery and distribution in each of the 6 zones

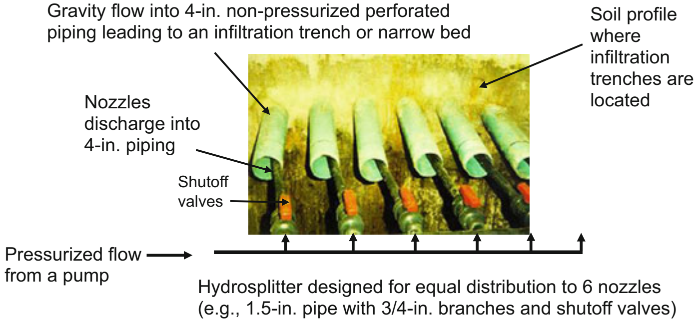


Fig. 11.28 Use of a hydrosplitter to achieve pressurized delivery of a dose into gravity flow distribution pipes connected to different trenches or narrow beds

11.99

- Design of dosing and pressure distribution networks
  - The basic design process is the same as that used for porous media biofilters and includes several steps (refer to Chap. 8 for design details)
    - Layout the STU, considering use of multiple zones
    - Layout the delivery and distribution network(s)
    - Choose an orifice size and calculate the discharge rate
    - Check the uniformity of distribution (orifice flow rates should vary by $<10\,\%$ of each other and the dose volume should be $>5\times$ the volume to fill the distribution piping)
    - Calculate the total dynamic head during a dosing event
    - Select a pump or siphon to deliver the dosing flow rate against the TDH calculated
    - Select a dosing approach—demand based or timer based
    - Calculate the dose volumes and timer settings if a timer is used

11.100

- D&I considerations—Resting and cyclic application
  - Resting cycles on the order of 1 year or more can help rejuvenate the infiltrability of a soil infiltrative surface
    - One option is to have two parallel units each equivalent to 75 % of the size required and alternate operation between the 2 units
    - Another option is to use dosing and sequential application to apply a cyclic higher HLR_D to one or more parts of a system
      - For example, a 4-trench network with a 4-year cycle of 1 year on-line and 3 years off-line and resting is shown in Fig. 11.29

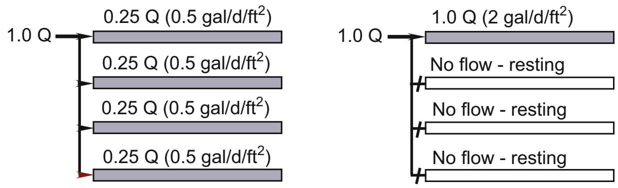


Fig. 11.29 Uniform distribution between four trenches (*left*) versus cyclic overloading to one trench while the other three are rested (*right*)

11.101

- D&I considerations—Design for long-term service
  - Options for design for a sustainable service life (e.g., 20 year+)
    - Rejuvenation of infiltrability if soil clogging becomes excessive
      - Apply higher effluent quality
      - Use physical or chemical rehabilitation methods
      - Enable long-term resting (e.g., 1 year or more)
    - Plan for installation of a new STU if needed
      - Reconstruction (e.g., new trenches between the old ones)
      - Reserve an area for a new STU (Fig. 11.30)—Resting of the original STU may restore capacity and enable use of the old along with the new unit

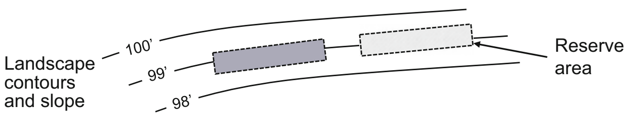


Fig. 11.30 Illustration of a reserve area for a new STU if it is needed in the future

11.102

- D&I considerations—Installation at the site
  - Careful installation and startup are critical to long-term performance of a STU
  - Construction practices must prevent damage to the soil and loss of infiltration capacity during installation
  - Recommended practices include:
    - Do not drive, or even walk, on the soil infiltrative surface
    - Do not dump or place gravel or other solid objects on the infiltrative surface
    - Avoid construction in fine-grained soils during wet periods
    - Complete the installation quickly and minimize the time the soil infiltrative surface is exposed
  - Figure 11.31 present photographs illustrating the installation of chamber-equipped soil treatment units for subsurface soil infiltration

11.103

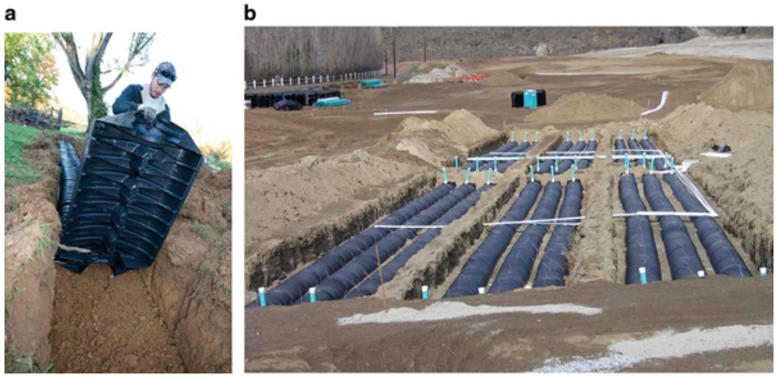


Fig. 11.31 Photographs illustrating (**a**) a chamber-equipped trench being placed without walking on the infiltrative surface and (**b**) a large system serving a clustered development established in a larger bed excavation but with unused separation between adjacent infiltration units (Photographs courtesy of Infiltrator® Water Technologies)

11.104

- Startup activities and events
  - Flush lines to remove debris resulting from construction
  - Verify that all piping connections are solid
  - Verify functioning of pumps, distributor valves, etc.
  - Record and examine initial readings (e.g., for units with pumps and controls)
  - Initiate application of treatment unit effluent to the STU
- It is generally advisable to avoid construction and startup during 'harsh' weather and climatic conditions (e.g., cold winter months)

11.105

- D&I considerations—Operation and maintenance
  - STUs can be designed to have limited O&M
  - Potential routine O&M requirements
    - Inspect and maintain all upstream treatment units (e.g., with a septic tank, clean effluent screens and pump septage)
    - Inspect and record conditions related to infiltration
      - Landscape inspection—inspect the ground surface above the STU to ensure no seepage is occurring—if seepage is present, consider corrective actions
      - Observe the magnitude of any ponding (*Note*: ponding is not necessarily an indicator of poor performance)—if excessive, reduce flow and/or plan for rehabilitation
      - Verify that all portions of the STU that are supposed to be operational are receiving flow—adjust as needed
      - Wastewater HLR_D and OLR—if outside design limits consider flow reduction and/or higher treatment

11.106

- D&I considerations—Monitoring and controls
  - Monitoring requirements depend on the application, but all systems should have:
    - A method to reliably measure and record daily flow (e.g., indoor water meter, dosing counter) and flow to each zone of a STU
    - A means for inspection (and maintenance) of the infiltrative surface (e.g., observation ports)
  - What may or may not be required and/or feasible
    - Monitoring of the wastewater to be treated in the STU is costly if done properly, and it is normally not needed, except for:
      - STUs serving commercial or institutional buildings or larger developments
    - Sampling and analysis of soil and groundwater is difficult and costly, and should only be considered for special cases, e.g.:
      - Larger systems (e.g., $\geq$ 25,000 gal/day), particularly those in sensitive environmental areas

11.107

- Other sensors and alarms can be included if deemed important for monitoring and process control purposes, e.g.:
  - Water level sensor for a dosing basin to detect and provide an alert if there is a pump or siphon problem
  - Level sensors to detect and measure the depth of ponding in a trench or bed
    - Could be used to turn off one portion of a STU and direct flow to another
  - Subsurface sensors for measuring soil water content and temperature below a soil infiltrative surface
  - Telemetry options for data acquisition and alarm communication

11.108

- Overcoming site limitations and use of design variants
  - Some sites are unsuitable for a normal STU application due to constraints related to:
    - Soil permeability being too high or too low, e.g.:
      * Permeability is too high and inadequate treatment may result
      * Permeability is too low and there is also susceptibility to construction damage
    - Inadequate soil depth to a limiting condition, e.g.:
      * Shallow soil over creviced or porous bedrock
      * Groundwater table is located near the ground surface
    - Landscape features e.g.:
      * Inadequate land area and insufficient setback distances
    - Site assimilative capacity, e.g.:
      * Inadequate hydrologic conditions for groundwater recharge without excessive water table mounding

11.109

- Design approaches can overcome some limitations (Table 11.18)

Table 11.18 Examples of system design approaches that can be used to overcome site conditions that prevent use of a normal STU

Limitation	Example design approach to overcome the limiting condition	Basis for overcoming the limitation
Permeability or soil depth constraints	Advanced treatment prior to soil infiltration	High quality effluents applied to the native soil profile can reduce soil clogging and reduce the treatment required within the soil profile
	Use of a sand lining in the infiltration trenches	
	Use of an at-grade or mounded STU laid out with low LLRs and dosed using timed-dosing	
High ground-water tables	Advanced treatment prior to a typical STU or use of a sand-lined STU	Provide high treatment prior to effluent release to the native soil profile to reduce treatment required in the soil profile
	Use of an at-grade or mounded STU laid out with low LLRs and dosed using timed-dosing	
	Water tables can be lowered using interceptor or curtain drains	Increase the depth of unsaturated soil and improve soil treatment
Inadequate suitable area	Advanced treatment prior to soil infiltration	Enable higher HLR_D and smaller infiltration areas; reduced setbacks

11.110

- Basic features of at-grade and mounded STUs
  - Infiltration units are constructed within fill sand placed on the landscape surface (Fig. 11.32)
  - Infiltration surface is placed in a chisel-plowed landscape surface or in a sand layer placed on the plowed soil surface
  - System performance can be equal or better than a typical STU (i.e., installed below the original ground surface) but costs are normally much higher

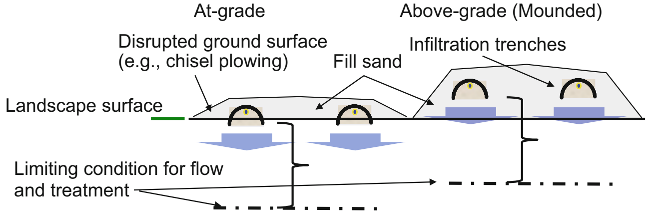


Fig. 11.32 Illustration of an at-grade or mounded STU

11.111

- Mound system design and application guidance
  - Considerations include those that are generally applicable to STUs as well as those that are unique to an at- or above-grade unit
  - Site conditions that are generally suitable for a mound system
    - * Depth below the landscape surface to a limiting condition
      - – High water table $\geq$ 10 in.
      - – Bedrock

        Creviced bedrock $\geq$ 2 ft.
        Non-creviced bedrock $\geq$ 1 ft.
    - * Soil permeability
      - – For the top 10 in., "moderately low" or better (e.g., percolation rate of 60–120 min/in)
    - * Landscape slope
      - – Up to 25 %

11.112

- Key design features
  - Sand fill
    - The sand fill is critical to system performance
    - Clean coarse sands are used for the mound fill
      $\leq$20 % coarse $>$2.0 mm and $<$5 % finer than 0.053 mm
  - Wastewater application rates
    - Based on wastewater composition and soil conditions
      Application rate to the sand fill (e.g., 1 gal/day/ft^2)
      Rate for the basal area native soil (e.g., 0.25 gal/day/ft^2)
  - Wastewater delivery and distribution
    - Intermittent dosing with pressurized distribution is normally required

- Figure 11.33 shows a photograph of the landscape where a mounded STU has been installed

11.113

The STU is placed in sand fill used to make the mound above the original landscape surface

Fig. 11.33 Photograph of a STU within an above-grade mound established using filter sand with certain specifications

11.114

- D&I considerations—Modeling tools
  - Modeling tools are available that can help with STU design and performance assessment
  - Examples of several modeling tools include:
    - Analytical and numerical models of flow and transport for a single system that can be used to evaluate:
      - Infiltrability loss during operation (Siegrist and Boyle 1987)
      - The potential for groundwater mounding under larger systems (Poeter et al. 2005a, b)
      - The fate of nitrogen (STUMOD) (Geza et al. 2014)
      - Complex flow and transport (Hydrus-2D) (Geza et al. 2014)
    - Watershed-scale models that can be used to evaluate:
      - The relative effects of a large number of decentralized systems on water quality within a watershed (WARMF) (Siegrist et al. 2005)

11.115

11-5. Summary

- Land-based wastewater systems have been used for more than 100 year, initially for simple short-term waste disposal but later for longer-term treatment and disposal
- Today, soil treatment units can be designed to achieve:
  - Tertiary treatment of primary or secondary effluents,
  - Natural disinfection during subsurface transport, and
  - Groundwater recharge of the reclaimed water
- Design and implementation requires integrated consideration of treatment goals, environmental conditions, and key factors affecting hydraulic and treatment processes in a soil profile
- Soil treatment units can provide a long service life with limited power and O&M requirements

11.116

11-6. Example Problems

- 11EP.1. Assessing site conditions and soil properties
  - Given information
    - The site of interest is located northwest of Golden, Colorado where a developer wants to build a new condominium building (Fig. 11EP.1)
  - Determine
    - Complete a preliminary assessment of soil and site conditions and the likely suitability for a STU

Fig. 11EP.1 Aerial view of the location of a proposed development northwest of Golden

11.117

- Solution
  - Preliminary assessment using Web Soil Survey (WSS)
    * Go to WSS URL: http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm and choose "Start WSS" green button
    * Under tab for "Area of Interest (AOI)", choose "Navigate By. . ." and enter state and county; select "View" button
    * With AOI interactive map, use icons for zooming and map movement to focus in on site of interest; note if you zoom in too far and reach the scale limit, you can zoom out
    * Choose the AOI icon (rectangle or polygon) and draw AOI boundary on map image around site of interest
    * Select tab for "Soil Map" to review soil map units in AOI
    * Under "Map Unit Legend", select "Map Unit Name" to view description and properties
    * Under tab for "Soil Data Explorer" choose "Sanitary Facilities" and "Septic Tank Absorption Fields" and "View Ratings"

11.118

- Location of development site as area of interest on WSS appears in Figs. 11EP.2 and 11EP.3

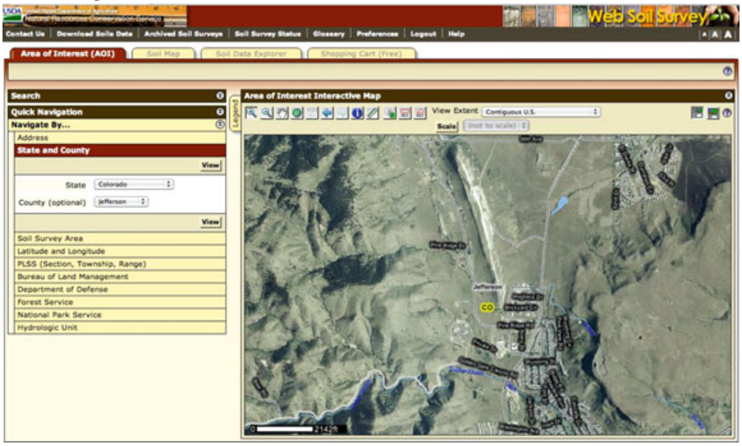


Fig. 11EP.2 Web page from Web Soil Survey that shows the location of a proposed development site northwest of Golden (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm)

11.119

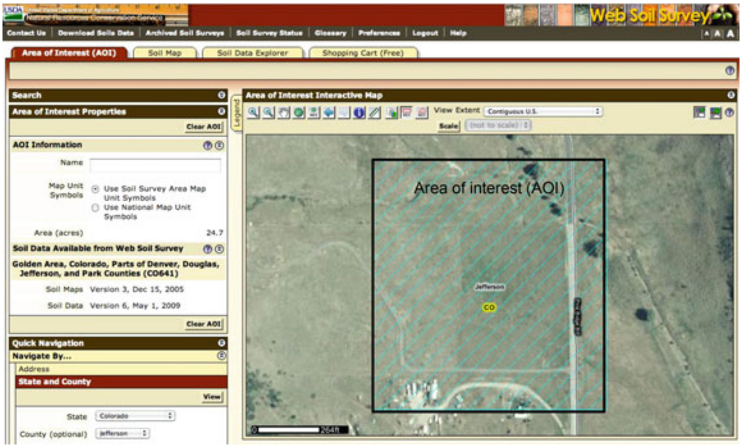


Fig. 11EP.3 Web page from Web Soil Survey that shows the area of interest (AOI) for the proposed development site northwest of Golden (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm)

11.120

- Soil mapping within the AOI is illustrated in Fig. 11EP.4

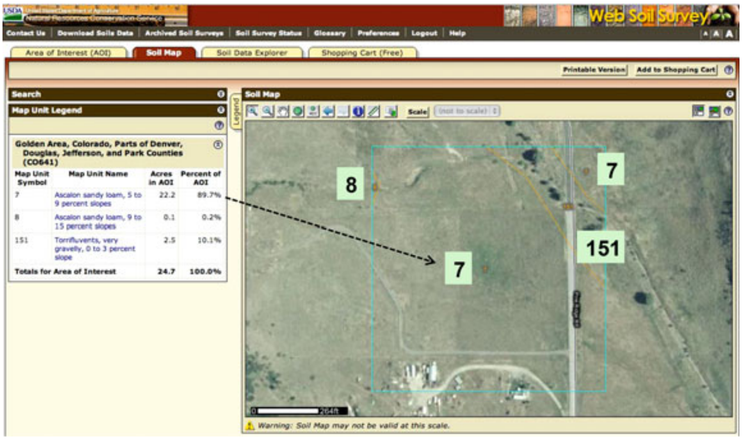


Fig. 11EP.4 Web page from Web Soil Survey that shows the soil series within the AOI for a proposed development site (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm)

11.121

- Soil properties for Map Unit 7 are shown in Fig. 11EP.5

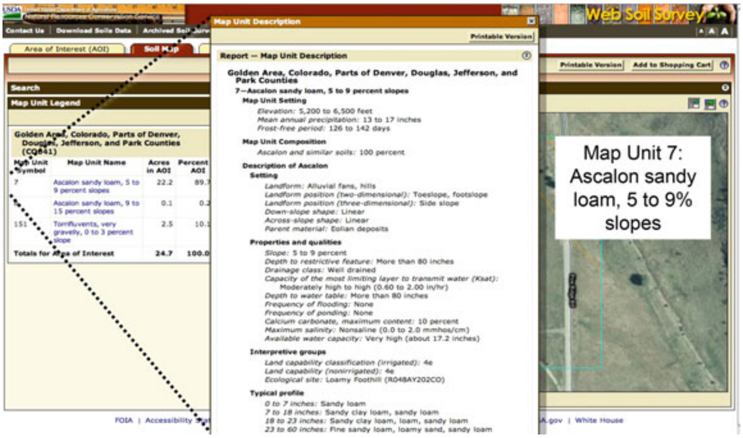


Fig. 11EP.5 Web page from Web Soil Survey that shows the soil properties for Map Unit 7 which occurs within the AOI for a proposed development site (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm)

11.122

○ Suitability ratings for the AOI are shown in Fig. 11EP.6

Fig. 11EP.6 Web page from Web Soil Survey that shows Map Unit 7 which occurs within the AOI for a proposed development site is not limited for soil-based effluent treatment (http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm)

11.123

○ Assessment of site suitability
- * Site appears suitable based on WSS reported conditions:
  - – Profile: Ascalon sandy loam
  - – Land slope: 5–9 %
  - – Depth to restrictive zone: $>$ 80 in
  - – Drainage class: Well drained
  - – K_S of most limiting layer:
    Moderately high to high (0.6–2.0 in./h)
  - – Depth to water table: $>$ 80 in
  - – Frequency of flooding: none
  - – Frequency of ponding: none
- * Site appears suitable, but a site investigation is needed
  - – Soil borings, backhoe test pits, and possibly hydraulic tests to verify conditions
  - – Assessment of site assimilative capacity might be warranted (e.g., groundwater mounding) depending on system size and design features

11.124

- 11EP.2. Design of a subsurface soil treatment unit
  - Given information
    - Site is located outside Denver, Colorado and a site evaluation revealed suitable site conditions with no limiting conditions observed during borings and testpits to 7 ft below ground surface
    - Convenience store with a design $Q_D = 2000$ gal/day
    - Treatment before the STU is a recirculating sand filter (RSF) (RSF effluent = 20 mg/L BOD_5, 20 mg/L TSS and 50 % N removal)
    - A network of trenches outfitted with chambers will be used and RSF effluent distribution will be by pressurized dosing
    - Treatment goal: remove BOD_5 and TSS as typical for a STU

11.125

- Determine
  - HLR_D and total infiltrative surface (IS) area required (ft^2), number of trenches needed and layout, and the total footprint area required (ft^2)
- Solution
  - Select a HLR_D based on LTARs in Colorado Regulation 43
    - Soil profile conditions (based on a site evaluation)
      - 0–30 in.: fine sandy loam (strong granular structure)
      - 30–60 in.: sandy loam (moderate granular structure)
      - 60–84 in.: medium to coarse sand (single grain)
    - LTAR in Colorado Reg. 43 (see Tables 11EP.1 and 11EP.2)
      - Soil Type 2 (assume placement of IS at 24 in. depth)
      - Treatment Level = 2 N (RSF with 50 % N removal)
      - Prescribed LTAR = 0.90 gal/day/ft^2

11.126

- Classification of the effluent to be applied to the STU
  - Based on typical RSF performance the treatment level = TL2N (Table 11EP.1)

Table 11EP.1 Treatment levels prescribed in Colorado Regulation 43 (CDPHE 2013)

Treatment level	$cBOD_5$[a] (mg/L)	TSS (mg/L)	Total nitrogen (mg/L)
TL 1[b]	145	80	60–80 mg/L
TL 2	25	30	60–80 mg/L
TL 2 N	25	30	>50 % reduction[c]
TL 3	10	10	40–60 mg/L
TL 3 N	10	10	20 mg/L

[a]$cBOD_5$ can be estimated as 0.85 × total BOD_5.
[b]Values for TL 1 are typical but design must account for site-specific information.
[c]NSF/ANSI Standard 245—Wastewater Treatment Systems—Nitrogen Reduction requires reduction of 50 % rather than achieving a specific value.
Source: Table 6.3. Colorado Reg. 43. June 2013.

11.127

- Determining the LTAR for sizing the STU
  - LTAR = 0.90 gal/day/ft^2 as shown in Table 11EP.2

Table 11EP.2 The LTAR prescribed for system sizing is based on the treatment level and soil type in Colorado Regulation 43 (CDPHE 2013)

Soil Type, Texture, Structure and Percolation Rate Range					Long-term Acceptance Rate (LTAR); Gallons per day per square foot				
Soil type	USDA soil texture	USDA soil structure-shape	USDA soil structure-grade	Percolation rate (MPI)	Treatment Level 1[2]	Treatment Level 2[2]	Treatment Level 2N[2]	Treatment Level 3[2]	Treatment Level 3N[2,3]
0	Soil Type 1 with more than 35% Rock (>2mm); Soil Types 2-5 with more than 50% rock (>2mm)	-	0	<5	Min. 3-ft. deep unlined sand filter required	Minimum 2-foot deep unlined sand filter required[2]			
1	Sand, Loamy Sand	-	0	5-15	0.80	1.25	1.25	1.40	1.40
2	Sandy Loam, Loam, Silt Loam	PR (prismatic) BK (blocky) GR (granular)	2 (Moderate) 3 (Strong)	16-25	0.60	0.90	0.90	1.00	1.00
2A	Sandy Loam, Loam,	PR, BK, GR	1 (Weak)	26-40	0.50	0.70	0.70	0.80	0.80

Source: Excerpt from Table 10-1. Colorado Reg. 43. June 2013.

11.128

- Infiltration depth and area required
  - The soil infiltrative surface will be placed at 24 in. depth bgs
    - $HLR_D = LTAR = 0.90$ gal/day/ft^2
    - Adjustments for architecture and pressurized dosing
      Sizing adjustment factor = 0.8 (Table 11EP.3)

$$A_{IS} = \left(\frac{Q_D}{HLR_D}\right)(AF) = \left(\frac{2000\,\text{gal/day}}{0.90\,\frac{\text{gal/day}}{\text{ft}^2}}\right)(0.8) \tag{11.9}$$

$$A_{IS} = \left(2222\,\text{ft}^2\right)(0.8) = 1778\,\text{ft}^2$$

11.129

Table 11EP.3 System sizing is adjusted based on the geometry of the STU and the method of effluent application in Colorado Regulation 43 (CDPHE 2013)

Type of Soil Treatment Area	Method of Effluent Application to Soil Treatment Area					
	Gravity		Dosed (Siphon or Pump)		Pressure Dosed	
	AF	EF	AF	EF	AF	EF
Trench	1.0	1.0	0.9	1.11	0.8	1.25
Bed	1.2	0.83	1.1	0.91	1.0	1.0

Source: Table 10-2. Colorado Reg. 43. June 2013.

11.130

- Check the organic loading rate

$$\mathrm{OLR} = \frac{(\mathrm{Q_D})(\mathrm{BOD_5})(\mathrm{F})}{\mathrm{A'_{IS}}} \tag{11.10}$$

$$\mathrm{OLR} = \frac{(2000\,\mathrm{gal\,day^{-1}})(20\,\mathrm{mg\,L^{-1}})(8.34 \times 10^{-6})}{\sim 1778\,\mathrm{ft^2}}$$

$$\mathrm{OLR} = 0.00019\ \mathrm{lb\text{-}BOD_5}\,/\,\mathrm{day\,per\,ft^2}$$

✓ Okay, since OLR $\ll$ 0.001 lb-BOD_5/day per ft^2, which is a suggested limit for Class I soil (see Table 11.16)

11.131

- Number of trenches required
  - Select trench width (narrow preferred) and length (based on footprint area available and to keep length reasonable to aid delivery and distribution (e.g., < 100 ft long))
  - Choose 2-ft wide trenches with a length of 75-ft.

$$\mathrm{L_T} = \left(\frac{\mathrm{A_{IS}}}{\mathrm{W_I}}\right) = \left(\frac{1778\ \ \mathrm{ft^2}}{2\ \ \mathrm{ft}}\right) = 889\ \ \mathrm{ft} \tag{11.11}$$

$$\mathrm{n} = \left(\frac{\mathrm{L_T}}{\mathrm{L_I}}\right) = \left(\frac{889\,\mathrm{ft}}{75\ \ \mathrm{ft}}\right) = 11.9 \quad \therefore \mathrm{chose}\ \ 12 \tag{11.12}$$

- Soil infiltrative surface area provided in the layout chosen

$$\mathrm{A'_{IS}} = \mathrm{n(L_I \times W_I)} = 12(75\,\mathrm{ft} \times 2\,\mathrm{ft}) = 1800\,\mathrm{ft^2} \tag{11.13}$$

11.132

- System layout
  - * Plan view of a 12-trench layout is shown in Fig. 11EP.7

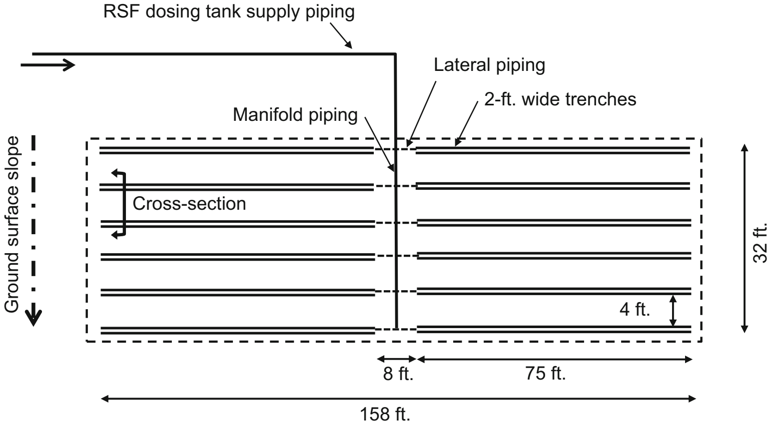


Fig. 11EP.7 Plan view of a layout for the STU

11.133

- * Cross-section of a set of trenches is shown in Fig. 11EP.8

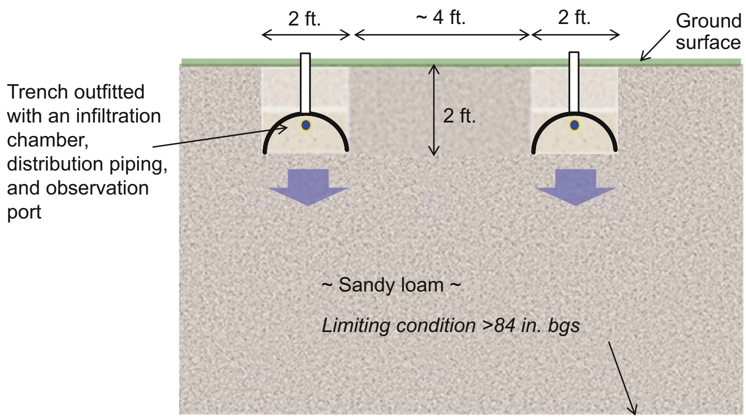


Fig. 11EP.8 Cross-section view of a set of trenches within the layout for the STU

11.134

- Footprint area required
  - Separation between trenches is needed to (1) enable construction and (2) to provide sidewall and undisturbed soil between trenches for oxygen diffusion to help maintain aerobic conditions in the soil profile within the STU
  - With 4-ft separation between 12, 2-ft wide trenches that are each 75-ft long on each side of a center manifold (2 groupings of 6 trenches each separated by 8 ft.) the footprint area can be calculated using Eq. 11.14

$$\begin{aligned} A_F &= (y)[(L_I)(W_I)(n) + (L_I)(W_U)(n-1)]+ \\ &\quad (y-1)\left[(L_U)(W)_I(n) + (L_U)(W_U)(n-1)\right] \end{aligned} \tag{11.14}$$

$$\begin{aligned} A_F &= (2)[(75)(2)(6) + (75)(4)(6-1)]+ \\ &\quad (2-1)[(8)(2)(6) + (8)(4)(6-1)] \\ A_F &= (2)[900 + 1,500] + 256 = 5056\,\text{ft}^2 \end{aligned}$$

11.135

- Influent (RSF effluent) delivery and distribution
  - Dosing frequency and volume
    - For a sandy loam soil: select 4 doses per day

      Check to be sure this no. is okay per the distribution network design and requirement that the dose volume is $>5\times$ the pipe fill volume
    - For 4 doses per day and $Q_D = 2000$ gal/day, dose volume under design conditions = 500 gal/day
  - Dosing tank size
    - Volume for HRT = 1 day and F = 0.8

$$\begin{aligned} V_{DT} &= (F)(HRT)(Q_D) \\ V_{DT} &= (0.8)(1\,\text{day})(2000\,\text{gal}/\text{day}) \\ V_{DT} &= 1600\ \text{gal} \end{aligned} \tag{8.13}$$

11.136

- Distribution within the trench network
  - Design of a pressurized network for a STU follows a process that is essentially the same as that used for pressure distribution to PMBs (See Chap. 8)
  - However, STUs are larger than PMBs and hydraulic design is important to avoid large pumps or siphons; e.g.:
    - For a 75-ft long lateral with 1/8-in. diam. orifices on 2-ft spacing, each trench has a lateral with 38 orifices
    - Orifice Q at $h_r = 4$ ft is 0.39 gal/min so lateral Q = 14.8 gal/min
    - Total network Q = (12)(14.8 gal/min) = 178 gal/min
    - To reduce pumping Q during a dose, consider dividing the STU into zones and using distributor valves, e.g.:

      4 valves for 4 zones with 3 trenches in each zone
      Pumping Q = (3)(14.8 gal/min) = 44.4 gal/min

11.137

- 11EP.3. Assessment of N removal in a soil treatment unit
  - Given information
    - Wastewater source is from an apartment complex with a design $Q_D = 1500$ gal/day
    - Treatment before the STU is a septic tank with effluent screen anticipated to yield STE with $NH_4^+ = 60$ mg-N/L and $NO_3^- = 1$ mg-N/L
    - A site evaluation revealed a sandy loam soil profile with no limiting conditions observed during borings and testpits to 8 ft below ground surface and a soil temperature = 15 °C
    - The STU will have a design $HLR_D = 0.75$ gal/day/ft^2 applied to chamber-equipped trenches and STE application will be by timed dosing and pressure distribution
    - The treatment goal is to remove BOD_5 and TSS consistent with expectations for a typical STU and also reduce the Total N to 20 mg-N/L by 3-ft depth below the soil infiltrative surface

11.138

- Determine
  - N concentration and mass flux at 3-ft depth below the soil infiltrative surface
- Solution
  - Assessment of N removal using STUMOD[e]
    - * Within Excel, open and start STUMOD (Fig. 11EP.9)
    - * Input values for each parameter (Figs. 11EP.10 and 11EP.11) and run STUMOD
    - * The STUMOD simulation results for concentration and mass flux are shown in Figs. 11EP.12 and 11EP.13
  - The Total N in soil pore water at 3 ft depth is 18 mg-N/L, which meets the 20 mg-N/L goal

[e]Soil Treatment Unit Model (STUMOD) was developed by Dr. Mengistu Geza (Geza et al. 2009, 2013) and the solution to this problem was provided by Dr. Geza in November 2015 (mgezanis@mines.edu).

11.139

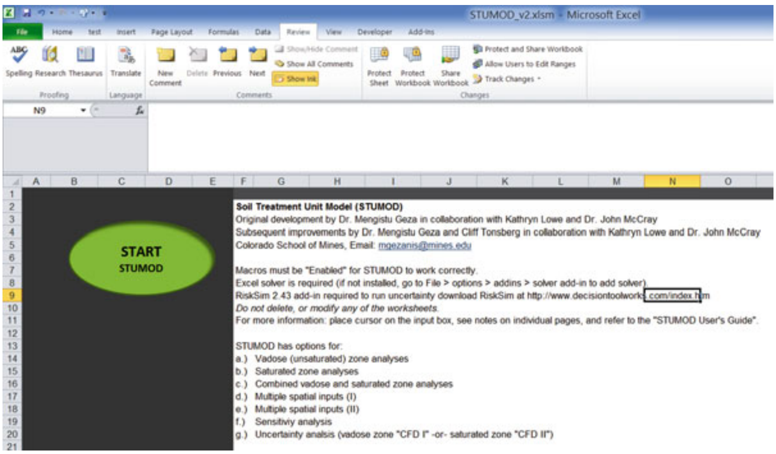


Fig. 11EP.9 Image of the opening screen of STUMOD

11.140

Click on the boxes shown on the right to input soil parameters, effluent concentrations, Hydraulic loading rate, soil temperature, and treatment depth.

The input boxes for each parameter are displayed as shown in Fig. 11EP.11.

Required input values:

Soil type = Sandy loam

$HLR_D = 3$ cm/d (0.75 gal/d/ft^2

Co-NH_4 = 60 mg-N/L

Co-NO_3 = 1 mg-N/L

T = 15C

Depth = 90 cm (3 ft.)

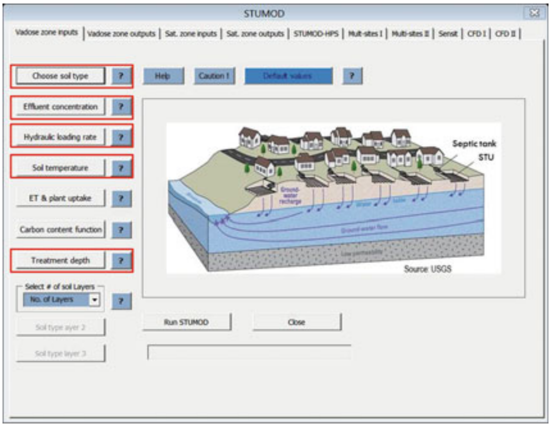


Fig. 11EP.10 Image of the opening input screen for STUMOD

11.141

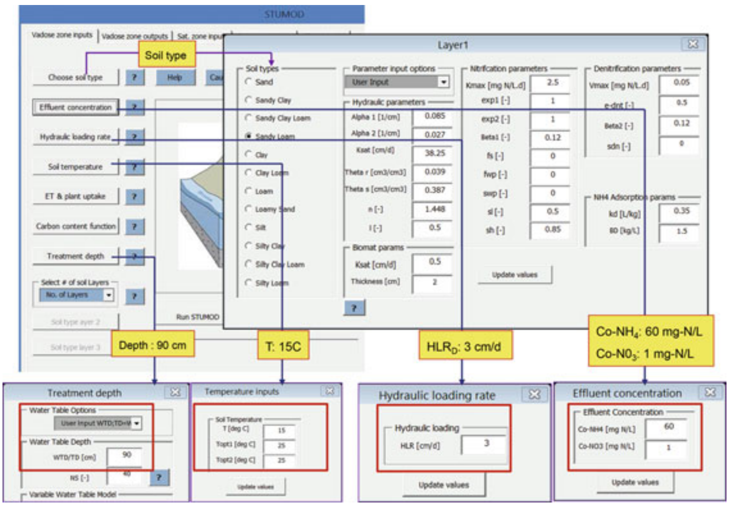


Fig. 11EP.11 Image of the screens for inputting parameter values for STUMOD

11.142

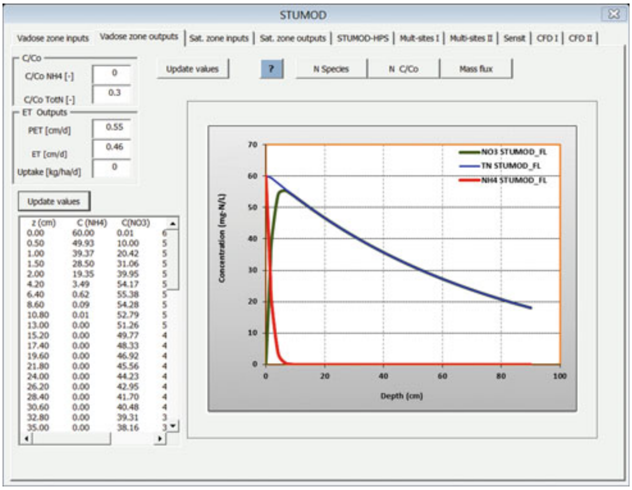


Fig. 11EP.12 Nitrogen concentrations versus depth below the soil infiltrative surface as simulated with STUMOD

11.143

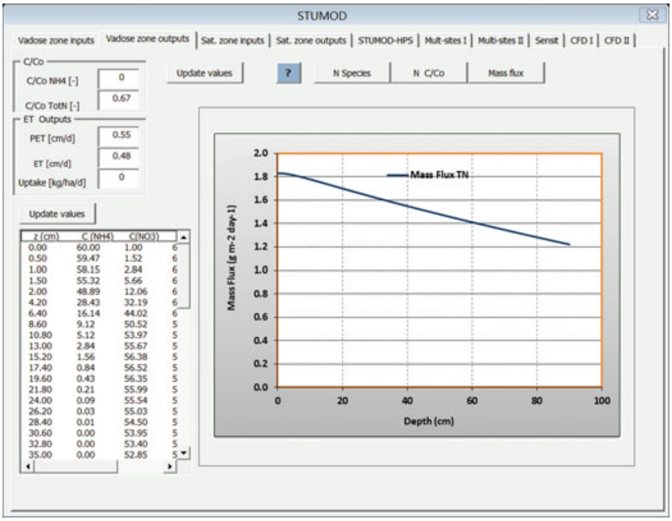


Fig. 11EP.13 Nitrogen mass flux versus depth below the soil infiltrative surface as simulated with STUMOD

11.144

Chapter 12

Treatment Using Landscape Drip Dispersal

12-1. Scope

Drip dispersal of wastewater into shallow soil beneath the landscape surface is an adaptation of drip irrigation, a high efficiency watering technique used to support plant growth. Landscape drip dispersal can be used for wastewater treatment while also achieving beneficial recovery of water and nutrients for irrigation purposes. This chapter presents the principles and processes involved in drip dispersal and describes design and implementation of a landscape drip dispersal unit within a decentralized system.

12-2. Key Concepts

- Drip dispersal can be used for wastewater treatment to achieve the combined goals of tertiary treatment with natural disinfection, reclaimed water assimilation, and beneficial recovery of water and nutrients.
- Landscape drip dispersal units (LDUs) used in decentralized systems often receive treated secondary effluent (e.g., aerobic treatment unit or porous media biofilter effluent) as the influent though primary effluents have also been dispersed in a LDU. Dispersal occurs just below the ground surface within shallow unsaturated and aerobic soil. Dispersal typically occurs within a layer of soil where plant root networks are, or can become, well established.
- For drip dispersal, a network of small-diameter pressurized tubing with drip emitters is buried in shallow soil just below the ground surface (typ. 6–12 in.). Wastewater is intermittently dosed into the network (typ. 3–5 times daily on average) and spurts from the array of drip emitters and is

R.L. Siegrist, *Decentralized Water Reclamation Engineering*,
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absorbed into the soil. Uniform distribution throughout the network can be assured by using drip emitters, which are pressure compensating and discharge a consistent rate (e.g., 0.60 gal/h) at line pressures ranging from 5 to 60 psi. Due to the high operating pressures, a high-head pump is normally required to pressurize the drip tubing network.

- Similar to the function of a soil treatment unit (STU), during landscape drip dispersal hydraulic and purification processes control wastewater movement and renovation in the soil environment. However, in the shallow subsurface, there can be enhanced processes due to higher soil porosity, organic matter content, and microorganism levels. Enhanced renovation of wastewater can also occur due to the presence of a rhizosphere, which is the zone around the roots of plants where complex relations exist among the plants, soil microbes, and the soil itself.
- Key factors that control wastewater movement and renovation in a LDU include: soil profile conditions, climate and hydrologic conditions, site conditions and installation practices, LDU operation and composition of the wastewater dispersed, and plant type and density. The hydraulic loading rate for design (HLR_D) for sizing the area required for a LDU can be selected based on soil type and wastewater composition or water and nitrogen mass balances. The HLR_D is applied to the 'footprint' of the network, which includes drip tubing often installed with emitters every 2 ft in a line and with a 2-ft separation between parallel lines of tubing. As a result, the effective HLR_D to the soil actually in contact with the emitter and along the tubing is nearly $10\times$ higher than the rate used for sizing the horizontal soil infiltrative surface area required for a STU trench or narrow bed.
- Treatment processes during landscape drip dispersal are similar to those in a STU and include physical (e.g., filtration), chemical (e.g., sorption), and biological (e.g., aerobic biotransformation) processes. In addition, plant-based processes can play a role (e.g., evapotranspiration, nutrient uptake). Under a network of drip dispersal tubing, in the soil pore water by 2-ft depth, it is reasonable for BOD_5 and TSS to be 5–10 mg/L, total N at or approaching 10 mg/L, total $P < 1$ mg/L, and pathogen removals that are > 99.99 %. Any percolate migrating beyond this depth and reaching groundwater under the site can be further attenuated and should be comparable to tertiary effluent with natural disinfection.
- Design of a LDU requires integrated analysis of the site conditions and soil properties, the wastewater source and options for treatment prior to dispersal into the soil, along with the treatment goals and method of assessment.

- Landscape drip dispersal can be designed to disperse primary or secondary effluents. With primary effluents (e.g., septic tank effluent) due to the higher concentrations of BOD_5, TSS, and NH_4^+, there is a greater risk of clogging including the tubing and emitters as well as the soil infiltrative surface area. To help minimize emitter clogging, a filtration device (e.g., 100 μm disk filter) is normally included in the pump discharge pipeline prior to connection to the drip tubing. In addition, automatic flushing of the drip tubing lines occurs periodically (e.g., about once every 20 dispersal cycles).
- A LDU needs to be properly located at a site where landscape features are appropriate and where soil and site conditions are suitable for the size and type of dispersal area to be established. A suitable site for a LDU should have a native soil profile that has adequate permeability, adequate depth to a limiting condition (e.g., to groundwater or bedrock), and conditions suitable for aerobic biological treatment.
- Compared to dispersal of primary effluents (e.g., septic tank effluent) site requirements for higher quality effluents (e.g., packaged biofilter or membrane bioreactor) can be relaxed (e.g., less unsaturated soil depth) and hydraulic loading rates can be increased.

■ For a particular site, design of a LDU involves a series of engineering steps including choices concerning: the type of treatment prior to discharge into the drip dispersal tubing; the wastewater application rate for footprint area sizing; the type of drip tubing and emitters to be used; the depth of installation of the drip tubing below ground surface; the layout and landscape placement of one or more dispersal zones; the number of dispersal events and time of occurrence; and the method of installation of the drip tubing. Engineering design of a LDU is an interactive and iterative process since design choices made in one step may affect the choices made in another.

■ A LDU needs to be properly sited in the landscape and carefully established to avoid construction damage. Drip dispersal requires electrical and mechanical components (e.g., pumps, valves, controls), which require routine operation and maintenance. Operation and maintenance requirements related to clogging problems are often lower if secondary effluents are dispersed.

12-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 12 are presented in the Slides section.

12-4. Terminology

Terminology introduced and used in Chap. 12 is define below.

Clogging—In the context of a landscape drip dispersal unit, clogging refers to the accumulation of material that plugs up dispersal tubing or the drip emitters in it or blocks and fills soil pores in the soil the tubing is installed in. Clogging can cause hydraulic problems and necessitate operation and maintenance actions.

Dispersal event—As applied to landscape drip dispersal, refers to the delivery and distribution of wastewater into a network of drip tubing from which it exits through drip emitters and infiltrates into shallow soil below the ground surface.

Drip dispersal—Refers to a method of intermittently distributing wastewater into shallow soil just below the ground surface. Drip dispersal of wastewater effluent is an adaptation of drip irrigation for agronomic purposes.

Emitters—Generally refers to devices placed in tubing to control the rate of water flow out of the tubing into the space surrounding the tubing. Emitters can be pressure compensating or turbulent flow. Emitters are used in small diameter specialty tubing that is manufactured and used for irrigation, wastewater effluent distribution, and wastewater effluent drip dispersal into the shallow subsurface.

Evapotranspiration (**ET**)—The amount of water removed by evaporation and transpiration under the current soil moisture and environmental conditions.

Field flushing—Refers to a process done where flow through drip dispersal tubing is high enough to cause scouring of solids and this flushing flow is captured and returned to a treatment tank or basin.

Potential evapotranspiration (**PET**)—The maximum amount of water removed by evaporation and transpiration under the current environmental conditions if water supply to plants is unlimited.

Precipitation—(1) Refers to forms of water (e.g., rain, sleet, snow) that fall from the sky toward the land surface. (2) A chemical process that involves reactions where a dissolved substance is removed from solution by conversion to a solid substance that can be physically separated from the solution. An example of chemical precipitation involves the removal of phosphate from solution by addition of lime ($Ca(OH)_2$) to create a hydroxyapatite solid ($Ca_{10}(PO_4)_6(OH)_2$) that forms when lime is used to raise the $pH > 10$.

Pressure-compensating emitters—A drip emitter that is designed to function where the flow rate dispersed is largely constant across a wide range in pressure within the drip tubing.

Run—Refers to a length of drip dispersal tubing that leads away from, or returns to, a supply manifold.

Zone—Refers to a portion of a treatment unit to which influent is distributed during an individual dosing event. A treatment unit (e.g., porous media biofilter, soil treatment unit, etc.) can have a single zone or a number of zones. Use of multiple zones can help with delivery and distribution (e.g., by reducing the discharge flow rate required by a dosing pump).

Zone control valves—Refers to an electrically actuated valve that can be programmed to direct a dose of effluent to one or another zone of drip dispersal area based on a dosing schedule.

12-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 12 are listed below.

AMC	American Manufacturing Co.
bgs	Below ground surface
BOD	Biochemical oxygen demand
GW	Groundwater
HLR	Hydraulic loading rate
HLR_D	Hydraulic loading rate for design
ID	True inside diameter of a pipe
L	Length
LDU	Landscape drip dispersal unit
LTAR	Long-term acceptance rate
NOAA	National Oceanic and Atmospheric Administration
O&M	Operation and maintenance
PF	Peaking factor
STU	Soil treatment unit
TDH	Total dynamic head
TSS	Total suspended solids

A_F	Landscape dispersal area
A_{FM}	Minimum footprint area required for dispersal
C	Concentration or Hazen-Williams coefficient
C_E	Concentration in the effluent
C_I	Concentration in the influent
C_{PW}	Concentration in soil pore water
dh/dz	Hydraulic gradient
D_{PD}	Dispersal events per day per zone
d_u	Depth of unsaturated soil
ET	Evapotranspiration
I	Supplemental irrigation
K_S	Saturated hydraulic conductivity
LF	Lineal feet

LF_Z	Lineal feet of tubing in a zone
LLR	Linear loading rate
L_N	Number of laterals in a zone
N_D	Nitrogen removal by denitrification
N_{DE}	Efficiency of nitrogen removal by denitrification (same as D_n)
N_L	Nitrogen loading via deep percolation
N_{R+F}	Nitrogen added by rainfall and fixation
N_U	Net plant uptake and storage of nitrogen (same as U_P)
N_{WW}	Nitrogen applied in effluent dispersed
N_Z	Number of zones
Perc	Deep percolation
PET	Potential evapotranspiration
P_M	Mean monthly precipitation
P_{off}	Time a pump is off between dispersal events under average daily flow conditions
P_{on}	Time a pump is running during dispersal to a zone
Prec	Precipitation
psi	Pounds per square inch
PW	Soil pore water
Q_A	Average daily flow
Q_D	Design daily flow rate
Q_{Dis}	Flow rate during dispersal in a zone
Q_E	Discharge rate from an emitter
Q_{Flu}	Flow rate during flushing in a zone
Q_P	Flow rate a pump needs to be able to deliver against the TDH
Q_S	Flow rate required for scouring
S_{DM}	Standard deviation of monthly precipitation
S_E	Spacing between emitters along the tubing
S_T	Spacing between parallel tubing lines
T	Temperature
T_D	Portion of a day during which dispersal occurs
V_d	Volume of dilutive water
V_{DE}	Volume of a dispersal event to a zone
V_{DT}	Volume of a dosing tank

12-6. Problems

12.1. Drip dispersal of treated wastewater effluent is an adaption of what agricultural method?

12.2. Can drip dispersal networks be used as a means of uniform distribution in porous media biofilters or soil treatment units?

12.3. A landscape drip dispersal unit is used to deliver treated wastewater to the shallow subsurface within the root zone of growing turf and/or plants. Small diameter tubing with drip emitters spaced every 2 ft is commonly used. Answer the following questions regarding the hydraulic and purification processes important to wastewater treatment in a landscape drip dispersal performance.

(a) Which one of the following best describes the infiltrative surface area used for sizing a DDU: only the infiltrative surface at the emitter and along the tubing or the entire landscape area encompassed by the network of drip tubing.

(b) What two methods (device or unit or operational procedure) are normally used to help protect the emitters from clogging, even if the treated wastewater being dispersed is of high quality?

(c) Is the hydraulic and treatment performance of a landscape drip dispersal unit enhanced over that of a soil treatment unit? If yes, what is the basis for this enhancement?

12.4. Indicate which of the following statements best describes a landscape drip dispersal unit (LDU) when compared to an infiltration soil treatment unit (STU).

(a) Depth below ground surface:
$\text{LDU} = \text{STU}$:___
$\text{LDU} < \text{STU}$:___ $\text{LDU} > \text{STU}$:___

(b) Amount of excavation required to install:
$\text{LDU} = \text{STU}$:___
$\text{LDU} < \text{STU}$:___ $\text{LDU} > \text{STU}$:___

(c) In-lateral pressure needed during dosing:
$\text{LDU} = \text{STU}$:___
$\text{LDU} < \text{STU}$:___ $\text{LDU} > \text{STU}$:___

(d) Degree of plant uptake of nitrogen:
$\text{LDU} = \text{STU}$:___
$\text{LDU} < \text{STU}$:___ $\text{LDU} > \text{STU}$:___

12.5. In a landscape drip dispersal unit, name two methods (device or unit or operational procedure) that can be used to help keep the dispersal tubing clean and protect the emitters from clogging.

12.6. A landscape drip dispersal unit is being designed for a site with a sandy loam soil profile and no limiting conditions. The available area for the LDU is 7500 ft^2. Is this area large enough for a LDU if $Q_D = 4000$ gal/day for aerobic treatment unit effluent dispersal with a $\text{HLR}_D = 0.6$ gal/day/ft^2?

12.7. A landscape drip dispersal unit is being designed to treat a design daily flow of 4000 gal/day of septic tank effluent (STE) from a multifamily dwelling unit. The site has a sandy loam soil profile and the depth to a limiting condition is >80 in. The available footprint area on the lot is estimated to be about 12,000 ft^2. Is this area large enough to enable use of a LDU if the appropriate hydraulic loading rate for STE dispersal is judged to be 0.2 gal/day/ft^2?

12.8. A landscape drip dispersal unit has 3 zones, each of which has 4 laterals. Each lateral is 250-ft long and has pressure-compensating emitters spaced every 2 ft. If the emitter discharge rate is 0.6 gal/h, what is the total flow rate during dispersal only and during dispersal plus line flushing (in gal/min)?

12.9. A landscape drip dispersal unit has two layouts. Layout 1 has three zones with 4200-ft long laterals per zone. Layout 2 has three zones with 3300-ft long laterals per zone. Each lateral has pressure-compensating emitters spaced every 2 ft and the emitter discharge rate is 0.60 gal/h. What is the maximum flow rate the dosing pump will have to be able to deliver during operation (in gal/min)?

12.10. A drip dispersal unit (LDU) is being designed to treat a design daily flow of 1000 gal/day of sand filter effluent from a multifamily dwelling unit. The site has a sandy loam soil profile and the depth to a limiting condition is >80 in. What footprint area (in ft^2) is needed for a LDU if the long term acceptance rate for the soil is 0.4 gal/day/ft^2 for secondary effluent? What is the length of drip tubing needed if the spacing between adjacent drip tubing lines is 2 ft?

12.11. The footprint area required for a drip dispersal unit is being determined using a water balance to maximize the plant uptake and limit the deep percolation to the underlying ground water. Given the environmental data in the table below, what footprint area is needed (in ft^2) to treat a design flow, $Q_D = 8000$ gal/day? If the drip tubing has emitters every 2 ft along the tubing and the tubing is spaced 2 ft apart, what length of tubing is required (in ft)?

Month	Avg. Prec. (in./month)	ET (in./month)	Deep percolation (in./month)	HLR[a] (in./month)	Month	Avg. Prec. (in./month)	ET (in./month)	Deep percolation (in./month)	HLR[a] (in./month)
Jan	6.9	0.24	11.2	4.5	Jul	6.1	6.18	11.2	11.3
Feb	6.7	0.45	10.1	3.8	Aug	5.1	5.60	11.2	11.7
Mar	8.7	1.27	11.2	3.8	Sep	6.1	3.93	10.8	8.6
Apr	6.4	2.25	10.8	6.6	Oct	5.0	2.11	11.2	8.3
May	5.7	3.80	11.2	9.3	Nov	6.0	0.98	10.8	5.8
Jun	5.8	5.26	10.8	10.3	Dec	6.3	0.39	11.2	5.3
					Yearly	74.8	32.5	131.4	

[a]*Note*: 1 in./month = 0.020 gal/day/ft^2.

12.12. The hydraulic loading rate for sizing a landscape drip dispersal unit was determined using a water balance and found to be equal to 0.135 gal/day/ft^2 (based on site ET and precipitation data and an allowable deep percolation rate of 0.208 gal/day/ft^2). Complete a nutrient balance using the information below to determine the allowable concentration of total N in the effluent dispersed (C_I) if the concentration of total N in the percolating soil water below the drip dispersal footprint area (C_P) has to be <10 mg-N/L.

Given information and assumed values: HLR $=$ 0.135 gal/day/ft^2 and Perc $=$ 0.208 gal/day/ft^2. Nitrogen fluxes: $N_L = 276$ lb/acre-year, $N_{R+F} = 5$ lb/acre-year, $N_U = 200$ lb/acre-year. Denitrification rate: $N_D = 30\,\%$ of applied N.

12.13. For the Mines Park housing development you are tasked to do a preliminary design of a landscape drip dispersal unit to handle the design daily flow. Based on the information given below, answer the following design questions. (1) What hydraulic loading rate (HLR) would you use to size a LDU to disperse PBF effluent at this site? (2) Using the design flow rate what is the total footprint area (ft^2) required? (3) Determine the total linear feet of drip tubing required (ft) for the LDU. (4) Assume a LDU layout has four zones, each of which has three laterals that are 300 ft long. Given the total linear ft of tubing required (from 3), how many layouts would be needed? (5) Assuming that PBF effluent delivery to each zone is controlled and occurs sequentially, what is the total flow rate per zone (in gal/min) during dispersal only and during dispersal plus line flushing (in gal/min)? (6) What is the total landscape footprint area required for installation of this LDU (ft^2)? (7) How does the footprint area of the LDU compare to that required for the STU (as calculated in Problem 11.16)?

Given information and assumed values: The average daily flow, $Q_A = 28{,}425$ gal/d. The design flow, Q_D, is based on the recurring maximum daily flow (PF $=$ 2.0). A STEP collection system will convey septic tank effluent to the treatment site. The STE quality expected: pH $=$ 6, COD $=$ 220 mg/L, $BOD_5 = 160$ mg/L, TSS $=$ 80 mg/L, TKN $=$ 60 mg/L. At the location of the LDU, a textile media packaged biofilter (PBF) will be used to produce a high quality, aerobic effluent. The PBF quality expected: $cBOD_5 = 5$ mg/L, TSS $=$ 5 mg/L, TKN $=$ 5 mg-N/L. The site investigation revealed an area that was ~1000 ft long (parallel to landscape topographic contours) and 400 ft wide where there was sandy loam soil with moderate to strong structure and conditions were suitable for drip dispersal. The LDU will have drip dispersal tubing with emitters every 2 ft and with parallel lines of tubing placed 2 ft apart.

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[1]References cited in Chap. 12 are listed along with other references that have content relevant to the topics covered in Chap. 12.

Slides of Chapter 12

Decentralized Water Reclamation

Chapter 12: Treatment Using Landscape Drip Dispersal

Contents

12.1

12.1. Introduction

- Drip dispersal is an adaptation of drip irrigation
  - A landscape drip dispersal unit (LDU) is an alternative to a soil treatment unit that was described in Chap. 11 (Fig. 12.1)
  - A LDU can achieve tertiary treatment with natural disinfection by exploiting specific characteristics of soil-plant ecosystems

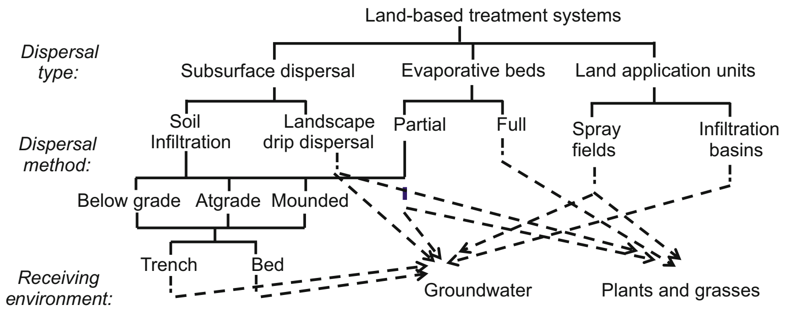


Fig. 12.1 Classification of land-based treatment systems

12.2

- Basic features of a LDU
  - Primary or secondary treated wastewater is intermittently dosed into a network of tubing placed in the shallow subsurface within a footprint area of landscape
  - Each dose of wastewater is uniformly delivered to the landscape using a pressurized network of small diameter tubing (e.g., 0.5 in.) with parallel runs of tubing (e.g., 2 ft apart) equipped with drip emitters spaced along the tubing (e.g., 2 ft apart)
  - The emitters deliver a very low rate of flow (e.g., 0.01 gal/min) for a short period of time and disperse it into the shallow soil the tubing is buried in
  - Hydraulic loading rates to the landscape footprint area of the drip tubing network are lower (e.g., 0.1–0.2 gal/day/ft^2) than those applied to subsurface soil treatment units (STUs)

12.3

- A schematic of a basic LDU is shown in Fig. 12.2

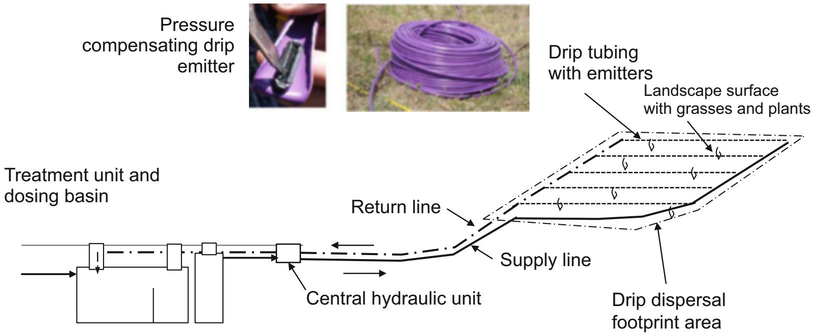


Fig. 12.2 Schematic view of a typical LDU used for decentralized wastewater treatment

12.4

- Where are LDUs used?
  - Where site conditions and soil properties are generally suitable for soil-based treatment and where landscape area is available for shallow drip dispersal
    - Landscape drip dispersal is most often used when there are constraints for using a typical soil treatment unit and where there is a desire to recover water and nutrients for their irrigation benefits
    - Choice of landscape drip dispersal over a soil treatment unit is often based on economic considerations
  - LDUs can be used in projects serving:
    - Isolated homes and businesses in rural areas
    - Clusters of sources and mixed-use development centers

12.5

12.2. Treatment Performance

- LDUs are normally designed to receive primary or secondary effluents and achieve tertiary quality percolate with natural disinfection
- LDUs can also be designed to receive tertiary quality effluents (e.g., from a membrane bioreactor) to recover water and nutrients for their fertilizer value and to provide for polishing of residual constituents of concern
- Within a LDU, treatment occurs within several zones in a fashion similar to what occurs in a STU
  - Within the rhizosphere and shallow soil of the drip dispersal area where vegetation roots are present
  - Within the unsaturated soil profile around and below the drip tubing
  - Within the deeper soil and groundwater system

12.6

- Treatment efficiency
  - Just like with a STU (Chap. 11), one approach for assessing removal efficiency (R_E) during landscape drip dispersal is to compare concentrations in soil pore water (C_{PW}) at a specific depth to the wastewater applied (C_I) (Fig. 12.3) using Eq. 12.1

$$R_E = \left[\frac{C_I - C_{PW}}{C_I}\right] \times 100\% \tag{12.1}$$

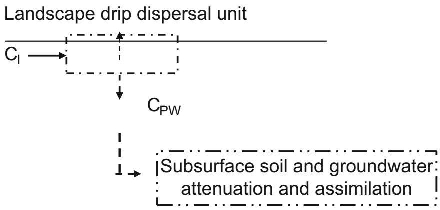


Fig. 12.3 Illustration of an approach to assess treatment efficiency in a LDU

12.7

- Treatment efficiencies achievable in a LDU are shown in Table 12.1

Table 12.1 Representative treatment efficiency achieved within a well designed and operated LDU

Constituent group	Percolate mg/L (C_{PW}) or % removal by 1–2 ft depth	Potential processes involved in treatment
BOD_5	<5 mg/L	Dissolved organics removal by biotransformation and particulate org. by sedimentation, filtration, biodegradation
TSS	<5 mg/L	Removed by sedimentation and filtration
Nitrogen	<10 mg-N/L NO_3^- 50–70 % total N	Bionitrification of NH_4+ compounds to NO_3^- with potential removal of total N by biodenitrification; plant uptake of N
Phosphorus	>90 % total P	Incorporation of P into cell mass and sorption, plant uptake; sedimentation and filtration of particulate P
Pathogens	>99.99 %	Decay, predation, phyto-inactivation, photo-inactivation
Trace organics	0 to >90 %	Near zero removal for some compounds but up to 90 % or more removal of compounds that are susceptible to sorption, volatilization and phytodegradation processes

12.8

12.3. Principles and Processes

- Many of the principles and key processes in a LDU are the same as those that occur in a STU
  - However, the processes can be more complex and dynamic
  - A network of small-diameter pressurized lines with drip emitters is used to disperse wastewater directly into near-surface soil
  - Wastewater is intermittently dosed into the network and spurts from the drip emitters and infiltrates into the native soil
  - The same hydraulic and purification processes that occur during subsurface infiltration can impact shallow drip dispersal in soil
    - However, in the shallow subsurface enhancements occur due to:
      - Higher soil porosity, organic matter content, and microbial activity
      - The presence of a “rhizosphere”

12.9

- Features of a rhizosphere
  - The rhizosphere is the zone near the ground surface that includes the root systems of growing plants (Fig. 12.4)
    - Complex relationships exist among the plant, the soil micro-organisms, and the soil itself
    - Processes and rates can be different in the rhizosphere as compared to those in deeper subsoils
    - For example, in the rhizosphere:
      - N uptake is greater
        - 34–38 % fertilizer N uptake (Engelsjord et al. 2004)
        - 20–32 % fertilizer N uptake (Horgan et al. 2002)
      - N transformations are faster
        - Nitrification: 3–7 times faster in the rhizosphere
        - Denitrification: 1.8–7 times faster in the rhizosphere (Højberg et al. 1996)

12.10

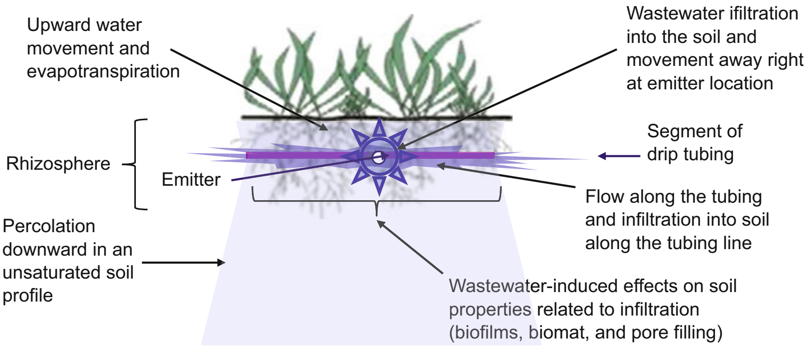


Fig. 12.4 Illustration of the rhizosphere and water movement in a segment of drip tubing placed in the rhizosphere just below the landscape surface. (*Note*: the boundaries on water movement shown are sharp and linear but in reality they are irregular and less distinct.)

12.11

- Wastewater movement from a drip emitter into soil
  - During a dosing event the drip tubing is initially empty but it is filled and then pressurized and the drip emitters discharge wastewater for a short time followed by de-pressurization and drainage
  - Wastewater that is discharged at an emitter infiltrates into the soil at the emitter and along the drip tubing (Fig. 12.4)
    - Wastewater percolates downward with some lateral movement depending on soil water content conditions
    - Upward water movement can occur by evapotranspiration (ET)
  - During and following a dispersal event, the soil water content increases, but then decreases as moisture redistribution occurs in the soil profile
  - In between dispersal events, the soil pores can become re-aerated

12.12

- Wastewater water movement in a LDU depends on site conditions and design factors (Table 12.2)

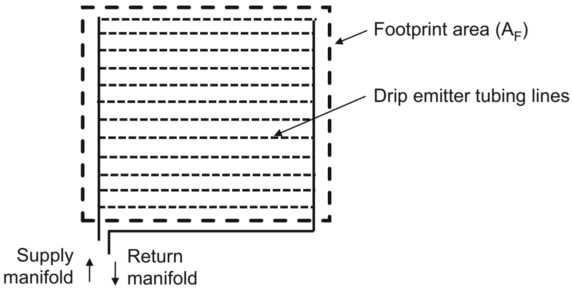
Table 12.2 Factors and their potential effects on water movement in a LDU

Factors	Considerations	Potential effects
Vegetation	Grasses and plants type and density	Nature of plant rooting and rhizosphere
Soil profile conditions	Soil texture, layering, depth to groundwater	Water content and storage, K_S
Climate and hydrology	Humid vs. semi-arid vs. arid	Relative rates of evapotranspiration vs. deep percolation
Effluent quality	Primary vs. secondary	Effluent-induced effects on infiltration, emitter plugging, field flushing frequency
Installation	Stony conditions, installation equipment	Compaction around emitter tubing, preferential flows along tubing and within subsurface
Operation	Dosing rate and frequency	Water absorption, uptake rates, effluent retention characteristics

12.13

- Landscape area needed for a drip dispersal network
  - The land area needed for drip dispersal is the footprint area encompassing the network of drip tubing installed (Fig. 12.5)
  - The size of the footprint area depends on the attributes of the drip dispersal network and the landscape's ability to assimilate water and treat pollutants and pathogens

Fig. 12.5 Illustration of the landscape footprint area associated with a network of drip dispersal tubing



12.14

- Hydraulic loading rates for sizing the dispersal area
  - The design hydraulic loading rate (HLR_D) can be based on optional approaches as illustrated in Fig. 12.6 that include:
    - HLR based on infiltration with a LTAR related to soil properties and wastewater composition similar to that used for a STU
    - HLR based on water and nitrogen mass balances for a given site

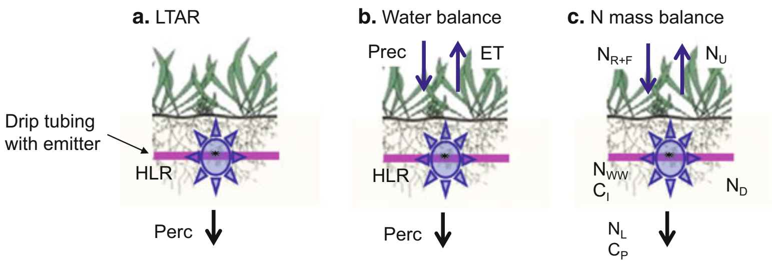


Fig. 12.6 Illustration of hydraulic loading rates for sizing a drip dispersal unit based on (**a**) a LTAR, (**b**) water balance, or (**c**) a nitrogen mass balance

12.15

- HLR_D based on a LTAR for infiltration
  - The HLR_D to the drip dispersal area is set based on a LTAR for the soil the dispersal tubing is placed in (Fig. 12.6a)
  - The LTAR-based HLR_D for sizing the footprint area leads to a ~10× higher HLR at the emitter and along the tubing (Fig. 12.7)

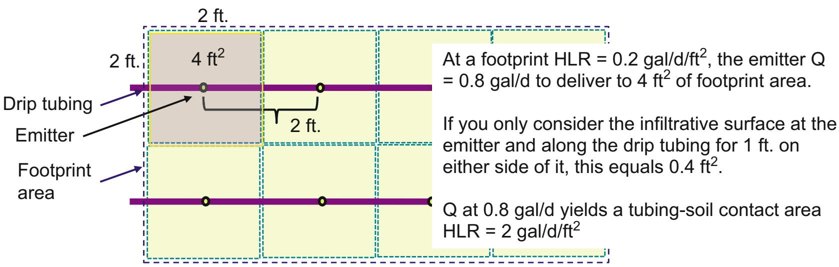


Fig. 12.7 Plan view schematic of a drip dispersal area and the hydraulic loading rates based on the footprint area versus the drip tubing infiltrative surface area

12.16

- The LDU footprint area can handle a relatively higher HLR due to dispersal in shallow soil with structural pores and where water movement is enhanced by rhizosphere processes
  - A biomat and biozone can develop in a LDU but it normally doesn't present as much resistance to flow as in a STU (Fig. 12.8)

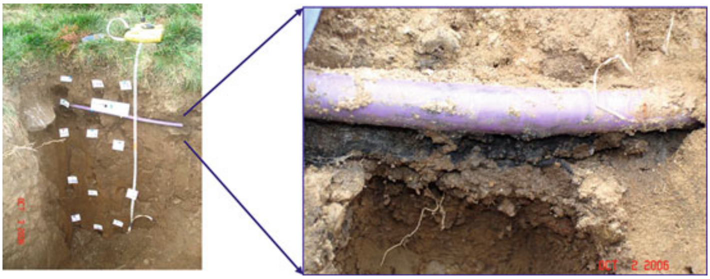


Fig. 12.8 Photographs of a segment of drip tubing placed in a sandy loam soil revealing soil structure and rooting near the landscape surface and a darkened biozone along the drip tubing (Parzen 2007)

12.17

- HLR_D based on a water balance
  - In this approach the HLR_D is set to maximize assimilation of water and nutrients while minimizing deep percolation and nutrient flux to groundwater
  - The HLR_D is selected based on a water balance (Fig. 12.6b) using Eq. 12.2
    - This approach also assumes the entire footprint area of the LDU is active

$$\mathrm{HLR_D} = (\mathrm{ET} - \mathrm{Prec}) + \mathrm{Perc} \tag{12.2}$$

Where:
HLR_D = maximum design HLR based on a water balance (gal/day/ft^2)
ET = actual evapotranspiration (gal/day/ft^2)
Prec = average monthly precipitation (gal/day/ft^2)
Perc = design deep percolation allowed into the soil profile (gal/day/ft^2)

Note: Equation 12.2 assumes no runoff onto the drip dispersal area and no change in storage in the subsurface.

12.18

- HLR_D based on a nutrient mass balance
  - In this approach the HLR_D is set to achieve a desired N loading in the deep percolation (e.g., 0 lb/acre/year) or a desired soil percolate concentration (e.g., $\leq$10 mg-N/L)
  - A nitrogen mass balance (Fig. 12.6c) is given by Eq. 12.3
    * This approach also assumes the entire footprint area of the LDU is active

$$N_L = (N_{WW} + N_{R+F}) - (N_U + N_D) \tag{12.3}$$

Where:
N_L = N loading via deep percolation flux (lb/acre/year)
N_{WW} = N applied in effluent dispersed (lb/acre/year)
N_{R+F} = N added by rainfall and fixation (lb/acre/year)
N_U = net plant uptake and storage of N (lb/acre/year)
N_D = N removal by denitrification (% of applied N)

12.19

12.4. Design and Implementation

- Considerations for design and implementation (D&I) of a landscape drip dispersal unit to achieve tertiary treatment with natural disinfection
  - Treatment goals and method of assessment
  - Site characteristics and suitability for drip dispersal
  - Wastewater treatment prior to drip dispersal
  - Loading rates for dispersal area sizing
  - Landscape area required for dispersal
  - Dispersal zones and layout of system components
  - Drip tubing features and networks
  - System hydraulics
  - Timed dosing for dispersal
  - Installation and startup, operation and maintenance, monitoring
  - Overcoming site limitations and use of design variants

12.20

- D&I considerations—Treatment goals and assessment
  - Treatment goals can be set in different ways in a fashion similar to that done for a STU (see Chap. 11)
    - It depends on how and where boundaries and points of compliance are set or required
    - Treatment goals can include the performance of the LDU alone (based on boundaries defined) or include the attenuation within the receiving soil and groundwater, e.g.: goals could be stated as:
      - NO_3—N concentrations in percolating water will be $<$10 mg-N/L when the reclaimed water reaches the groundwater table, or
      - NO_3—N concentrations in groundwater at the property boundary will be $<$10 mg-N/L
    - Assessments can be done *a priori* or after operation startup
  - Further details concerning this topic can be found in Chap. 11

12.21

- D&I considerations—Site characteristics and suitability
  - Site characteristics and soil properties need to be suitable for shallow subsurface dispersal of wastewater effluent
    - Key attributes include:
      - Climate and hydrology
      - Land area available
      - Surface slope and orientation attributes
      - Surface water drainage
      - Soil texture and structure at the dispersal depth
      - Soil profile features and limiting conditions
  - A soil and site evaluation is normally done to assess the suitability of a potential site for drip dispersal of wastewater
    - Site evaluation elements are similar to those completed for a STU as described in Chap. 11

12.22

- D&I considerations—Treatment prior to drip dispersal
  - Drip dispersal has been used for primary quality effluents
    - Domestic septic tank effluent has been used but this can pose a greater risk to hydraulic operation
      - There can be clogging in the tubing and at the emitters
      - There can be a greater rate and extent of soil clogging caused by higher concentrations of BOD, TSS, and NH_4^+—N
    - For a LDU that is receiving STE,
      - Use of a lower HLR_D is normally required
      - LDU design and operation must ensure line flushing and enable maintenance
  - Drip dispersal with secondary quality effluents
    - Higher quality effluents pose a lower risk of clogging, so higher loading rates can be employed, yielding smaller footprint areas
    - Aerobic treatment units and porous media biofilters are common

12.23

- Inline pre-filtration
  - To protect the drip tubing and emitters, disk filters or similar units are used to remove larger particles (e.g., >100 μm) (Fig. 12.9)
    - Filters are typically used in pairs and filtered water from one is used to periodically backflush the other with the backflush water discharged back into an upstream treatment tank
  - For larger flows other filtration technologies are also available

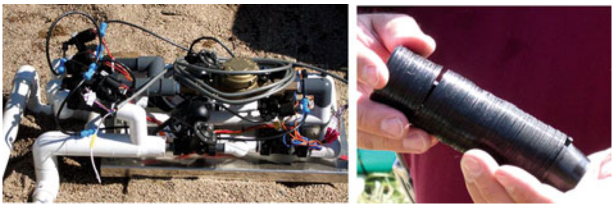


Fig. 12.9 Illustration of a hydraulic control unit (*left*) which houses two disk filters (*right*) and provides for automatic backwashing and flow metering

12.24

- In-line filtration is valuable as a precautionary measure to prevent high TSS loadings to a drip tubing network
  - However, effluent quality for most parameters after in-line pre- filtration is not markedly improved (Table 12.3)

Table 12.3 Average results from monitoring a spin-disk filtration unit in Colorado (after Parzen 2007)

Parameter	Units	STE	Disk filter effluent	% Removal
COD	mg/L	294	283	4 %
Total solids	mg/L	505	506	None
Suspended solids	mg/L	25	24	4 %
Total nitrogen	mg-N/L	72	70	3 %
Ammonium—N	mg-N/L	61	57	7 %
Nitrate—N	mg-N/L	1	1	None
Phosphate—P	mg-P/L	26	26	None
Fecal coliforms	cfu/100 mL	80,900	150,000	None

Note: Results shown are average values based on 6 grab samples collected over a 4-month period.

12.25

- D&I considerations—HLR_D for dispersal area sizing
  - The HLR_D can be determined during the design process for a LDU based on consideration of treatment goals and relevant soil properties and site conditions
  - The HLR_D can be determined based on three approaches
    - Setting the HLR_D based on infiltration of wastewater effluent in a fashion similar to that used for design of STUs
      - This accounts for soil clogging and loss in infiltrability during operation of a LDU
    - Setting the HLR_D based on a water balance
      - This can account for a desired level of deep percolation to groundwater under a LDU site
    - Setting the HLR_D based on a nitrogen mass balance
      - This can account for the concentration or mass of nitrogen that reaches groundwater under a LDU site

12.26

- Setting the HLR_D based on wastewater infiltration and soil clogging
  - In this approach, the HLR_D is set for the entire footprint area encompassing the drip tubing network
  - The footprint HLR_D is generally set based on soil type and wastewater composition in a fashion similar to that used for sizing the soil infiltrative surface area within STU (see Chap. 11)
  - HLR_Ds have been proposed by manufacturers of drip dispersal tubing and they may also be stipulated in applicable regulations
  - Table 12.4 presents several loading rates excerpted from the design guidance of two manufacturers, e.g.:
    - Sandy loam with moderate to strong structure receiving STE
      - $HLR_D = 0.15–0.2\ gal/day/ft^2$
    - Sandy loam soil with moderate to strong structure receiving secondary effluent
      - $HLR_D = 0.15–1.0\ gal/day/ft^2$

12.27

Table 12.4 Example footprint HLR_D based on soil properties and effluent quality[a]

Soil type (USDA texture)	Soil structure	American Manufacturing Co.		NETAFIM™
		Adv. primary (30–220 mg/L) (gal/day/ft^2)	Aerobic secondary ($BOD_5 < 30$ mg/L) (gal/day/ft^2)	Secondary effluent with BOD and TSS <30 mg/L and FOG < 20 mg/L (gal/day/ft^2)
Coarse sands	None	0.3–0.4	0.3–1.6	1.5
Loamy sand	Weak to strong	0.25–0.3	0.25–1.4	0.5–0.8
Loamy fine sand	Mod. to strong	0.2–0.3	0.25–0.9	0.8
Sandy loam	Mod. to strong	0.15–0.2	0.15–1	0.5
Sandy loam	Weak, platy	0.15–0.2	0.15–0.6	0.3
Silty clay loam	Weak, weak platy	<0.1	0.1–0.3	0.2
Clay	Massive, weak	0	0	0.05
Silty clay	Mod. to strong	<0.1	0.1–0.3	0.1

[a]The rates shown are excerpted from manufacturer's design guidance that includes rates for more conditions than shown based on soil texture and soil structure.

12.28

- Setting the HLR_D based on a water balance
  - The water balance for a LDU is given by Eq. 12.2 (Eq. 12.2 was also presented in the previous section)
    - Equation 12.2 assumes no supplemental irrigation or runoff onto the dispersal area and no change in storage in the subsurface

$$HLR_D = (ET - Prec - I) + Perc \tag{12.2}$$

Where:
HLR_D = maximum design HLR based on a water balance (gal/day/ft^2)
ET = actual evapotranspiration (gal/day/ft^2)
Prec = average monthly precipitation (gal/day/ft^2) (*Note*: if there is runoff from the site including all precipitation as an input is conservative)
I = supplemental irrigation (gal/day/ft^2) (typically assume I = 0)
Perc = design deep percolation allowed into the soil profile (gal/day/ft^2)

Note: 1 gal/day/ft^2 = 586.5 in./year.

12.29

- Estimating the precipitation and deep percolation at a site
  - Precipitation (Prec) can be obtained from weather records
    - A value used for Prec in Eq. 12.2 can be based on the monthly precipitation as estimated by Eq. 12.4

$$Prec = P_M + 0.85(SD_M) \tag{12.4}$$

Where:
Prec = design monthly precipitation rate (in./month)
P_M = mean monthly precipitation based on 30-year data (in./month)
SD_M = standard deviation of monthly precipitation based on 30-year data (in./month)

  - A value for deep percolation (Perc) can result from one of the following approaches:
    - Perc based on a water balance with a chosen HLR
    - Perc selected *a priori* as design parameter, e.g., <5–10 % of K_S of a low permeability layer

12.30

- Estimating the rate of evapotranspiration (ET) at a site
  - * ET rates depend on environmental conditions:
    - – Net radiation, wind speed, humidity
    - – Type and density of grasses, shrubs, trees, etc.
    - – Amount of water supplied to the vegetation
  - * Potential ET vs. Actual ET (Eq. 12.5)
    - – Potential ET (or PET) is the maximum amount of water removed by evaporation and transpiration under the current environmental conditions if water supply to plants is unlimited
    - – Actual ET is the amount of water removed by evaporation and transpiration under the current soil moisture and environmental conditions

$$\mathrm{ET} \leq \mathrm{PET} \tag{12.5}$$

Where:
ET = actual evapotranspiration rate (in./month)
PET = potential evapotranspiration rate (in./month)

12.31

- * Potential ET can be calculated using empirical models
  - – Models differ in formulation and applicability
  - – One model is based on the Thornthwaite method and Eqs. 12.6, 12.7, and 12.8

$$\mathrm{PET} = 1.6\mathrm{L_d}\left(\frac{10\mathrm{T}}{\mathrm{I}}\right)^{\mathrm{A}} \tag{12.6}$$

$$\mathrm{I} = \sum_{j=1}^{12}\left[\mathrm{I_j} = \frac{(\mathrm{T_j})^{1.514}}{5}\right] \tag{12.7}$$

$$\mathrm{A} = 0.000000675\ \mathrm{I}^3 - 0.0000771\ \mathrm{I}^2 + 0.01792\ \mathrm{I} + 0.49239 \tag{12.8}$$

Where:
PET = 30-day potential evapotranspiration rate (cm/month)
L_d = time from sunrise to sunset in multiples of 12 h
T = monthly mean air temperature (°C) (30-year ave. obtained from NOAA)
I = annual heat index = sum of monthly indices, I_j for j = 1–12
I_j = monthly heat index used to calculate I
A = power term derived from the annual heat index, I

Source: EPRI 2004; Lu et al. 2005

12.32

- Setting the HLR_D based on a nitrogen mass balance
  - Equation 12.3 represents a mass balance on nitrogen (Eq. 12.3 was also presented in the previous section)

$$N_L = (N_{WW} + N_{R+F}) - (N_U + N_D) \quad (12.3)$$

Where:

N_L = N loading via deep percolation flux (lb/acre/year)
= Percolation water flux times the N concentration

N_{WW} = N applied in wastewater dispersed (lb/acre/year)

N_{R+F} = N added by rainfall and fixation (lb/acre/year) (5 lb/acre/year typ.)

N_U = net plant uptake and storage of N (lb/acre/year)

N_U depends on location and vegetation type and needs to be carefully chosen. 200 lb/acre/year has been used for grasses like tall fescue, ryegrass, Kentucky bluegrass (EPRI 2004)

N_D = N removal by denitrification = $N_{DE} \times (N_{WW} + N_{R+F})$ (lb/acre/year)

N_{DE} = Denitrification removal efficiency (%)

e.g., loamy sand = 5–15 %, sandy loam = 15–50 %, Loam or clay loam = 30–70 % (Beggs et al. 2011)

12.33

- Example of how to set a HLR_D that yields $N_L = 0$
  - * The input parameter values needed for Eq. 12.3 are based on literature data relevant to the site and soil conditions
    - – For this example, assume:

      N_{R+F} = 5 lb-N/acre/year and N_U = 200 lb-N/acre/year
      N_{DE} = 40 % of $(N_{WW} + N_{R+F})$ to yield N_D in lb-N/acre/year
  - * Using these input parameters for Eq. 12.3 you can find the allowable rate of N addition in the wastewater (N_{WW})

$$N_L = (N_{WW} + N_{R+F}) - (N_U + N_D) = 0 \quad (12.3)$$

$$N_{WW} = 328 \ \frac{\text{lb} - \text{N}}{\text{acre}/\text{year}}$$

12.34

* Next, you use Eq. 12.9 to find the HLR_D that yields the calculated N_{WW} for the goal of $N_L = 0$

$$HLR_D = \left(\frac{N_{WW}}{C_I}\right)0.0075 \tag{12.9}$$

Where:
HLR_D = design hydraulic loading rate for $N_L = 0$ (gal/day/ft^2)
N_{WW} = mass of N applied in the effluent dispersed (lb/acre/year)
C_I = concentration of N in the effluent dispersed (mg-N/L)
0.0075 = conversion factor for lb-N/acre/year and mg-N/L to gal/day/ft^2

* For Eq. 12.9, you must select the N concentration in the wastewater dispersed in the LDU
  – In this example, 50 mg-N/L was used and the calculated HLRD of 0.0495 gal/day/ft^2 should yield $N_L = 0$

$$HLR_D = \left(\frac{328\ \ \text{lb N/acre/year}}{50\,\text{mg} - \text{N/L}}\right)0.0075 = 0.0495\,\frac{\text{gal/day}}{\text{ft}^2} \Rightarrow 0.58\ \ \frac{\text{in.}}{\text{week}}$$

12.35

○ Example of how to set a HLR_D that yields $C_P = 10$ mg-N/L
  * Equations 12.10 and 12.11 are relationships for N_L and N_{WW} that can be input into the mass balance in Eq. 12.3

$$N_L = (\text{Perc})(C_P)132.8 \tag{12.10}$$

$$N_{WW} = (\text{HLR})(C_I)132.8 \tag{12.11}$$

Where:
N_L = N loading via deep percolation flux (lb/acre/year)
Perc = design deep percolation allowed into the soil profile (gal/day/ft^2)
C_P = concentration of N in the deep percolate (mg-N/L)
N_{WW} = mass of N applied in the effluent dispersed (lb/acre/year)
HLR = hydraulic loading rate (gal/day/ft^2)
C_I = concentration of N in the effluent dispersed (mg-N/L)
132.8 = conversion factor gal/day/ft^2 and mg/L to lb/acre/year

12.36

* Inserting Eqs. 12.10 and 12.11 into Eq. 12.3 yields Eq. 12.12a

$$N_L = (HLR_D)(C_I)132.8 + N_{R+F} - N_U - N_{DE}[(HLR_D)(C_I)132.8 + N_{R+F}] \quad (12.12a)$$

* Substituting the previously assumed values for N_U, N_{R+F}, and N_{DE} into Eq. 12.12a provides Eq. 12.12b for N_L
  – Eq. 12.12b is unique for a specific denitrification efficiency, which in this case is $N_{DE} = 40\,\%$

$$N_L = 0.60(HLR_D)(C_I)132.8 - 197 \quad (12.12b)$$

12.37

* Solving for the HLR_D that yields $C_P = 10$ mg-N/L
  – N_L is given by both Eq. 12.10 and Eq. 12.12b and Perc can be determined by the water balance given by Eq. 12.2 with ET = 32.46 in./year = 0.055 gal/day/ft^2, Prec = 74.8 in./year = 0.128 gal/day/ft^2, and I = 0

$$(Perc)(C_P)132.8 = 0.60(HLR_D)(C_I)132.8 - 197 \quad \text{Eq. } 12.10 = 12.12b$$

$$(HLR_D - 0.055 + 0.128 + 0)(10)132.8 = 0.60(HLR_D)(50)132.8 - 197$$

$$HLR_D = \frac{293.9}{2656} = 0.111\,\frac{gal/day}{ft^2} = 0.45\,\frac{cm}{day} = 1.29\,\frac{in.}{week}$$

○ Comparison of HLR_D for a sandy loam soil with $C_I = 50$ mg-N/L
  * Based on a LTAR → 0.20 gal/day/ft^2 (Table 12.4)
  * For $N_L = 0$ ($N_{DE} = 40\,\%$) → 0.049 gal/day/ft^2
  * For $C_P = 10$ mg-N/L ($N_{DE} = 40\,\%$) → = 0.11 gal/day/ft^2

12.38

- D&I considerations—Landscape area for dispersal
  - The landscape footprint area required for the LDU can be calculated using Eq. 12.13
    - The value for HLR_D can be based on one of the three approaches just discussed: (1) the infiltration of wastewater effluent, (2) a water balance, or (3) a nitrogen mass balance

$$A_F = \frac{(Q_D)}{HLR_D} \tag{12.13}$$

Where:
A_F = landscape dispersal area (footprint area) required (ft^2)
Q_D = design daily flow (gal/day) (e.g., $Q_A \times PF$)
HLR_D = design HLR (gal/day/ft^2)

Note: HLR_D can be based on a selected value to achieve effluent infiltration and deep percolation or to account for water and nutrient balances. Footprint area may have to be increased by other considerations such as groundwater mounding and nutrient loadings.

12.39

- The minimum landscape footprint area required for the LDU can also be directly calculated based on a nitrogen mass balance
  - A simplified approach to estimating the minimum footprint area needed was proposed by Beggs et al. (2011)
  - This approach is used to determine the minimum area required for a drip dispersal area so the nitrate concentration in shallow groundwater does not exceed 10 mg-N/L
  - This approach accounts for the following processes and employs the following assumptions
    - The water balance interactions with N fate
    - Dilution that can occur by clean water from other sources of deep percolation or groundwater that flows into the dispersal area recharge zone
    - The only sources of N are from the effluent applied
  - Figure 12.10 shows the schematic illustrating the parameters used in Eq. 12.14

12.40

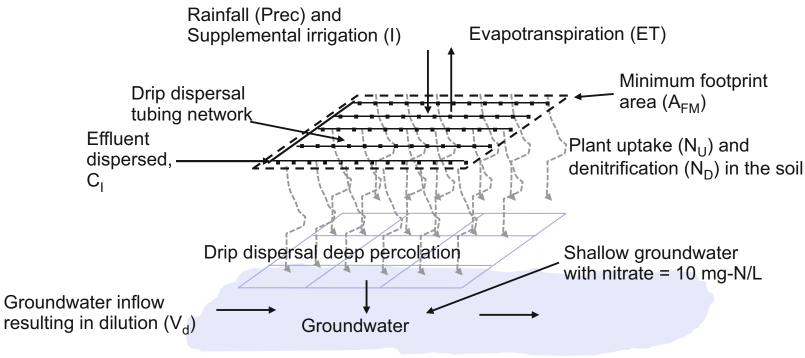


Fig. 12.10 Definition schematic for calculating the minimum footprint area required to maintain a nitrate concentration of 10 mg-N/L in shallow groundwater under a drip dispersal area

12.41

$$A_{FM} = \frac{[C_I(1 - N_{DE}) - 10]HLR_D - 10V_d}{10(Prec + I - ET) + N_U} \tag{12.14}$$

Where:

A_{FM} = minimum dispersal footprint area required (m^2)

C_I = nitrogen applied in effluent (mg/L, g/m^3)

N_{DE} = denitrification (% of applied N)

Loamy sand (5–15 %; Typ. 10 %), Sandy loam (15–50 %; Typ. 30 %), Loam or clay loam (30–70 %; Typ. 50 %)

HLR_D = wastewater application rate (m^3/year)

N_U = plant nitrogen uptake (g/m^2/year)

Prec = precipitation (m/year)

I = supplemental irrigation (m/year)

ET = actual plant evapotranspiration (m/year)

V_d = additional dilutive volume (m^3/year) (e.g., groundwater dilution flow from designated area surrounding the dispersal field)

12.42

- The landscape footprint area required is typically controlled by wastewater infiltration or nutrient loading considerations
  - Key factors affecting the size of the LDU footprint area required
    - Soil type and effluent quality (BOD_5, TSS)
    - Climate and hydrology (e.g., ET, Prec)
    - Nitrogen concentrations in the wastewater dispersed
    - Limitations on nitrogen concentrations in groundwater
  - In general, the following holds true concerning LDU sizing
    - For well-draining sands and similar coarse-grained soils
      - Footprint area would be controlled by nitrogen limitations due to a lower rate of denitrification in these soils
    - For slowly-draining silt loams and other fine-grained soils
      - Footprint area would be controlled by infiltration rate limitations due to a lower infiltrability and the effects of soil clogging in these soils

12.43

- Accounting for seasonality effects during LDU area sizing
  - Some project locations have environmental conditions that vary from month to month during a typical calendar year
    - Conditions can affect system function and sizing, e.g.:
      - ET is limited when the landscape is snow covered
      - Plant uptake of N is limited when grasses and plants are dormant or in slow growth mode
  - How to account for seasonality in drip dispersal sizing?
    - HLR_D based on wastewater infiltration and deep percolation
      - Assumed to be independent of seasonality
    - HLR_D based on water and nutrient balances
      - Balances are done on a monthly basis
      - HLR_D can be selected to represent a limiting period
  - There are two basic options for periods of limiting conditions
    - Have HLR_D for different seasons and excess footprint area
    - Provide storage using basins or ponds

12.44

- D&I considerations—Use of multiple dispersal zones
  - The LDU footprint area required is often divided into zones
    - Multiple zones have advantages
      - Enable dispersal areas and lines to best fit the landscape
      - Keep the pumping rate for dispersal and flushing at a manageable level (if zone flow control valves are used)
    - Multiple zones do not need to be equal in footprint area when pressure-compensating emitters are used (Fig. 12.11)
      - Emitter gal/h is constant and with timed dosing, the time of delivery each day can be the same for all zones

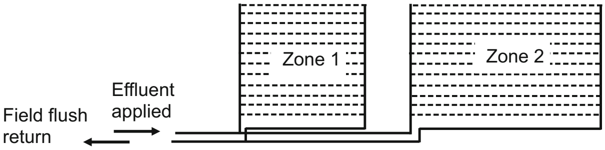


Fig. 12.11 Illustration of a LDU divided into two dispersal zones of different sizes

12.45

- D&I considerations—Layout of LDU components
  - Layout for setback distances
    - Treatment unit, dosing tank, and dispersal network are subject to setback requirements (e.g., property lines, buildings, etc.)
  - Layout for return flows
    - Central hydraulic unit with the in-line pre-filtration and backflushing apparatus should be located so the pre-filter backflush and drip tubing flush water can flow by gravity back to the treatment unit (Fig. 12.12)

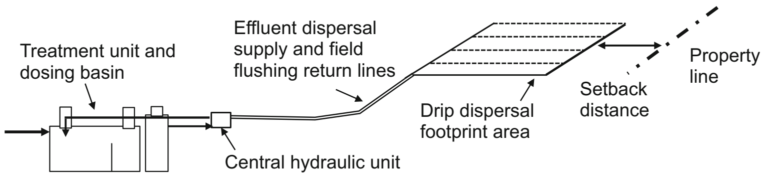


Fig. 12.12 Illustration of system layouts to satisfy setback distances and return flows

12.46

- An example layout with two drip dispersal zones is shown in Fig. 12.13

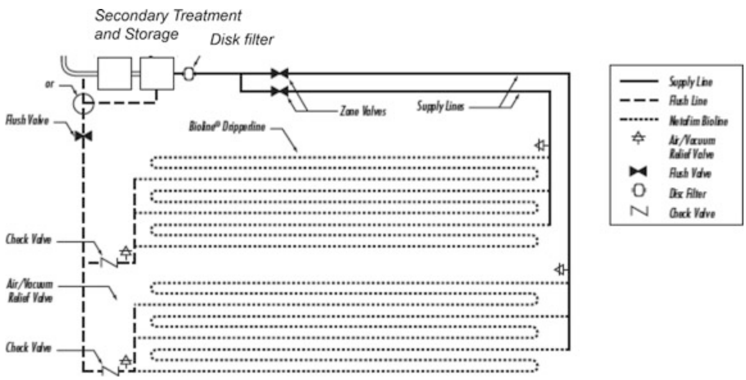


Fig. 12.13 Illustration of a layout with two drip dispersal zones (NETAFIM™ 2008)

12.47

■ D&I considerations—Drip dispersal tubing

- Drip dispersal tubing features
  - Tubing is commonly manufactured in 500 or 1000 ft long rolls
  - Drip emitters are part of the manufactured tubing and are commonly spaced 2 ft apart (Fig. 12.14)

Fig. 12.14 Photographs of a length of drip dispersal tubing outfitted with pressure compensating emitters every 2 ft

12.48

- Emitters can be turbulent flow or pressure-compensating
  - Turbulent flow emitters
    - Flow rate depends on in-line pressure (Fig. 12.15)
    - Higher flow rates are possible
  - Pressure-compensating emitters
    - Flow rate is largely constant (Fig. 12.15)
    - Flow rates are limited to lower rates

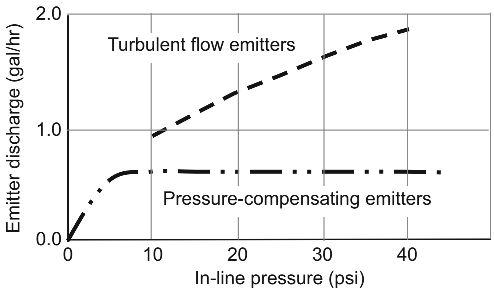


Fig. 12.15 Flow rate characteristics of different types of LDU emitters (after USEPA 2002)

12.49

- Drip tubing length and installation depth
  - Tubing is normally installed along the landscape contours
    - Parallel tubing runs are typically spaced 2 ft apart
    - Tubing separation can be made somewhat closer based on soil properties and ET rates
  - Total linear feet of tubing is given by Eq. 12.15

$$LF = \frac{A_F}{S_T} \tag{12.15}$$

Where:
LF = linear feet of drip tubing (ft)
A_F = footprint area (ft^2)
S_T = spacing between parallel lines of tubing (ft)

  - Tubing is installed at a consistent depth in a shallow adequately permeable soil horizon
    - Typical depths are 6–18 in. below ground surface

12.50

- Drip tubing network layout
  - Drip tubing layouts include run(s) and laterals (Fig. 12.16)
    - * Single runs: 1 lateral = 1 run Looped runs: 1 lateral = 2 runs
      - – Drip tubing laterals are the effluent distribution laterals
    - * Manifolds deliver flow to a zone and return flow from periodic tubing flushing back to the treatment unit
    - * Air/vacuum release and check valves are used to prevent air blockage and control backflow in the network

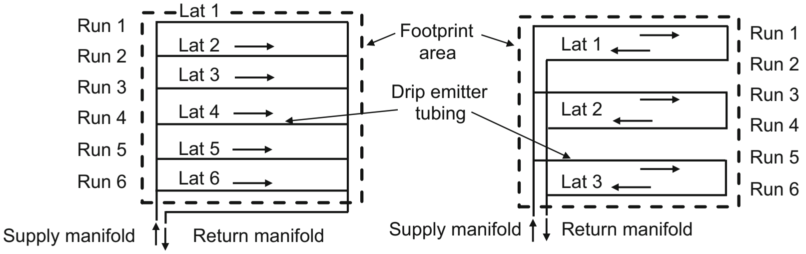
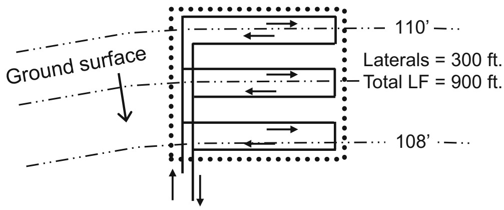


Fig. 12.16 Illustration of single (*left*) versus looped runs (*right*) in a LDU network layout

12.51

- General guidance concerning drip tubing layouts
  - * Laterals should be laid so they are parallel with the landscape contours (Fig. 12.17)
    - – A run along a given contour should not vary off an even grade by more than about 0.5 ft/100 ft
  - * Each lateral in the same zone should be roughly the same length and be ≤300 ft to control head loss
  - * On sloping sites, downslope laterals can be overloaded
    - – To minimize this, small diameter manifolds should be used with zones that do not exceed 1200 ft of tubing

Fig. 12.17 Illustration of a LDU network layout on a sloping site



12.52

- Illustration of a two-zone system with supply and return is shown in Fig. 12.18

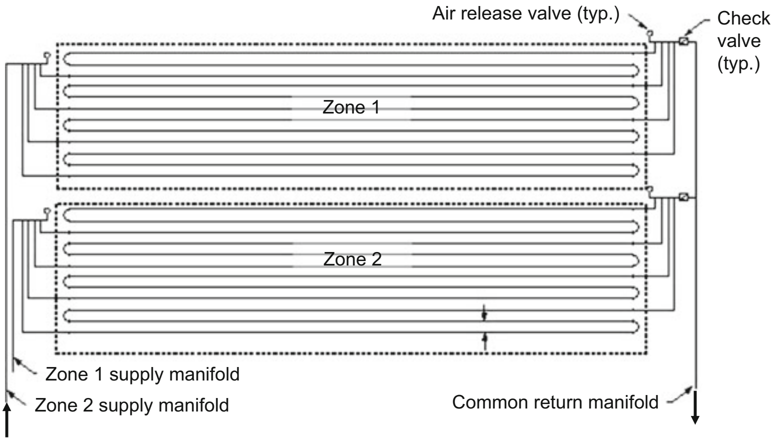


Fig. 12.18 Illustration of a LDU with two zones showing two supply manifolds and a common return line. (*Source*: www.americanonsite.com/american/tfz243-r.html) 12.53

- D&I considerations—System hydraulics
  - Dosing tank
    - Provides flow equalization and emergency storage
    - Dosing tank volume is based on the design daily flow according to Eq. 12.16

$$V_{DT} = (Q_D)(HRT) \tag{12.16}$$

Where:
V_{DT} = volume of the dosing tank (gal)
Q_D = design daily flow (gal/day) (e.g., $Q_A \times PF$, with PF = 1.5)
HRT = hydraulic retention time (days) (typ. 1–2 day)

    - The dosing tank can be a separate compartment within a treatment unit (e.g., a septic tank or aerobic unit) or a separate tank or basin

12.54

- Central hydraulic unit (pump plus filter assembly)
  - The size of the hydraulic unit impacts system costs
    - Using multiple zones can enable smaller units
  - Options often include self-contained or skid-mounted units[a]
    - Self-contained units—15 or 25 gal/min for 2, 4, 8 or 16 zones
    - Skid-mounted units which need heated housing—Start at 40 gal/min for systems with multiples of 8 zones
  - Figure 12.19 shows a central hydraulic unit being installed

Note: It is critical in cold climate applications that the hydraulic unit and piping is protected from freezing conditions.

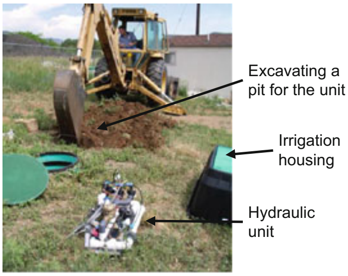


Fig. 12.19 Photographs taken during installation of a hydraulic control unit in sprinkler irrigation housing at the Mines Park Test Site in Colorado

[a]*Source*: www.americanonsite.com.

12.55

- Flow regime for a drip dispersal unit
  - An illustration of the flow regime within a typical drip dispersal unit is shown in Fig. 12.20
  - Flow and residuals from back-flushing the drip network pre-filtration unit and the routine field flushing of the drip tubing lines are returned to the treatment unit

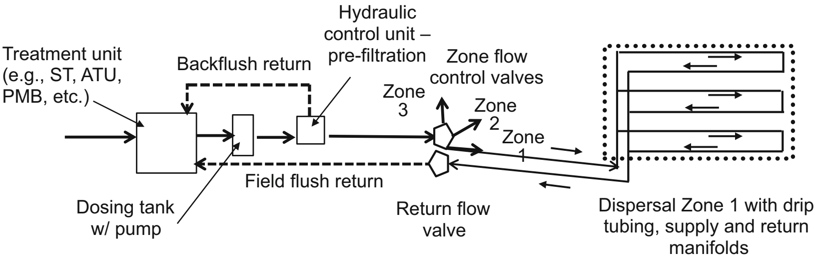


Fig. 12.20 Illustration of the flow regime within a typical LDU containing multiple zones and zone flow control valves

12.56

- Flow rates during dosing for drip dispersal and tubing flushing
  - Dispersal
    - Zone flow rate = no. emitters × emitter flow rate

      e.g. for 150 pressure-compensating emitters, the zone flow rate = (150)(0.65 gal/h)/(60 min/h) = 1.6 gal/min
  - Flushing
    - Drip tubing lines need to be flushed periodically to remove accumulated solids and biofilms

      Can set the field flushing to occur based on dispersal cycles—e.g., every 20 dispersal cycles
    - An adequate velocity for flushing is typically about 2 ft/s

      To achieve this velocity in 0.5-in. diam. tubing (ID = 0.57 in.) requires a flow rate of about 1.6 gal/min

12.57

- Flow rate during dispersal to a zone is given by Eq. 12.17

$$Q_{Dis} = \left(\frac{LF_Z}{S_E}\right)(Q_E) \tag{12.17}$$

Where:
Q_{Dis} = flow rate during dispersal in a zone (gal/min)
LF_Z = lineal length of tubing in a zone (ft)
S_E = spacing between emitters along the tubing (ft)
Q_E = discharge rate from an emitter (gal/min)

- Flow rate during flushing a zone is given by Eq. 12.18

$$Q_{Flu} = (Q_S)(L_N) \tag{12.18}$$

Where:
Q_{Flu} = flow rate during flushing in a zone (gal/min)
Q_S = flow rate required for scouring (gal/min) (typ. 1.6 gal/min in 0.5-in. diameter tubing)
L_N = number of laterals in a zone (–)

12.58

- Total pumping flow requirement is given by Eq. 12.19
  - Pumping requirements = flow during flushing a zone
  - If multiple zones are used and not the same size, the largest zone will have the highest total Q

$$Q_P = Q_{Dis} + Q_{Flu} \tag{12.19}$$

Where:
Q_P = flow rate a pump needs to be able to deliver against the TDH (gal/min)
Q_{Dis} = flow rate during dispersal in the largest zone (gal/min)
Q_{Flu} = flow rate during flushing of the tubing in the largest zone (gal/min)

12.59

- Potential head loss components in a drip dispersal network
  - The hydraulic design is based on determining the total dynamic head (TDH) for the system based on potential components (Eq. 12.20)

$$TDH = (h_s + h_{fu}) + (h_{fs} + h_{fr}) + (h_{fl} + h_r) \tag{12.20}$$

Where:
h_s = system static or elevation head (ft)
h_{fu} = head loss in the pump discharge and filtration unit (ft)
h_{fs} = friction loss in the transport piping and supply manifold delivering flow to a zone (ft)
h_{fr} = friction loss in the return manifold and transport piping delivering flow from a zone back to the treatment unit (ft)
h_{fl} = friction loss in a lateral drip line during a flushing event (ft)
h_r = pressure loss at an emitter during flow out of the emitter (ft)

12.60

- Description of potential TDH components
  - Static head (h_s) is the difference in elevation between the pump-off water level in the dosing tank and the elevation of the drip tubing lines
  - Filtration unit head losses (h_{fu}) include those through the pump and filtration unit
    - h_{fu} is tabulated in manufacturer literature for specific types of pumping and filtration units
      - For example, for different AMC filtration units, a chart of head loss vs. flow is given[a]

        e.g., for a "25 gal/min unit" size, with a flow of 15 gal/min, the headloss is 10 ft.
    - Head losses due to pipe fittings are generally negligible for small units

[a]*Source*: http://www.americanonsite.com/american/pdf/Engineering%20Catalog%20-%20FILTRATION%20HEAD%20LOSS%20CHARTS.pdf

12.61

- Friction losses result from delivery and return flows (h_{fs}, h_{fr})
  - In the transport piping to and from the zone (or zone flow control valve) and return valve locations
  - In the supply manifold and return manifold during flushing
  - Headloss during flow can be estimated for a pipe diameter and length using the Hazen-Williams equation (Eq. 12.21)

$$h_{fp} = 10.5(L)\left(\frac{Q}{C}\right)^{1.85}\left(D^{-4.87}\right) \tag{12.21}$$

Where:
h_{fp} = headloss caused by flow in the pipe (ft)
L = length of pipe (ft)
Q = pipe flow rate (gal/min)
C = Hazen-Williams coefficient (C = 150 for new plastic pipe)
D = true inside diameter of pipe (in.)

Note: Manufacturers can use specialty tubing that has different head loss characteristics during dosing and flushing. Manufacturer's literature often presents charts that can be used for specific applications.

12.62

- Friction losses occur during dispersal flow in a lateral (h_{fl})
  - During dispersal, head loss depends on tubing type, diameter, length, and emitter spacing
    - Estimate the head loss using Eq. 12.21 but account for the flow out of the emitters in the tubing
    - The h_f in a lateral with flow out of it along its length is roughly about 1/3 of that in equal-length solid-wall tubing
  - You can also use a manufacturer's table or graph for head loss vs. flow during dispersal[a]
    - For example, in a 300-ft long lateral with 150 orifices each dispersing at 0.65 gal/h, the flow is about 1.6 gal/min
    - For these dispersal conditions, the lateral head loss is listed as about 6.2 ft (2.7 psi)

[a]*Source*: http://www.americanonsite.com/american/pdf/Engineering%20Catalog%20-%20DRIPPER%20LINE%20HEAD%20LOSS%20CHARTS.pdf

12.63

- Friction losses occur during flushing flow in a lateral (h_{fl})
  - During flushing, h_{fl} is greater than during dispersal only
    - Pressure required for flushing needs to yield a flushing velocity while also accounting for the lateral head loss that is normal during dispersal
  - During flushing, estimate h_{fl} using Eq. 12.21
    - Include the 1.6 gal/min flushing flow (to yield 2 ft/s) plus the calculated dispersal flow (this is conservative)
  - Manufacturers also tabulate head loss per 100 ft of lateral during a flushing event
    - For example, for a 300-ft long lateral, to sustain a field flushing flow of 1.6 gal/min throughout the entire lateral, requires a head loss of 35 ft (15.2 psi) as listed by AMC[a]

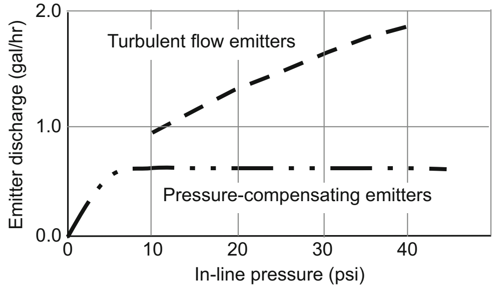
[a] *Source*: http://www.americanonsite.com/american/pdf/Engineering%20Catalog%20-%20DRIPPER%20LINE%20HEAD%20LOSS%20CHARTS.pdf

12.64

- Head loss (h_r) occurs at an emitter during flow through the emitter into the soil outside it
  - For a pressure-compensating emitter, if the in-line pressure is in the range of 15 psi to 45 psi the emitter discharge will be controlled at about 0.6 gal/h (0.01 gal/min) (Fig. 12.21)

    Note: An in-line pressure of 15 psi at an emitter is equivalent to a head loss of about 35 ft during discharge through the emitter.

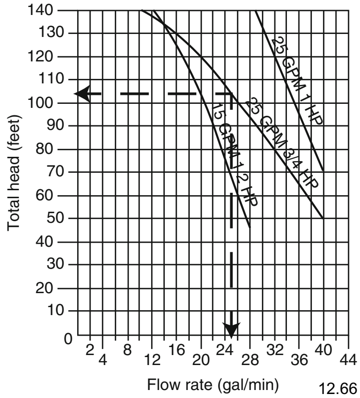
Fig. 12.21 Flow rate characteristics of different types of emitters (after USEPA 2002)



12.65

- Pumping requirements
  - Total pump Q = flow during flushing a zone
  - If multiple zones are used and not the same size, the largest zone will have the highest total Q_P (Eq. 12.19)
  - Pump must have capacity to deliver Q_P against the TDH in the delivery and dispersal system
  - Pumps are typically high head and moderate flow (Fig. 12.22)

Fig. 12.22 Illustration of a ¾-HP pump delivering about 25 gal/min against a TDH of about 100 ft (*Source*: www.americanonsite.com)



12.66

- D&I considerations—Dose volume and timed dosing
  - Dose volume for dispersal and time of occurrence
    - Wastewater is intermittently dosed into each zone of a LDU
      - Volume per dose during dispersal to a zone needs to provide for uniform distribution throughout the zone
      - The number of dispersal events (i.e., dosing events) and time of occurrence can be adjusted through the controls included with a landscape drip dispersal unit
    - Typically there are 3–5 dispersal events per day under average daily flow conditions with a higher number occurring during periods of peak daily flows
    - The number of dispersal events per day can be constrained due to network size
      - The dose volume used for dispersal needs to be 3–5× the network fill volume so that uniform distribution can occur after the network is pressurized

12.67

- Volume per dose during dispersal (without flushing) to a zone within a LDU with equal size zones is given by Eq. 12.22

$$V_{DE} = \frac{Q_A}{(N_Z)(D_{PD})} \tag{12.22}$$

Where:

V_{DE} = volume of a dispersal event to one of two or more equal size zones (gal)

Q_A = average daily flow (gal/day)

N_Z = number of equal size zones within the drip dispersal unit (–)

D_{PD} = dispersal events per day per zone under average flow (typ. 3–5 are used, but more doses per day can be needed for lower permeability soil conditions)

Notes: The number of doses can be constrained due to network size since the dose volume needs to be 3–5× the network fill volume to achieve uniform distribution. If the daily flow varies above or below average, D_{PD} can correspondingly increase or decrease.

12.68

○ The pump on and off times during wastewater dispersal under average daily flow conditions are given by Eqs. 12.23 and 12.24

$$P_{on} = \frac{V_{DE}}{Q_{Dis}} \tag{12.23}$$

$$P_{off} = \left[\frac{T_D}{N_Z \times D_{PD}}\right] - P_{on} \tag{12.24}$$

Where:

P_{on} = time a pump is running during dispersal to a zone (min)

P_{off} = time a pump is off between dispersal events under average daily flow conditions (min) (under higher flow conditions more doses occur and P_{off} time is lower)

V_{DE} = volume of a dispersal event to a zone (gal)

Q_{Dis} = the flow rate during dispersal (gal/min)

T_D = portion of a day dispersal occurs (min/day) (e.g., 18–24 h/day)

N_Z = number of zones the pump will deliver dispersal flow to

D_{PD} = dispersal events per day per zone under average flow conditions (e.g., 3–5 for drip dispersal)

12.69

- D&I considerations—Installation at a site
  - Installation practices can be dictated by site conditions
    - ○ When shallow subsurface conditions have stones and rock fragments, dispersal tubing placement may be more disruptive (e.g., using a narrow backhoe rather than a continuous trencher)
    - ○ Figure 12.23 shows the installation of a LDU at a site in Colorado
  - Startup activities
    - ○ Careful startup is important to long-term performance
    - ○ Avoid startup during cold periods when vegetation is dormant
    - ○ If the LDU is installed in a relatively barren landscape, placing sod or planting vegetation can aid rhizosphere development and the function and performance of the LDU

12.70

- Key startup activities and events
  * Verify that all piping connections are solid
  * Flush lines to remove any soil and debris
  * Clean the disk filters at the end of start-up
  * Record and examine initial readings including: cycle counters, elapsed time meters, original timer settings, pump amperage, and zone flow rates (dispersal and flushing) and operating pressures
  * Initiate dosing of the drip dispersal area

12.71

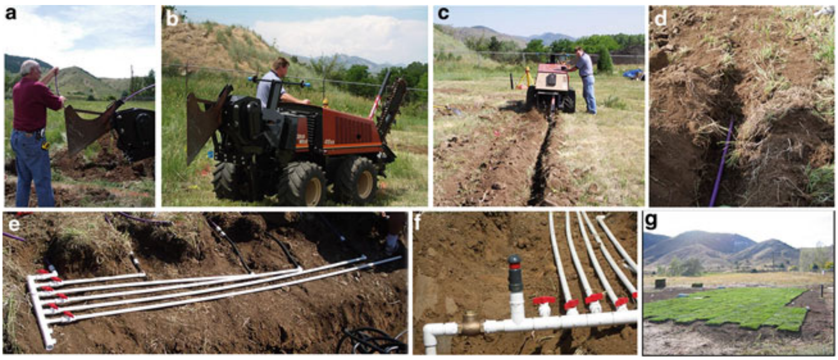


Fig. 12.23 Photographs taken during installation of a drip dispersal unit showing: (**a**–**c**) the installation machinery, (**d**) the tubing placed in a narrow shallow trench, (**e**) a supply manifold with lateral shutoff valves, (**f**) a return manifold with shutoff valves and a check valve and air release valve, and (**g**) placement of Kentucky Bluegrass

12.72

- D&I considerations—Operation and maintenance
  - LDUs have components and controls that require O&M
  - Potential routine O&M requirements
    - Inspect and maintain all upstream treatment units (e.g., with a septic tank, clean effluent screens and pump septage)
    - Inspect and record conditions related to infiltration
      - Landscape inspection—inspect the ground surface above the LDU to ensure no seepage is occurring—if seepage is present, consider corrective actions (e.g., increase the number of doses/day, reduce the HLR)
      - Verify that all portions of the LDU that are supposed to be operational are receiving flow—adjust as needed
      - Wastewater influent flow or quality—if outside design limits consider flow reduction and/or higher treatment

12.73

- D&I considerations—Monitoring and controls
  - Basic monitoring is enabled by the controls within a LDU
    - Total flow to the system and each zone within it, including zone flow rates and pressures
    - Controls also include various sensors and alarms, e.g.: for zone high and low flow rates, unusual water levels, pump failure
    - Basic monitoring can be done remotely via telemetry
  - What may or may not be required and/or feasible
    - Monitoring of the wastewater to be dispersed in the LDU is costly if done properly, and it is normally not needed, except for:
      - LDUs serving commercial or institutional buildings or developments
    - Sampling and analysis of soil and groundwater is difficult and costly, and should only be considered for special cases, e.g.:
      - Larger systems (e.g., $\geq$ 25,000 gal/day), particularly those in sensitive environmental areas

12.74

- Other sensors and alarms can be included if deemed important for monitoring and process control purposes, e.g.:
  - Water level sensor for a dosing basin to detect and provide an alert if there is a pump or siphon problem
  - Level sensors to detect and measure the depth of ponding along the dispersal tubing
  - Subsurface sensors for measuring soil water content and temperature below the footprint area of a LDU
  - Data acquisition and communication can often be accomplished using telemetry options

12.75

- D&I considerations—Overcoming site limitations and use of design variants
  - There are situations with limitations for a LDU and site modifications or variants on system design may be warranted
  - Example conditions posing constraints to a LDU include:
    - Sites with soil profile limitations
      - At sites where there is high groundwater, shallow bedrock or low permeability soil conditions the LDU could be placed in fill materials at-grade or in a mound (refer to Chap. 11)
    - Sites with freezing winter conditions and short growing seasons
      - A LDU could be deployed and used only during the warm weather growing season
      - During freezing conditions wastewater treatment would occur by another means
      - For example, the LDU could be used during the growing season and a STU could function during winter conditions

12.76

12.5. Summary

- Landscape drip dispersal involves a network of pressurized tubing with emitters that is inserted just below the land surface
  - Wastewater effluent is intermittently dispersed from drip emitters and infiltrates into the soil (secondary effluents pose less O&M risks compared to primary effluents)
  - LDU hydraulic and purification processes are similar to those in a STU but also include the rhizosphere and climate processes
  - Enhanced treatment and assimilation can occur due to the presence of a rhizosphere, which is the zone where complex relations exist among the plants, soil microbes, and the soil itself
- A well-designed and operated LDU can achieve tertiary treatment with natural disinfection and enable beneficial reuse of water and nutrients

12.77

12.6. Example Problems

- 12EP-1. Determining the HLR_D for dispersal based on water and nutrient balances
  - Given information
    - A drip dispersal unit is being considered for treatment of domestic STE with a total N concentration = 50 mg-N/L
    - Site conditions
      - Project site is located in northern Georgia
      - Slope = 3 %, soil = clay loam, vertical $K_S \approx 2.25$ gal/day/ft^2
      - HLR_D based on a LTAR for infiltration ≈ 0.125 gal/day/ft^2
  - Determine
    - The HLR_D based on water and nutrient balances:
      - Water balance HLR_D to limit deep percolation to 10 % of K_S based on climate data and with supplemental irrigation = 0
      - Nutrient balance HLR_D to yield $N \leq 10$ mg-N/L in the deep percolation

12.78

- Solution
  - HLR_D to yield Perc = 10 % of K_S
    - Water balances require climatic data for the project site
      - Percolation was set at 10 % of the HLR_D
      - Average precipitation is calculated using Eq. 12.4

$$\text{Prec} = P_M + 0.85(SD_M) \tag{12.4}$$

      - Potential ET can be calculated for each month using a selected approach/model (e.g., Thornthwaite model)

$$\text{PET} = 1.6L_d \left(\frac{10T}{I}\right)^A \tag{12.6}$$

$$I = \sum_{j=1}^{12} \left[I_j = \frac{(T_j)^{1.514}}{5} \right] \tag{12.7}$$

$$A = 0.000000675\ I^3 - 0.0000771\ I^2 + 0.01792\ I + 0.49239 \tag{12.8}$$

    - Table 12EP.1 presents the weather record data and calculated climatic data

12.79

Table 12EP.1 Climatic data obtained from weather records for the project site and calculations made from that data

Month	Daylight hour (units of 12 h)	Avg. monthly air temp. (°F)	Avg. rainfall (30-year. avg.; 1 S.D.)	Design Prec (in./month)	Heat index (–)	Potential ET (in./month)
Jan	0.87	40.0	5.39; 1.79	6.9	0.84	0.25
Feb	0.85	43.9	4.88; 2.19	6.7	1.53	0.47
Mar	1.03	51.8	6.10; 3.05	8.7	3.30	1.30
Apr	1.09	59.0	4.48; 2.32	6.4	5.28	2.29
May	1.21	66.8	4.20; 1.73	5.7	7.75	3.84
Jun	1.21	74.3	4.20; 1.86	5.8	10.4	5.27
Jul	1.23	78.2	4.44; 1.92	6.1	11.9	6.19
Aug	1.16	77.1	3.83; 1.52	5.1	11.5	5.61
Sep	1.03	71.1	4.07; 2.38	6.1	9.24	3.95
Oct	0.97	59.9	3.24; 2.01	5.0	5.55	2.15
Nov	0.86	50.9	4.46; 1.83	6.0	3.07	1.01
Dec	0.85	42.9	4.49; 2.13	6.3	1.34	041
Total	–	–	53.8	74.8	71.7	32.7
Source	Data from Tables 5–10 and 5–11 in EPRI 2004			Eq. 12.4	Eq. 12.7	Eq. 12.6

12.80

* Using the climate data, the monthly water balance HLR data are calculated using Eq. 12.2 (Table 12EP.2)

$$\mathrm{HLR_D} = (\mathrm{ET} - \mathrm{Prec} - \mathrm{I}) + \mathrm{Perc} \tag{12.2}$$

Table 12EP.2 Water balance calculations for the project site

Month	Days	ET (in./month)[a]	Prec. (in./month)	Perc (in./month)[b]	HLR (in./month)
Jan	31	0.25	6.9	11.2	4.6
Feb	28	0.47	6.7	10.1	3.9
Mar	31	1.30	8.7	11.2	3.8
Apr	30	2.29	6.4	10.8	6.7
May	31	3.84	5.7	11.2	9.3
Jun	30	5.27	5.8	10.8	10.3
Jul	31	6.19	6.1	11.2	11.3
Aug	31	5.61	5.1	11.2	11.7
Sep	30	3.95	6.1	10.8	8.7
Oct	31	2.15	5.0	11.2	8.4
Nov	30	1.01	6.0	10.8	5.8
Dec	31	0.41	6.3	11.2	5.3
Total	365	32.7	74.8	131.4	89.3

[a]Assumes ET = PET. [b]Percolation is limited by design to be 10 % of K_S which is 0.10×2.25 gal/day/ft^2 = 0.36 in./day.

12.81

* The $\mathrm{HLR_D}$ for a water balance is based on the limiting month
  – The water balance reveals that the allowable HLR for March is the lowest and thus would be limiting

    HLR for March = 3.8 in/mon = 0.078 gal/day/ft^2
* The $\mathrm{HLR_D}$ to limit Perc to 10 % of K_S = 0.078 gal/day/ft^2

○ $\mathrm{HLR_D}$ that yields C_P = 10 mg-N/L
* Equations 12.10 and 12.12a are used to determine the $\mathrm{HLR_D}$ that limits the N in the deep percolation to 10 mg-N/L
* Input parameter values are given or selected based on site conditions and assumptions as shown in Table 12EP.3

Note: 1 gal/day/ft^2 = 586.5 in/year.

12.82

Table 12EP.3 Input parameters and values used in the nitrogen mass balance calculations

Parameter	Definition (units)	Value	Basis
ET	Actual evapotranspiration (gal/day/ft^2)	0.055	Climate records (Table 12EP.2)
Prec	Average monthly precipitation (gal/day/ft^2)	0.128	Climate records (Table 12EP.2)
Perc	Deep percolation into the soil profile (gal/day/ft^2)	–	Perc = HLR_D – ET + Prec
HLR_D	Design hydraulic loading rate (gal/day/f^{t2})	–	To be calculated
N_L	N loading via deep percolation (lb/acre/year)	–	N_L = Perc × C_P
C_P	Concentration of N in the deep percolate (mg-N/L)	10	Chosen as a design goal
C_I	Concentration of N in the effluent dispersed (mg-N/L)	50	Characterization data given
N_{WW}	N applied in the effluent disperse based on the HLR_D (lb/acre/year)	–	$N_{WW} = C_I \times HLR_D$
N_{R+F}	N added by rainfall and fixation (lb/acre/year)	5	Chosen based on typical values
N_U	Net plant uptake and storage of N (lb/acre/year)	200	Chosen based on typical values for grasses
N_{DE}	Percent of the applied N ($N_{WW} + N_{R+F}$) that is removed by denitrification (%)	50 %	Chosen as 50 % which is typical for a clay loam soil
N_D	Nitrogen removal by denitrification (lb/acre/year)	–	$N_D = N_{DE}\,(N_{WW} + N_{R+F})$

Note: 1 gal/day/ft^2 = 586.5 in/year 1 in/year = 0.001705 gal/day/ft^2.

12.83

* The nitrogen mass balance is given by Eq. 12.3 and is solved using Eq. 12.12a with input parameter values chosen for N_{DE}, N_U and N_{R+F}

$$N_L = (N_{WW} + N_{R+F}) - (N_U + N_D) \tag{12.3}$$

$$N_L = (HLR_D)(C_I)132.8 + N_{R+F} - N_U - N_{DE}[(HLR_D)(C_I)132.8 + N_{R+F}] \tag{12.12a}$$

$$N_L = (HLR_D)(C_I)132.8 + 5 - 200 - 0.50[(HLR_D)(C_I)132.8 + 5]$$

$$N_L = 0.50(HLR_D)(C_I)132.8 - 197.5$$ Eq. 12.12a for N_{DE} = 50 %

12.84

* Solving for HLR_D with input parameters for ET, Prec and C_I

$$N_L = N_L \qquad \text{Eq. 12.10} = \text{Eq. 12.12b with } N_{DE} = 50\%$$

$$(\text{Perc})(C_P)132.8 = 0.50[(HLR_D)(C_I)132.8] - 197.5$$

$$(HLR_D - ET + \text{Prec})(C_P)132.8 = 0.50[(HLR_D)(C_I)132.8] - 197.5$$

$$(HLR_D - 0.055 + 0.128)(10)132.8 = 0.50[(HLR_D)(C_I)132.8] - 197.5$$

$$(HLR_D)(1328) + 96.9 = 0.50(HLR_D)(6640) - 197.5$$

$$(HLR_D)(1328) = (HLR_D)(3320) - 197.5 - 96.9$$

$$HLR_D = \frac{294.4}{1992} = 0.148\,\frac{\text{gal/day}}{\text{ft}^2} = 0.60\,\frac{\text{cm}}{\text{d}} = 1.66\,\frac{\text{in.}}{\text{week}}$$

- Comparison of HLR_D values (given and calculated)
  * For LTAR: 0.125 gal/day/ft^2
  * For Perc = 10 % of K_S: 0.078 gal/day/ft^2
  * For C_P = 10 mg-N/L: 0.148 gal/day/ft^2

12.85

■ 12EP-2. Determining the minimum drip dispersal footprint area required

- Given information
  - A drip dispersal unit is being considered for treatment of 1000 gal/day of domestic STE with a total N concentration = 50 mg-N/L
  - Site conditions (same as in problem 12EP-1)
    * Project site is located in the southeastern U.S.
    * Slope = 3 %, soil = clay loam, vertical $K_S \approx 2.25$ gal/day/ft^2
    * HLR_D based on a LTAR for infiltration ≈ 0.125 gal/day/ft^2
- Determine
  - The minimum dispersal area required to yield N = 10 mg-N/L in the shallow groundwater under the site
  - Based on the minimum dispersal area calculated, what is the HLR_D that results with a design flow of 1000 gal/day

12.86

- Solution
  - The minimum footprint required (A_{Fm}) can be determined using Eq. 12.14 using given and assumed input parameters

$$A_{FM} = \frac{[C_I(1 - N_{DE}) - 10]HLR_D - 10V_d}{10(Prec + I - ET) + N_U} \quad (12.14)$$

A_{FM} = minimum dispersal footprint area required (m^2) (1 m^2 = 10.76 ft^2)
C_I = nitrogen applied in effluent: given as 50 mg/L = 50 g/m^3
N_{DE} = denitrification (% of applied N): select 50 % for the clay loam
HLR_D = wastewater application rate: given as 1000 gal/day = 1382 m^3/year
N_U = plant nitrogen uptake: assume 200 lb/acre/year = 22.5 g/m^2/year
Prec = rainfall: based on records = 74.8 in./year = 1.9 m/year
I = supplemental irrigation: assume 0 in./year = 0 m/year
ET = evapotranspiration: based on records = 32.7 in./year = 0.83 m/year
V_d = additional dilutive volume: assume 0 m^3/year

12.87

$$A_{FM} = \frac{[C_I(1 - N_{DE}) - 10]HLR_D - 10V_d}{10(Prec + I - ET) + N_U}$$

$$A_{FM} = \frac{[50\ g/m^3(1 - 0.5) - 10\ g/m^3]1382\ m^3/year - 10g/m^3(0\ m^3/year)}{10g/m^3(1.9\ m/year + 0 - 0.83\ m/year) + 22.5\ g/m^2/year}$$

$$A_{FM} = \frac{[15\ g/m^3]1382\ m^3/yr}{10g/m^3(1.9\ m/year + 0 - 0.83\ m/year) + 22.5\ g/m^2/year}$$

$$A_{FM} = \frac{20{,}730\ g/year}{10.7g/m^2/year + 22.5\ g/m^2/year} = \frac{20{,}730\ g/year}{33.2\ g/m^2/year}$$

$$A_{FM} = 624\ m^2 = 6717\,ft^2$$

12.88

- Based on the dispersal area calculated (A_{FM}) and the given design daily flow rate (Q_D), the resulting hydraulic loading rate (HLR_D) can be calculated

$$HLR_D = \frac{Q_D}{A_{FM}}$$

$$HLR_D = \frac{(1000 \ \text{gal/day})}{(624 \ \text{m}^2)\left(\frac{10.76 \ \text{ft}^2}{\text{m}^2}\right)}$$

$$HLR_D = 0.149 \ \text{gal/day/ft}^2$$

- The HLR_D determined here (0.149 gal/day/ft^2) is essentially the same as that calculated in Problem 12EP-1 ($HLR_D = 0.148$ gal/day/ft^2)
- If supplemental irrigation had been included ($I > 0$) in either problem, the HLR_D values would have been >0.149 gal/day/ft^2

12.89

- 12EP-3. Design of a drip dispersal unit
  - Given information
    - Slope <5 %, well-drained, sandy loam soil, $K_S = 24.5$ gal/day/ft^2
    - Average daily flow = 400 gal/day, PF = 2.5
    - Influent = STE with $BOD_5 = 125$ mg/L and TSS = 40 mg/L
    - Dosing and delivery components
      - Dosing tank volume = 2× design daily flow
      - STE delivery and return lines (transport and manifold) = 300 ft each, 1-in. nominal PVC; Δ Elev. between pump and drip system = +10 ft.
      - Pre-filtration = inline disk filter (120 μm)
      - Drip tubing = 125 ft runs, run spacing = 2 ft, emitter spacing = 2 ft, emitter operating pressure = 15 psi
    - Treatment goal: secondary treatment with natural disinfection during infiltration and deep percolation plus some recovery of water and nutrients for turf growth

12.90

- Determine
  - Total footprint area (ft^2) required for drip dispersal based on a LTAR for the soil properties in the dispersal area
  - Total linear feet of drip tubing required (ft)
  - Number of equal size zones, run length and runs per zone, and linear ft of drip tubing per zone for a standard AMC Z451 design (http://www.americanonsite.com/american/manuals/designguide1-zdt.html)
  - Flow rate per zone (in gal/min) for dispersal and for line flushing
  - Total flow the pumping unit must provide
  - Estimate the TDH for the system (assume an AMC 25 gal/min unit)
  - Determine the dosing volume and pump run and rest times for standard flow and peak enable conditions

12.91

- Solution
  - Total footprint area (A_F) and linear feet (LF) of drip tubing required
    - For sandy loam soil and STE select HLR_D based on a LTAR = 0.20 gal/day/ft^2 (refer to Table 12.4)
    - Set spacing between adjacent drip tubing lines (S_T) = 2 ft.

$$A_F = \frac{(Q_D)}{HLR_D} \tag{12.13}$$

$$A_F = \frac{(400 \ \text{gal/day} \times 2.5)}{0.20 \ \dfrac{\text{gal/day}}{\text{ft}^2}}$$

$$A_F = 5000 \ \text{ft}^2$$

$$LF = \frac{A_F}{S_T} \tag{12.15}$$

$$LF = \frac{5000 \ \text{ft}^2}{2 \ \text{ft.}}$$

$$LF = 2500 \ \text{ft.}$$

12.92

- Drip dispersal zones and features
  - Select AMC Z451 layout (Figure 12EP.1)
    - 4 zones with 5 runs per zone at 125 ft per run
    - Total linear feet of drip tubing = 2500 ft.

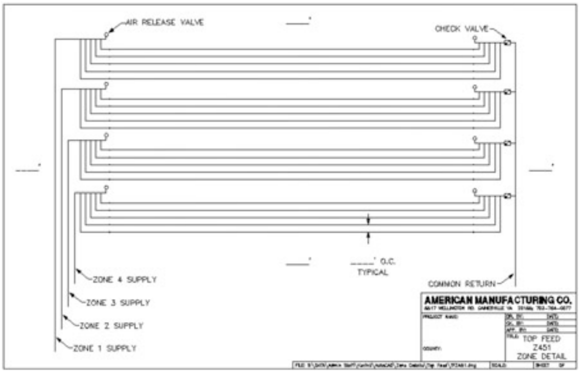


Fig. 12EP.1 Plan view schematic of the AMC Z451 drip dispersal unit layout (www.americanonsite.com/american/manuals/designguide1-zdt.html)

12.93

- Flow rates per zone
  - Flow rate during dispersal in each zone

$$Q_{Dis} = \left(\frac{LF_Z}{S_E}\right)(Q_E) \tag{12.17}$$

$$Q_{Dis} = \left(\frac{625\ \text{ft.}}{2\ \text{ft.}}\right)(0.65\ \text{gal/h})\left(\frac{1\ \text{h}}{60\text{min}}\right)$$

$$Q_{Dis} = 3.39\ \text{gal/min}$$

  - Flow rate for flushing each zone

$$Q_{Flu} = (Q_S)(L_N) \tag{12.18}$$

$$Q_{Flu} = (1.6\ \text{gal/min per lateral})(5\ \text{laterals/zone})$$

$$Q_{Flu} = 8.0\ \text{gal/min}$$

12.94

- Total flow rate per zone
  - Total flow rate for pump sizing occurs during dosing when there is a field flush return flow in addition to dispersal flow

$$Q_P = Q_{Dis} + Q_{Flu} \tag{12.19}$$

$$Q_P = 3.39 + 8.0$$

$$Q_P = 11.4\,\text{gal/min}$$

12.95

- Total dynamic head (TDH)
  - TDH for pump sizing to deliver Q per zone = 11.4 gal/min

$$TDH = (h_s + h_{fu}) + (h_{fs} + h_{fr}) + (h_{fl} + h_r) \tag{12.20}$$

  - TDH = 99.7 ft based on headloss components listed below:

h_s = +10 ft (given)
h_{fu} = 7 ft (lookup an AMC 25 gal/min unit for Q = 11–12 gal/min)
h_{fs} = 20.1 ft (based on Q = 11.4 gal/min to zone with 5 laterals)
h_{fr} = 11.1 ft (based on Q = 8 gal/min from zone with 5 laterals)
h_{fl} = 16.5 ft (based on Q = 1.6 + 0.67 gal/min thru a 125-ft lateral)
h_r = 35 ft (based on emitter discharge at 15 ft psi)

Note: h_{fs}, h_{fr}, h_{fl} were determined using Eq. 12.21:

$$h_{fp} = 10.5(L)\left(\frac{Q}{C}\right)^{1.85}\left(D^{-4.87}\right) \tag{12.21}$$

12.96

- Dosing volume during a dispersal event

$$V_{DE} = \frac{Q_A}{(N_Z)(D_{PD})} \tag{12.22}$$

$$V_{DE} = \frac{400 \text{ gal/day}}{(4 \text{ zones})(4 \text{ doses/day per zone})}$$

$$V_{DE} = 25 \text{ gal/dose}$$

12.97

- Dose timing—pump-on and -off times
  - * Dispersal under average flow conditions[a]

$$P_{on} = \frac{V_{DE}}{Q_{Dis}} \tag{12.23}$$

$$P_{on} = \frac{25 \text{ gal/dose}}{3.39 \text{ gal/min}}$$

$$P_{on} = 7.4 \text{ min/dose}$$

$$P_{off} = \left[\frac{T_D}{N_Z \times D_{PD}}\right] - P_{on} \tag{12.24}$$

$$P_{off} = \left[\frac{1440}{4 \times 4}\right] - 7.4$$

$$P_{off} = 82.6 \text{ min}$$

[a]*Note*: if the daily flow exceeds the average, the number of doses will increase. P_{on} time will stay the same but P_{off} time will be lower.

12.98

Chapter 13

Treatment for Nutrient Reduction

13-1. Scope

Nitrogen and phosphorus can be of special concern in areas that are sensitive to nutrient inputs associated with wastewaters and reclaimed waters (e.g., locations where groundwater is used as a source of drinking water or inland lakes where eutrophication is a concern). In these areas nutrient reduction is often a major requirement for decentralized water reclamation and reuse. Nitrogen and phosphorus can also be of special interest due to their value as a fertilizer or soil amendment. This chapter describes the principles and processes of nutrient reduction and the design and implementation of different strategies and technologies for treatment and also recovery.

13-2. Key Concepts

- Nitrogen (N) and phosphorus (P) in wastewaters can be viewed either as: (1) constituents of concern that need to be removed during treatment due to their potential adverse water quality and human health effects or (2) constituents of special interest that need to be recovered and used due to their resource value.
  - If the project involves discharge of treated wastewater to the environment, concentrations of N and P can be of special concern due to their potential adverse effects including: dissolved oxygen depletion and hypoxia in lakes, rivers, and coastal zones, ammonia toxicity to aquatic life, organic N and inorganic P as contributors to eutrophication in surface waters, and nitrate N in drinking water as a cause for methemoglobinemia. In general, N is of greater concern where

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groundwater or coastal zone water is the receiving environment while P is of greater concern where wastewater is discharged to, or reaches, inland surface waters like rivers and lakes.
- If the project involves water reuse, concentrations of N and P can be of concern or of interest. If water reuse includes use of reclaimed water for nonpotable purposes (e.g., toilet flushing and cooling), removal of N and P may be warranted to minimize biological growth, biofouling and scale formation. However, if water reuse involves use of reclaimed water for irrigation (e.g., of grasses, plants, shrubs, trees), concentrations of N and P can be of special interest due to their value as a fertilizer.
- If the project involves resource recovery, the concentrations of N and P (and other wastewater nutrients plus organic matter) can be of special interest due to their value as a fertilizer or soil amendment.

■ Nitrogen reduction can be accomplished using source separation or different types of treatment operations.
- Source separation can isolate N out of a wastewater. For example, urine diversion using urine diverting toilets (UDTs) can achieve up to about 85 % removal of N and enable its use as a fertilizer.
- Confined treatment units and natural system operations can remove N from a wastewater. Primary reactions include nitrification (autotrophic bioconversion of NH_4^+ to NO_3^-) and denitrification (heterotrophic or autotrophic bioconversion of NO_3^- to N_2). These reactions can be implemented in different types of treatment units, which will have different requirements for power, chemicals, residuals management, operation and maintenance, and costs.
  - ○ Treatment unit operations normally used for removal of other constituents like BOD_5, TSS, and NH_4^+ can include design modifications to enhance N removal efficiencies (e.g., aerobic treatment units, porous media biofilters, membrane bioreactors). For example, high removals of total N (e.g., 50–70 %) can be achieved by strategic sequencing of anaerobic and aerobic zones and/or by recirculation of nitrified effluent back to an anoxic zone where there is available organic carbon.
  - ○ Appreciable N removal can occur in unit operations such as constructed wetlands, subsurface soil treatment units or landscape drip dispersal units. The extent to which N removal occurs depends on site conditions and system design and implementation. For example, for constructed wetlands cold temperatures (<10 °C) can inhibit nitrification reactions and in general the wetland surface area required for nutrient removal is much larger than that required for BOD_5 and TSS removal (e.g., 2.5$\times$ larger). For subsurface soil treatment units and landscape drip dispersal units, low removals of N can occur in soil profiles with coarse-

grained textures and shallow groundwater (e.g., <20 %). In contrast, very high removals of N can be achieved in soil profiles with finer-grained textures, deeper depths to groundwater and assimilation into the associated groundwater system (e.g., >70 %).

- If very high levels of N reduction are required before release of a treated wastewater to the environment (e.g., discharge to subsurface soils, landscape surfaces, or surface waters), specialized confined treatment units can be deployed within decentralized systems.
  - A prominent nutrient reduction unit to remove NO_3^- (e.g., in the effluent produced by a nitrification unit) involves denitrification in an anoxic biofilter established with organic media (e.g., wood chips) or inorganic media (e.g., beads of elemental sulfur). These units can achieve >95 % removal of N with effluent concentrations of <3 mg-N/L. These biofilters also appear able to reduce bacteria levels substantially (e.g., to <200 fecal coliforms/100 mL).

- Phosphorus reduction can also be accomplished using source separation or different types of treatment operations.
  - Source separation can isolate P out of a wastewater. For example, urine diversion using UDTs can achieve up to 50 % removal of P and enable its use as a fertilizer.
  - Confined treatment units and natural system operations can remove P from a wastewater. Primary reactions involved in P removal include sorption and precipitation of PO_4^{-3}. These reactions can be implemented in different types of treatment technologies, which will have different requirements for power, chemicals, residuals management, operation and maintenance, and costs.
    - Significant P removal can occur in unit operations such as constructed wetlands, subsurface soil treatment units or landscape drip dispersal units. The extent to which P removal occurs depends on site conditions and system design and implementation. For example, for constructed wetlands the wetland surface area required for P removal is larger than that required for BOD_5 and TSS removal. For subsurface soil treatment units and landscape drip dispersal units, low removals of P can occur in soil profiles with coarse-grained textures and shallow groundwater (e.g., <20 %). In contrast, very high removals of P can be achieved in soil profiles with finer-grained textures, deeper depths to groundwater and assimilation into the associated groundwater system (e.g., >90 %).
    - Treatment technologies are available for use in decentralized systems that are specifically designed to achieve high levels of P reduction. For example, porous media biofilters are commonly used in decentralized systems and they can be designed using

P-sorbent media to achieve ≥99 % removal of total P. While more complicated and difficult to implement in decentralized systems, chemical or biological methods (e.g., chemical precipitation and solids separation or enhanced biological P removal, respectively) can achieve high levels of P removal and produce effluent P concentrations of <0.10 mg-P/L.

- If very high levels of P reduction are required before release of a treated wastewater to the environment (e.g., discharge to subsurface soils, landscape surfaces, or surface waters) or for a reuse purpose, specialized confined treatment units can be deployed within decentralized systems.
    - A prominent nutrient reduction unit to remove soluble phosphorus (orthophosphate, PO_4^{-3}) involves media sorption/precipitation in P-sorptive media filters (e.g., with light weight clay aggregate). These units can achieve ≥99 % removal of total P but have a finite run length depending on media characteristics and filter properties. Once saturated with P, the media can be replaced.

■ Where nutrient recovery is desired so N or P can be used as a fertilizer or soil amendment, several source separation and treatment options can be considered.

- Source separation can isolate N and P from a wastewater. For example, urine diversion can enable recovery of up to about 85 % and 50 % of the N and P in domestic wastewaters, respectively. After processing, this diverted urine can be used as a fertilizer under certain conditions.
- Wastewater treatment in a confined unit operation that doesn't normally remove N or P can be used to produce an effluent suitable for irrigation purposes with respect to public health (i.e., pathogens) and water quality (e.g., concentrations of N and P for fertilizer value with an acceptable pH and salt content). For example, a membrane bioreactor (MBR) could produce a very high quality effluent without high N or P removal so the MBR effluent could be beneficially used as an irrigation water.
- Wastewater treatment in a confined unit operation that removes nutrients can be designed to produce a nutrient-rich residual that is suitable for use as a fertilizer. For example, this could be a biofilter with P-sorptive media that can be removed from the filter when the media is saturated with P and then used as a slow release source of P as a fertilizer for agronomic applications.
- Wastewater treatment in a landscape drip dispersal unit can yield fertilizer benefits. For example, while treatment also achieves removal of other constituents of concern such as BOD_5 and

pathogens, N and P can fertilize plant growth and help reduce or eliminate use of chemical fertilizer amendments.
- Where N or P removals are based in part, or entirely, on nutrient uptake by plants, harvesting and removal of plants from the site can be needed to accomplish what may be considered permanent removal of nutrients.

■ When larger numbers of decentralized systems are applied at a development or watershed scale, concerns can be focused on N or P fate and their effects on groundwater and surface waters.
- Particular concerns that are often confronted include: (1) NO_3^- loadings to groundwater which may be used as a source of drinking water or is connected to sensitive surface waters, (2) PO_4^{-3} loadings to inland surface waters, and (3) N and P loadings to estuaries and coastal waters.
- Concerns often arise where there are decentralized systems that release treated effluent that still contains N and P to the land surface or subsurface environment and count on nutrient attenuation as water is reclaimed and assimilated into a local hydrologic system. An example of this situation occurs when there are large numbers of soil-based systems in a given geographic area. Water quality loadings and effects are dependent on the location and circumstances and a valid environmental assessment requires a careful site-specific evaluation.

13-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 13 are presented in the Slides section.

13-4. Terminology

Terminology introduced and used in Chap. 13 is defined below.

Activated sludge—A biological process where microorganisms are grown under aerobic conditions using organic matter in the influent wastewater as a source of food and energy.

Aeration zone—A term that describes the physical system within which active aeration occurs. Examples include an aeration chamber or compartment in a larger tank, a stand-alone tank of basin devoted to aeration, and so forth.

Aerobic—Refers to a biochemical state where microorganisms require oxygen to survive and function by using oxygen as an electron acceptor.

Anaerobic—Refers to a biochemical state where microorganisms do not require oxygen and utilize organic matter or hydrogen as electron donors and inorganic (e.g., nitrate, sulfate) or organic matter as electron acceptors. Some anaerobic organisms may react negatively or even die if oxygen is present.

Anammox—The name of a biological process that involves the simultaneous oxidation of ammonia nitrogen combined with denitrification of nitrite nitrogen.

Assimilation—Refers to the ability of subsurface soil and groundwater to accept and integrate water reclaimed from wastewater treated in a land-based treatment operation into the hydrologic cycle.

Attenuation—Refers to a set of soil and groundwater processes (e.g., biological and chemical reactions along with dilution and dispersion) that can reduce the concentrations of constituents of potential concern in water as it moves from a depth below a soil-based treatment operation (e.g., subsurface soil treatment unit or landscape drip dispersal unit) and recharges groundwater and moves away from the recharge location.

Autotrophic—Refers to a group of microorganisms that use an inorganic material as an electron donor (e.g., elemental sulfur) and acceptor (e.g., nitrate nitrogen).

Complexation—In water chemistry complexation refers to a chemical process that involves the combination of individual atom groups, ions or molecules to create one large ion or molecule. Complexation can also involve reactions that occur at surfaces that carry a charge that depends on pH and composition of the solution. The charge enhances sorption of ions with a charge opposite to the surface and repels ions with the same charge as the surface. Complexation of phosphate to aluminum or ferric oxides and hydroxides can contribute to P removal from wastewater by soil-based and chemical treatment operations.

Confined unit operation—Refers to treatment units that can be established in containers (e.g., a tank or basin) and can be isolated from the effects of environmental processes such as precipitation, evaporation, and temperature fluctuations. Aerobic treatment units, porous media biofilters, and membrane bioreactors are examples of confined unit operations.

Development scale—Refers to a geographic location where larger numbers of decentralized systems are used and there is potential for cumulative effects on groundwater or surface water quality.

Enhanced biological nutrient removal (EBNR)—Refers to biological treatment systems that are specifically designed to achieve high levels of N or P removal.

Ion exchange—A process that involves the exchange of ions between a solution and a solid polymer or mineral resin.

Lightweight aggregate (LWA)—Lightweight aggregate is made by heating clay to a high temperature (e.g., 1200 °C) in a rotary kiln causing gases to expand the clay and form a microporous structure when cooled. LWAs have a high phosphorus sorption capacity (PSC) and can be used as a reactive porous media in constructed wetlands and phosphorus removal filters. LWA can be produced in different spherical size ranges (e.g., 0.1–4 mm, 4–10 mm diameters). Forms of LWA are manufactured in several countries and carry trade names such as LECA® or Filtra P.

Methaemoglobinemia—A blood disorder that involves the presence of an elevated level of methemoglobin, a form of hemoglobin, that is useless for carrying oxygen in a human body. Since hemoglobin is the key carrier of oxygen in the blood, its replacement by methemoglobin can cause a slate gray-blueness of the skin (cyanosis) and potentially cause more serious symptoms due to insufficient oxygen.

Natural system unit operation—Refers to treatment systems that involve natural processes and are typically open in the environment and rely on natural environmental processes for wastewater treatment and water reclamation. Constructed wetlands, subsurface soil treatment units, and landscape drip dispersal units are examples of natural systems.

Phosphorus sorption capacity (PSC)—Refers to the ability of a porous media to remove phosphorus from wastewater or other impaired waters by sorption processes. The PSC is typically expressed in terms of the weight of P sorbed to a unit weight of dry porous media (e.g., 1 g-P per kg media dry wt.).

Point of compliance—Refers to the location in space associated with a decentralized system(s) where a water quality criteria must be satisfied (e.g., NO_3^- N concentration $\leq$10 mg-N/L). An example point of compliance is the effluent discharged from a confined unit operation such as a textile biofilter. Another example for a soil-based treatment operation is the groundwater quality measured in a groundwater observation well placed at the downgradient property line or in the groundwater as it reaches the edge of a local stream.

Polonite®—A media ($CaO \cdot SiO_2$, $CaSiO_3$) that is derived from mining and processing calcium silicate rock and has a high porosity and specific surface area. It has a high phosphorus sorption capacity and has been used in phosphorus sorptive filters.

Precipitation—(1) Refers to forms of water (e.g., rain, sleet, snow) that fall from the sky toward the land surface. (2) A chemical process that involves reactions where a dissolved substance is removed from solution by conversion to a solid substance that can be physically separated from the solution. An example of chemical precipitation involves the removal of phosphate from solution by addition of lime ($Ca(OH)_2$) to create a hydroxyapatite solid ($Ca_{10}(PO_4)_6(OH)_2$) that forms when lime is used to raise the pH >10.

Sorption—A general term used to refer to the process or processes that cause a substance in solution to become attached to a solid. Sorption is generally used to include absorption where a substance is incorporated into another substance (e.g., NH_3 gas absorbed into a basic solution) and adsorption where a substance is bound to the surface of another phase (e.g., PO_4^{-3} adsorbed to a soil mineral surface).

Stage—In wastewater and water treatment, can refer to a major component in a treatment train (e.g., first stage is primary treatment and a second stage is secondary treatment) or to parts of a component (e.g., a stage could be a single sequence of aerobic and anaerobic zones within a biological treatment component).

13-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 13 are listed below.

Al	Aluminum
BFS	Blast furnace slag
BOD_5	Five-day biochemical oxygen demand
CDPHE	Colorado Department of Health and Environment
CSM	Colorado School of Mines
D&I	Design and implementation
DN	Denitrification
DO	Dissolved oxygen
DU	Dwelling unit
d.w.	Dry weight
EAF	Electric arc furnace slag
EBNR	Enhanced biological nutrient removal
EBPR	Enhanced biological phosphorus removal
Fe	Iron
GIS	Geographic information system
HLR	Hydraulic loading rate
HRT	Hydraulic retention time
LECA	Lightweight expanded clay aggregate
MGD	Million gallons per day
N	Nitrogen or an empirical parameter
NO_X	Sum of $NO_2^- + NO_3^-$
O&M	Operation and maintenance
P	Phosphorus
PAO	Phosphorus accumulating organisms
PSC	Phosphorus sorption coefficient

ST	Septic tank or system type
STE	Septic tank effluent
STU	Soil treatment unit
STUMOD	Soil treatment unit model
T	Time
TIN	Total inorganic nitrogen
TKN	Total Kjeldahl nitrogen
TN	Total nitrogen
TP	Total phosphorus
TSS	Total suspended solids
WARMF	Watershed analysis risk management framework model

A_{BS}	Cross-sectional surface area of the biofilter
A_{FS}	Area of the filter surface
C	Concentration of solute in solution in equilibrium with the mass sorbed onto the solid
C_E	Concentration in the effluent
C_I	Concentration in the influent
C_{GW}	Concentration in groundwater
C_{PW}	Concentration in soil pore water
D_B	Depth of the biofilter bed
d_F	Diameter of the filter vessel
F	Conversion factor
G	Mixing velocity gradient
K	Distribution coefficient for sorption
K_D	Linear distribution coefficient for sorption
K_S	Saturated hydraulic conductivity
Q	Daily flow rate (design or actual)
Q_D	Design daily flow rate
Q_{VLR}	Flow volumetric loading rate
M_{SM}	Mass of filter media
M_{VLR}	Mass volumetric loading rate
n_e	Effective porosity
N_{LS}	Nitrogen loading to the edge of a surface water
N_s	Nutrient load from a source (e.g., house)
R_E	Reduction efficiency (%)
R_{NR}	Rate of NO_3^- removal
R_{TU1}	Fractional removal of N or P in a 1st treatment unit (e.g., septic tank)
R_{TU2}	Fractional removal of N or P in a 2nd treatment unit (e.g., sand filter)
R_{STU}	Fractional removal of N or P in a soil-based treatment unit (e.g., infiltration trenches)
S	Mass of solute sorbed per mass of media (mg/kg)

S&GA	Fractional removal of N or P in subsurface soil and groundwater by attenuation during water movement from the STU boundary to the edge of stream
V	Volume of effluent processed at media saturation
V_B	Total volume of the biofilter
$V_{B'}$	Empty bed volume for flow in the biofilter
V_{SM}	Volume of filter media required
v/v	Volume per volume
α	Empirical parameter
β	Empirical parameter

13-6. Problems

13.1. Removal of nutrients from reclaimed water can be important for several reasons. Give two reasons.

13.2. Which nutrient—nitrogen or phosphorus—is most commonly the constituent of concern related to contamination of groundwater used for drinking water?

13.3. Would you expect that substantial removal of nitrogen would occur in a cesspool installed in the Florida Keys where there is little or no depth of unsaturated soil between the bottom of the cesspool and the shallow groundwater?

13.4. Compared to nitrogen, is phosphorus more likely to be removed during wastewater attenuation and assimilation in subsurface soil and groundwater? If yes, what processes are involved in the removal?

13.5. Source separation using urine diverting toilets (UDTs) can remove what percentage of the mass of nitrogen and phosphorus contained in the annual per capita contributions to wastewater?

13.6. Briefly explain how phosphorus can be removed from reclaimed water by biological processes.

13.7. Briefly explain why predicting the removal of total nitrogen in soil during subsurface infiltration is complicated.

13.8. A denitrifying biofilter is being designed to treat recirculating textile filter effluent ($Q_D = 1000$ gal/day, $BOD_5 = 20$ mg/L, TSS $= 20$ mg/L NO_3^- N $= 40$ mg-N/L. The denitrifying biofilter will be established using saturated up flow through a bed of wood chips ($K_S = 325$ ft/day, $n_e = 0.6$). If the goal is to produce an effluent with NO_3^- N ≤ 3 mg-N/L and the nitrate removal rate $= 0.4$ lb/day per 1000 ft^3, what is the required total volume of the biofilter (in ft^3) and what is the hydraulic retention time in the biofilter (days)?

13.9. Nutrient reduction can be important to design of decentralized systems being installed in sensitive areas. Select which nutrient removal strategy or treatment technology from the list given below (a to e) is most likely capable of reducing the total inorganic N to $\leq$3 mg-N/L and one that is most likely capable of reducing the total P to $\leq$1 mg-P/L.

(a) Septic tank effluent treated by a soil treatment unit.
(b) Aerobic treatment unit effluent treated by a recirculating textile media filter.
(c) Septic tank effluent treated in an intermittent sand filter followed by a saturated upflow biofilter packed with beads of sulfur.
(d) Aerobic treatment unit effluent treated in a packed bed filter containing sorptive light expanded clay aggregate.
(e) Urine diversion and recovery for processing and use as a fertilizer.

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Slides of Chapter 13

Decentralized Water Reclamation

Chapter 13: Treatment for Nutrient Reduction

Contents

13.1

13-1. Introduction

- Wastewaters contain nitrogen (N) and phosphorus (P)
  - N and P are critical nutrients that can occur in particulate and soluble forms that can be biodegradable or non-biodegradable
  - Table 13.1 lists the forms of N and P and terms commonly used

Table 13.1 Forms and terms of expression for nitrogen and phosphorus in water and wastewater

Nutrient	Term	Consists of
Nitrogen	Total N	Organic N + inorganic N
	Total inorganic N (TIN)	$NH_4^+ + NO_2^- + NO_3^-$
	Total Kjeldahl N (TKN)	Organic N + NH_4^+
	NOx	$NO_2^- + NO_3^-$
Phosphorus	Total P	Organic P + inorganic P
	Total inorganic P	Orthophosphate + polyphosphate
	Orthophosphate	PO_4^{-3} and H_3PO_4, $H_2PO_4^-$, HPO_4^{-2}
	Polyphosphate	Condensed phosphates (e.g., triphosphate ($P_3O_{10}^{5-}$))

13.2

- Wastewater N and P can be viewed in two distinct ways
  - Nutrients as constituents of concern
    - Nutrients in wastewater can cause adverse effects, including:
      - Depletion of DO in water causing hypoxia and fish kills
      - NH_3 toxicity to aquatic life in surface waters
      - Methaemoglobinemia caused by NO_3^- in drinking water
      - Water quality effects of N inputs to estuaries, marine waters and other sensitive waters
      - P as a limiting nutrient for eutrophication in inland waters
    - Examples of situations where nutrients can be of concern
      - N inputs to groundwater potentially used for drinking water
      - N and P inputs to inland surface waters
      - N inputs to inland springs
      - N inputs to coastal zones and estuaries

13.3

- Nutrients as constituents of value
  - Nutrients are needed to support plant growth and for this purpose, chemical fertilizers have been widely used
    - Nitrogen for commercial fertilizers is based on industrial processing
      - Production and use of N compounds in fertilizers is energy intensive and costly and can contribute to greenhouse gas emissions
    - Phosphorus for commercial fertilizers is based on mining of phosphate ores
      - Production is energy intensive and sources of phosphate are diminishing
  - Nutrients recovered from wastewater represent a potentially valuable alternative to commercial chemical fertilizers

13.4

- Nutrient reduction strategies and unit operations
  - Different strategies and unit operations can be used to remove N and P from wastewaters and treated effluents and, in some cases, to beneficially recover and reuse the nutrients
    - Source separation can isolate N and P out of wastewater
      - Reduces the concentrations in the wastewater being treated
      - Enables processing and use of the N and P as fertilizer
    - Treatment unit operations can remove N and P from wastewater
      - Reduces concentrations in the effluent discharged or reused
      - Enables recovery as fertilizer for irrigation purposes
  - A classification scheme for nutrient reduction strategies and unit operations is presented in Fig. 13.1 with brief descriptions provided in Table 13.2

13.5

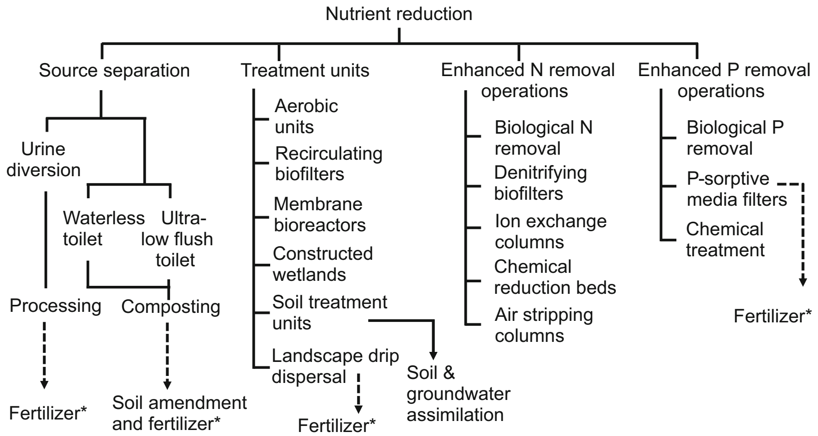


Fig. 13.1 Classification of nutrient reduction strategies and technologies (*Note:* An "*" denotes a potential fate and recovery path for the nutrients removed)

13.6

Table 13.2 Nutrient reduction strategies and unit operations applicable to decentralized systems

Nutrient reduction	Description	Nutrient recovery?
Urine diversion	Urine diverting toilets are connected to a collection basin	Yes—Urine can be processed and used for fertilizer
Blackwater management	Blackwater (feces plus urine) can be separated with dual plumbing	Yes—Blackwater can be composted or otherwise processed
Aerobic units, membrane bioreactors, porous media biofilters	These unit operations are often not designed to achieve enhanced nutrient removal but they can have some incidental removal of total N and P	Maybe—if treated effluents are used for irrigation, the N and P can provide fertilizer benefits
Constructed wetlands	Constructed wetlands can be designed to achieve partial N and P removal	Yes—if vegetation is harvested and used as a soil amendment
Soil-based treatment	N and P can be removed in soil the extent of which depends on soil and site conditions	Yes—N and P can be taken up by grasses and plants
Enhanced biological N and P removal	Biological treatment can be specifically designed for enhanced N and P removal	Not likely—possibly some recovery in biosolids that are produced
Denitrifying biofilters	Porous media biofilters for biological N removal	No—not really feasible
P-sorptive media filters	Porous media filters with high P sorption	Maybe—Media used as a fertilizer
Chemical treatment	Chemicals can be added to one or more locations in a treatment train to remove P	Maybe—possible depending on use of the P-rich sludge produced

13.7

13-2. Treatment Performance

- Approaches to assessing nutrient reduction performance
  - The performance assessment approach depends on the type of nutrient reduction strategy or unit operation deployed, e.g.,:
    - Source separation can isolate N and P out of an effluent
      - An approach is to compare the N or P concentration or mass in the wastewater after source separation to that present before
    - Treatment technologies can remove N and P from wastewater
      - Confined unit operations (e.g., membrane bioreactor)
        - An approach is to compare the N or P concentration or mass in the effluent from the unit to the influent to it (e.g., Fig. 13.2)
      - Land-based unit operations (e.g., soil treatment unit)
        - An approach is to compare the N or P concentration or mass flux in the soil pore water or groundwater at some location to the effluent dispersed (e.g., Fig. 13.3)

13.8

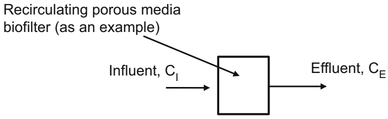


Fig. 13.2 Cross-section view of a confined unit operation to illustrate the typical approach to determine treatment efficiency

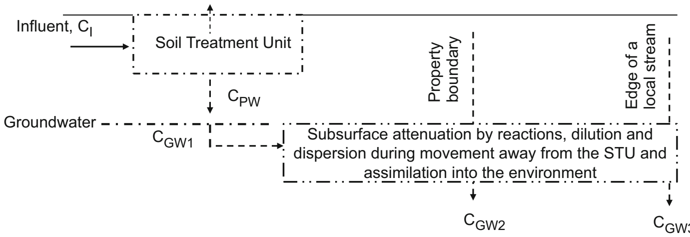


Fig. 13.3 Cross-section view of a site to illustrate different locations where a point of compliance could be set and used to determine treatment efficiency

13.9

- Nutrient reduction efficiency (R_E)
  - Equations 13.1 and 13.2 can be used to determine nutrient reduction efficiency for a confined unit operation and land-based unit operation, respectively (see Figs. 13.2 and 13.3)

$$R_E = \left[\frac{C_I - C_E}{C_I}\right] \times 100 \ \% \tag{13.1}$$

$$R_E = \left[\frac{C_I - (C_{PW} \ \text{or} \ C_{GW})}{C_I}\right] \times 100 \ \% \tag{13.2}$$

Where:
R_E = reduction efficiency (%)
C_I = concentration or mass of N or P in the influent (mg/L or g/day)
C_E = concentration or mass of N or P in the effluent (mg/L or g/day)
C_{PW} = concentration of N or P in soil pore water at a location (mg/L or g/day)
C_{GW} = concentration of N or P in groundwater at a location (mg/L or g/day)

13.10

- Examples of achievable N and P reductions appear in Table 13.3

Table 13.3 Nutrient reduction efficiencies achievable with several technologies

Nutrient reduction	Nitrogen[a]		Phosphorus[a]		Comments
	Removal (%)	C_E (mg/L)	Removal (%)	C_E (mg/L)	
Urine diversion	85 %	9	50 %	5	Assumes 100 % urine diversion and processing
Aerobic units	20–50 %	30–48	10–30 %	7–9	Typ. suspended growth or fixed film ATU
Recirculating biofilters	45 to 70 %	18–33	–[b]	10	Sand and synthetic media for BOD and TSS removal
Membrane bioreactors	50–85 %	9–30	30–90 %	$\leq$1–7	Removals depend on MBR operating parameters
Constructed wetlands	20 %	48	–	10	Wetlands designed for BOD and TSS removal
Soil infiltration/ dispersal	20–70 %	18–48	$\geq$99 %	$\leq$0.1	Removals depend on conditions and do not include removal in groundwater
N removal biofilters	95 %	3	–	10	Designs with bionitrification and bio denitrification
P sorptive filter units	–	60	$\geq$90 %	$\leq$1	Filter designs using P-sorptive media
Chemical treatment	–	–	>90 %	<0.1	Depends on design and operation

[a]Influent N = 60 mg-N/L, P = 10 mg-P/L.
[b]"–" = negligible removal for common system designs.

13.11

13-3. Principles and Processes

- Nutrients handled in decentralized systems
  - Different nutrient species can be present
    - Raw waste and wastewaters
      - Nitrogen is typically present as organic N and NH_4^+
      - Phosphorus is typically present as organic P and ortho PO_4^{-3}
    - Septic tank effluent (STE)
      - Most influent organic N is converted to NH_4^+
        - There still can be 5–20 % organic N in STE
      - Influent polyphosphates are converted to PO_4^{-3}
    - Aerobic treatment unit or porous media biofilter effluents
      - Most of the influent NH_4^+ is converted to NO_2^-/NO_3^-
        - Conversion % depends on design and operation
      - Influent PO_4^{-3} remains largely unchanged

13.12

- Representative nutrient concentrations
  - Diverted urine (diluted 1:2 with flush water)
    - Total N = 2000 mg-N/L
    - Total P = 200 mg-P/L
  - Septic tank effluent from residential sources
    - Typical total N = 60 mg-N/L
    - Typical total P = 10 mg-P/L
  - Septic tank effluent from nonresidential sources
    - High N sources—Rest areas, campgrounds, schools, etc.
      - Total N can be 150 mg-N/L or higher
    - High P sources—Laundries, rest areas, etc.
      - Total P can be 40 mg-P/L or higher

13.13

- Primary reactions involved in N removal
  - Nitrogen in wastewater can participate in a variety of reactions
    - Biological uptake of N
    - Biological nitrification of NH_4^+ to NO_2^- and NO_3^-
    - Biological denitrification of NO_2^- or NO_3^- to N_2O and N_2
    - Sorption of NH_4^+ to surfaces
    - Volatilization of NH_3 at elevated pH
    - NO_3^- anion exchange
    - NO_3^- chemical reduction
  - These reactions can occur in natural environments or in treatment units that have different levels of design complexity, power and chemical use, operation and maintenance (O&M) needs, and costs
    - For decentralized systems, biological nitrification and denitrification occur most frequently with the greatest effects

13.14

- Nitrification: Autotrophic bioconversion of NH_4^+ to NO_3^-
  - Nitrification is carried out under oxic conditions by autotrophic bacteria that utilize O_2 as an electron acceptor and CO_2 as a carbon source
  - Nitrite formers (Eq. 13.3) include bacteria in the ammonia oxidizing group

$$NH_4^+ + 1.5O_2 \Rightarrow NO_2^- + 2H^+ + H_2O \tag{13.3}$$

  - Nitrate formers (Eq. 13.4) include bacteria in the nitrite oxidizing group

$$NO_2^- + 0.5O_2 \Rightarrow NO_3^- \tag{13.4}$$

  - Equation 13.5 is the overall biooxidation of ammonia to nitrate

$$NH_4{}^+ + 2O_2 \rightarrow NO_3^- + 2H^+ + H_2O \tag{13.5}$$

13.15

  - Ammonia can also be incorporated into bacterial cells according to Eq. 13.6

$$4CO_2 + HCO_3^- + NH_4^+ + H_2O \rightarrow C_5H_7O_2N + 5O_2 \tag{13.6}$$

  - Equation 13.7 is an overall equation[a] for nitrification of NH_4^+ to NO_3^-

$$\begin{aligned} &NH_4^+ + 1.731O_2 + 1.962HCO_3^- \rightarrow \\ &0.038C_5H_7O_2N \;+\; 0.962NO_3^- + 1.077H_2O + 1.769H_2CO_3 \end{aligned} \tag{13.7}$$

  - According to Eq. 13.7, for each g of NH_4^+ (as N) converted:
    * 3.96 g of O_2 are used
    * 0.31 g of new cells (as $C_5H_7O_2N$) are formed
    * 7.01 g of alkalinity (as $CaCO_3$) are consumed

[a] *Source:* Crites and Tchobanoglous (1998).

13.16

- Denitrification: Heterotrophic bioconversion of NO_3^- to N_2 gas
  - Denitrification is carried out under anoxic conditions (DO < 0.5 mg/L) by heterotrophic bacteria (different anaerobic and facultative bacteria) that utilize organic matter for energy and C
  - Equation 13.8 shows the conversion of N species while Eq. 13.9 shows NO_3^- removal when methanol is used as a C source[a]

$$NO_3^- \rightarrow 2NO_2^- \rightarrow 2NO \rightarrow N_2O \rightarrow N_2 \tag{13.8}$$

$$NO_3^- + 1.183CH_3OH + 0.273H_2CO_3 \rightarrow \tag{13.9}$$
$$0.091C_5H_7O_2N + 0.454N_2 + 1.820H_2O + HCO_3^-$$

  - According to Eq. 13.9, for each g of NO_3^- (as N) converted:
    * 2.70 g of methanol (CH_3OH) are utilized
    * 0.74 g of new cells are formed
    * 3.57 g of alkalinity (as $CaCO_3$) are formed

[a]*Source*: Crites and Tchobanoglous (1998).

13.17

- Denitrification: Autotrophic bioconversion of NO_3^- to N_2 gas
  - Denitrification can be carried out under anoxic conditions by autotrophic bacteria (*Thiobacillus denitrificans* and *Thiomicrospira denitrificans*) that can use sulfur as an electron donor and NO_3^- as an electron acceptor
  - Equation 13.10 shows the overall equation for NO_3^- removal using S as a electron donor[a]

$$55S^o + 20CO_2 + 50NO_3^- + 38H_20 + 4NH_4^+ \rightarrow \tag{13.10}$$
$$4C_5H_7O_2N + 55SO_4^{-2} + 25N_2 + 64H^+$$

  - According to Eq. 13.10, for each g of NO_3^- (as N) converted:
    * 2.5 g of SO_4^{-2} (as S) are generated
    * 0.64 g cells are formed
    * 4.5 g alkalinity (as $CaCO_3$) are consumed

[a]*Source*: Sengupta and Ergas (2006).

13.18

- Characteristics of nitrification and denitrification reactions
  - The basic biochemistry is summarized in Table 13.4
  - The factors affecting N removal process function and efficiency are summarized in Table 13.5
  - Table 13.6 shows other reactions and processes for N removal

Table 13.4 Basic biochemistry features of nitrification and denitrification reactions

Factor	Nitrification	Denitrification	
Bacteria responsible	Autotrophs	Heterotrophs	Autotrophs
Oxygen	Requires O_2 (>1 mg/L)	O_2 must be absent	
Carbon source	Inorganic	Organic	Inorganic
Electron donor (typ.)	NH_4^+	Organic	Sulfur
Electron acceptor (typ.)	O_2	NO_2^-/NO_3^-	
Alkalinity consumption or production	7.0 g consumed (as $CaCO_3$) per g NH_4^+-N converted	3.57 g produced (as $CaCO_3$) per g NO_3^--N converted	4.5 g consumed (as $CaCO_3$) per g NO_3^--N converted

13.19

Table 13.5 Factors affecting nutrient removal processes and efficiency achieved

Factor	Effect on nitrification	Effect on denitrification
Concentration of ammonium and nitrite	Affects growth rate of nitrifying organisms; conversion of ammonia to nitrite is the rate limiting step	Not applicable
Concentration of nitrate	Not applicable	At low NO_3^- affects growth rate of organisms responsible for denitrification
Concentration of carbon	Not applicable	Generally controls the rate of denitrification
BOD_5/TKN ratio	Affects the fraction of microorganisms that are nitrifiers	Affects are system dependent
DO level	DO > 1 mg/L is critical	DO < 0.5 mg/L is critical
Temperature	<10 °C much lower growth; optimum growth at 25–30 °C	Optimum growth at 25–35 °C
pH	Optimal growth at pH 7.5–8.6	Optimal growth at pH 6.5–7.5
Biotoxics	Can inhibit biological rxn.; Quaternary ammonium salts are notably bad	
Time	Transformation and removal reactions are time dependent	

13.20

Table 13.6 Less common reactions and processes for nitrogen removal

Reaction/ process	Description	Comments	References
Anammox	Simultaneous anaerobic oxidation of NH_4^+ and denitrification of NO_2^- by autotrophic bacteria	The autotrophs reduce the NO_2^- to N_2 gas while utilizing the O_2 from the NO_3^- to oxidize the NH_4^+ to NO_3^- No organic carbon is required	Jetten et al. (1997, 1999) and Smith et al. (2008)
Air stripping of ammonia	Conversion of NH_4^+ to NH_3 gas at high pH (e.g., > 9; pKa $= 9.3$)	Requires elevated pH, so after stripping, pH adjustment is needed	Antonini et al. (2011)
Sorption of ammonium	Use of granular activated carbon, zeolites, clay minerals or similar media that can adsorb NH_4^+	Sorption capacities can range from: GAC = 54 g/kg Sepiolite = 63–67 g/kg Zeolite = 1–10 g/kg	Ji et al. (2011)
Nitrate chemical reduction	Use of zero valent iron (Fe^0) filings in a packed bed to chemically reduce NO_3^-	Can function for NO_3^- removal but generation of NH_4^+ can occur and cause concerns	Cheng et al. (1997) and Ji et al. (2011)
Nitrate anion exchange	Use of nitrate selective polymer beads in a column to exchange NO_3^- ions for Cl^- ions	Requires a very clean water; Removal efficiencies of nearly 100 % can be obtained, but NO_3^- selective resin is extremely expensive	ResinTech SIR-100-HP

13.21

- Primary reactions involved in P removal
  - Phosphorus in wastewaters can participate in a variety of reactions:
    - Biological uptake of P
    - Sorption of PO_4^{-3} to surfaces
    - Mineral formation from PO_4^{-3} sorbed to surfaces
    - Precipitation/complexation of PO_4^{-3} out of solution
    - Struvite precipitation of PO_4^{-3} out of solution
  - These reactions can occur in natural environments and in treatment units that have different levels of design complexity, power and chemical use, O&M needs, and costs
    - For decentralized systems, sorption, mineral formation, and precipitation occur most frequently with the greatest effects

13.22

- Sorption of PO_4^{-3}
  - Phosphate can sorb to surfaces that have a positive charge at typical pH values (e.g., Fe and Al metal-oxides)
  - Sorption can be described by isotherms
  - A linear isotherm, typically applicable for $P < 10$ mg/L, is shown in Fig. 13.4 and described by Eq. 13.11

$$S = K_D C \tag{13.11}$$

Where:
S = mass of solute sorbed per mass of media (mg/kg)
C = concentration of solute in solution in equilibrium with the mass sorbed onto the solid (mg/L)
K_D = linear distribution coefficient for sorption (L/kg)

13.23

- Nonlinear isotherms are also used (Freundlich or Langmuir)
  - Used to represent a finite capacity for sorption when higher concentrations of P are in solution (Fig. 13.5)
  - Freundlich isotherm is given by Eq. 13.12 and the Langmuir isotherm is given by Eq. 13.13

$$S = KC^N \tag{13.12}$$

$$\frac{C}{S} = \frac{1}{\alpha\beta} + \frac{C}{\beta} \tag{13.13}$$

Where:
S = mass of solute sorbed per mass of media (mg/kg)
C = concentration of solute in solution in equilibrium with the mass sorbed onto the solid (mg/L)
K = distribution coefficient for sorption (L/kg)
N = empirical parameter (–)
α, β = empirical parameters (–)

13.24

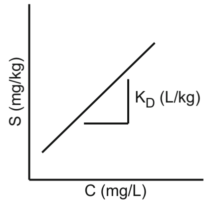


Fig. 13.4 Illustration of a simple linear isotherm for phosphorus sorption as expressed in Eq. 13.11

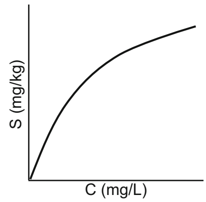


Fig. 13.5 Illustration of a nonlinear isotherm for phosphorus sorption as expressed in Eqs. 13.12 and 13.13

13.25

- Mineral formation from PO_4^{-3} sorbed to surfaces
  - PO_4^{-3} sorbed to surfaces (e.g., soil surfaces) can gradually form precipitated solids
  - Mechanisms are complex and occur over time as a function of pH
  - Under acidic conditions ($pH < 5.5$), Al^{+3} and Fe^{+3} can react with PO_4^{-3} to form amorphous Al- and Fe-phosphates, which gradually change to the crystalline structures, Variscite and Strengite:

$$AlPO_4 \bullet 2H_2O \quad \text{Variscite}$$
$$FePO_4 \bullet 2H_2O \quad \text{Strengite}$$

  - Under alkaline conditions ($pH > 7.3$), Ca^{+2} can react with PO_4^{-3} to form Hydroxyapatite:

$$Ca_{10}(PO_4)_6(OH)_2 \quad \text{Hydroxyapatite}$$

13.26

- Precipitation/complexation of PO_4^{-3} out of solution[d]
  - PO_4^{-3} can be removed by chemical precipitation and complexation with chemicals like $Al_2(SO_4)_3$, $FeCl_3$, or $Ca(OH)_2$
  - Simplified reactions of PO_4^{-3} with alum and ferric chloride

$$Al_2(SO_4)_3 \bullet 14H_2O + 2H_3(PO_4) \rightarrow 2Al(PO_4)_{(S)} + 3H_2SO_4 + 18H_2O \tag{13.14}$$

$$FeCl_3 + H_3(PO_4) \rightarrow Fe(PO_4)_{(S)} + 3HCl_3 \tag{13.15}$$

* An important mechanism for P removal with alum and ferric chloride involves formation of metal hydroxides

$$Al_2(SO_4)_3 \bullet 14H_2O + 3H_2O \rightarrow 2Al(OH)_{3(s)} + 3H_2SO_4 \tag{13.16}$$

$$2FeCl_3 + 3Ca(HCO_3)_2 \rightarrow 2Fe(OH)_{3(s)} + 3CaCl_2 + 6CO_2 \tag{13.17}$$

* PO_4^{-3} forms bonds with the metal hydroxides and is removed from solution by precipitation

[d]*Source*: Stensel and Neethling 2010.

13.27

- Simplified reactions of PO_4^{-3} with $Ca(OH)_2$
  * Addition of calcium hydroxide increases pH
  * Ca^{+2} initially combines with CO_3^{-} to produce $CaCO_3$
  * As pH rises above 10, phosphate can be precipitated out as Hydroxyapatite (Eq. 13.18)

$$10\,Ca(OH)_2 + 6\,H_3PO_4 \rightarrow Ca_{10}(PO_4)_6(OH)_2 + 18\,H_2O \tag{13.18}$$

- Effects of pH on residual dissolved PO_4^{-3}
  * The residual P concentration in solution is inversely related to the chemical dose and has a complex relationship with pH
  * Illustration of equilibrium solubility diagrams for Fe and Al phosphates and phosphate minerals is shown in Fig. 13.6

13.28

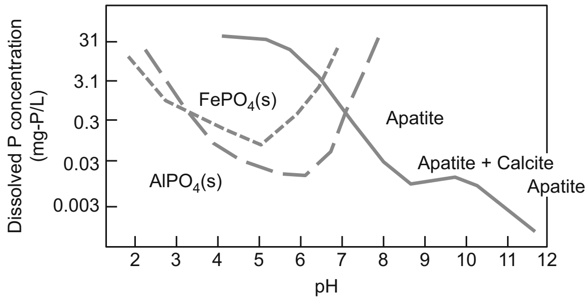


Fig. 13.6 Illustration of residual dissolved P concentration as a function of pH based on equilibrium solubility diagrams for Fe and Al phosphates and phosphate minerals

13.29

- Precipitation of PO_4^{-3} out of solution as struvite
  - Struvite is the phosphate mineral, magnesium ammonium phosphate, that has the formula:

$$MgNH_4PO_4 \bullet 6\ H_2O$$

  - Formation of struvite occurs at higher concentrations of Mg^{+2}, NH_4^+ and PO_4^{-3} according to Eq. 13.19

$$Mg^{+2} + NH_4^+ + PO_4^{-3} + 6H_2O \rightarrow MgNH_4PO_4 \bullet 6H_2O_{(s)} \tag{13.19}$$

  - Conditions favoring the reaction shown in Eq. 13.19
    * Molar ratios of 1:1:1 for Mg^{+2}: NH_4^+: PO_4^{-3}
    * High alkalinity and high pH increase the potential for crystallization
    * NH_4^+ is required for struvite precipitation
  - Struvite precipitation has been applied to diverted urine and treated wastewaters

13.30

13-4. Design and Implementation

- Considerations for design and implementation (D&I) of nutrient reduction strategies and unit operations for decentralized applications
  - Nutrient reduction and recovery goals and requirements
  - D&I considerations of specific strategies and technologies that are most widely applicable in decentralized systems include:
    - Source separation
    - Activated sludge biological systems
    - Porous media biofilters and constructed wetlands
    - Land-based treatment operations
    - Specialty denitrifying biofilters
    - Specialty P-sorptive media filters
    - Chemical treatment systems
  - D&I considerations related to development-scale situations
    - Land-based treatment operations and cumulative effects

13.31

- D&I considerations—Nutrient reduction goals
  - Projects can have requirements concerning N, P, or both
  - Treatment goals and requirements can take forms such as:
    - Achieve a target percent removal—e.g., achieve $\geq$50 % removal of influent total N
    - Achieve a target effluent concentration—e.g., achieve an effluent total P $\leq$ 1 mg-P/L
    - Achieve a target effluent concentration along with a minimum removal—e.g., achieve $\geq$ 50 % N removal and an effluent total N $\leq$ 10 mg-N/L
    - Reduce the nutrient loading to the edge of a stream near a development served by decentralized systems—e.g., reduce the N loading to 50 % of the N per capita wastewater generation rate
  - Recovery goals and requirements can include beneficial use of the wastewater nutrients—e.g., use of N and P as a fertilizer

13.32

- D&I considerations—Source separation
  - Source separation is a strategy that can recover nutrients at the point of generation and simplify treatment and nutrient reduction in the wastewaters handled by decentralized treatment systems
  - As discussed in Chap. 4, source separation options are varied and can have major effects on N and P fate and recovery
  - Two options in particular have substantial potential benefits in terms of reducing the N and P in the wastewater generated (mass of N or P per day per capita)
    - Urine diversion and processing—Potential for removal and recovery of up to 87 % of the N and 50 % of the P
    - Composting toilets—For urine and feces, potential for removal and recovery of up to 97 % of the N and 90 % of the P
  - Source separation must be done carefully to ensure a desired outcome and long-term sustainability (refer to Chap. 4)

13.33

- D&I considerations—Activated sludge systems[a]
  - N and P removal can be achieved in conventional activated sludge systems
    - Activated sludge systems are often designed to achieve secondary treatment of septic tank effluent or similar primary effluents with the primary goal of removal of BOD_5 and TSS
    - If these systems are designed with extended aeration and long SRTs, much of the NH_4^+ can be converted to NO_3^- (Fig. 13.7)
    - A small portion of the P can be taken up into biological cell mass

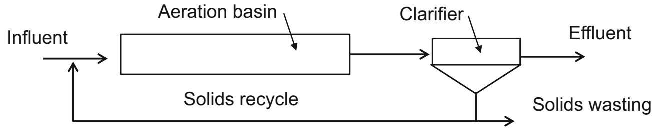


Fig. 13.7 Combined BOD_5 and NH^+ removal in an extended aeration system

[a]*Note:* Chapter 7 covers biological treatment and Chap. 9 covers membrane bioreactors.

13.34

- Enhanced biological nutrient removal of N and P (EBNR) can be achieved in activated sludge systems that are designed specifically for this purpose
  - Systems designed for EBNR are more widely used for centralized treatment systems serving larger design flows (e.g., $>10^6$ gal/day)
    - This is likely because most EBNR system designs are more complicated and costly than other nutrient removal options for decentralized systems and they require more O&M
    - Use of EBNR can be considered for certain applications of decentralized systems (e.g., higher strength wastewaters) as long as continuous operation and routine O&M requirements can be assured
  - Enhanced N and P removal using activated sludge designs is briefly described in the following pages

13.35

- Enhanced N removal in activated sludge systems
  - Activated sludge systems including aerobic treatment units and membrane bioreactors, can be specifically designed to achieve enhanced N removal by nitrification and denitrification
    - 50–85 % removal of total N is possible yielding effluents with total N concentrations at or below 10 mg-N/L
  - System designs for N removal require sequential zones
    - Aerobic zone for nitrification
      - Often use extended aeration bioreactors with long SRTs
      - Alkalinity in the bioreactor has to be adequate to sustain the nitrification process
    - Anaerobic zone for denitrification
      - Requires a source of organic carbon
    - System designs can include one stage or multiple stages, each of which can have aerobic and anaerobic zones
  - Example configurations are shown in Figs. 13.8, 13.9, and 13.10

13.36

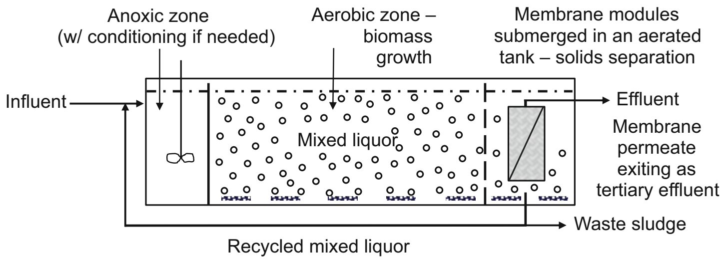


Fig. 13.8 A membrane bioreactor used for combined BOD_5 and NH^+ removal and internal recycling for denitrification and total N removal

13.37

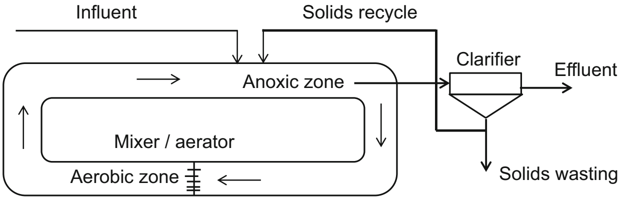


Fig. 13.9 Combined BOD_5 and total N removal in a channel reactor (oxidation ditch)

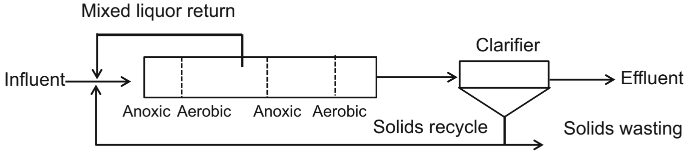


Fig. 13.10 Combined BOD_5 and total N removal in a plug-flow bioreactor system

13.38

- Enhanced P removal in activated sludge systems
  - Activated sludge systems can be specifically designed to achieve enhanced P removal by biological uptake
    - Microbes use P for cell synthesis and energy transport
    - In conventional activated sludge treatment, about 10–30 % of influent P is removed in biomass waste solids
  - Enhanced biological P removal (EBPR) is possible
    - General formula for a bacteria cell in a conventional activated sludge system is $C_5H_7O_2NP_{0.08}$, which has a P content of about 2.1 % dry wt.
    - P accumulating microorganisms (PAOs) can store excess P as an energy reserve in polyphosphate granules in their cells
    - Under anaerobic conditions, PAOs release ortho PO_4^{-3}, utilizing the energy to accumulate simple organics
    - Under aerobic conditions, PAOs grow on the stored organic material, using some of the energy to take up ortho PO_4^{-3}

13.39

  - In practice, EBPR can be achieved by sequencing reactors or zones with the proper anaerobic and aerobic conditions
    - An example configuration is shown in Fig. 13.11
      (*Note:* This system layout does not achieve nitrification)
    - Waste biosolids can contain up to 5 wt.% P compared to about 2 wt.% in non-EBPR systems

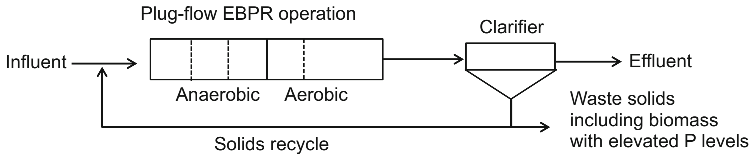


Fig. 13.11 Example of a system configuration for enhanced biological removal of phosphorus

13.40

- Combined N and P removal in activated sludge systems
  - There are a number of configurations that can be conceived for combined removal of N and P
  - An illustration of one configuration is shown in Fig. 13.12

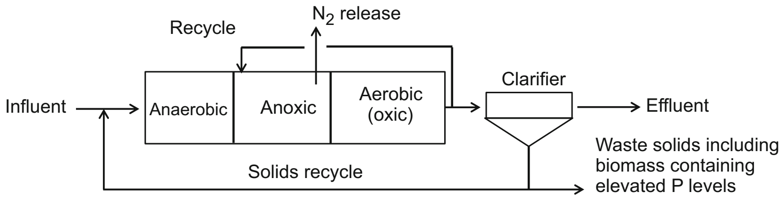


Fig. 13.12 Illustration of the flow regime for the A^2/O process for combined N and P removal. *Note:* Information on the A^2/O process can be found at http://www.veoliawaterst.com/oxidationditch/en/ao.htm

13.41

- Expected effluent quality from activated sludge N and P removal
  - Table 13.7 provides achievable N and P removal using different types of activated sludge systems
  - Achieving the N and P removal potential shown requires proper design and operating conditions consistent with design assumptions
- Design calculations
  - Design of activated sludge systems to achieve enhanced removal of N and P requires careful consideration of many factors and is beyond the scope of this text
  - Design details can be found in other texts (e.g., Grady et al. 2011; Tchobanoglous et al. 2014)

13.42

Table 13.7 Representative N and P concentrations achievable after treatment in conventional and enhanced activated sludge systems

Treatment system	Target treatment	Typical treatment processes	Nitrogen	Phosphorus
			C_E (mg-N/L)	C_E (mg-P/L)
Conventional activated sludge	BOD and TSS removal with only incidental N and P removal	Screening; solids separation; aerobic treatment; clarification; chlorination/dechlorination	20–30	4–6
Enhanced biological nutrient removal	BOD and TSS plus N and P removal	Screening, solids separation; multiple stages of anaerobic, anoxic, and aerobic zones; clarification, chlorination/dechlorination	1–6	0.25–1.0
Enhanced biological nutrient removal with chemical treatment	BOD and TSS plus N and P removal	Screening, solids separation; multiple stages of anaerobic, anoxic, and aerobic zones; clarification; chemical addition with media filtration; chlorination/dechlorination	0.6–4.0	0.01–0.36

Source: Based on the published data compilation presented in Table 4, CDPHE 2010. These data are no specific to decentralized system applications and presented here for general information only.

13.43

- D&I considerations—Porous media biofilters[a]
  - Porous media biofilters that employ a single pass flow regime (e.g., single pass sand filter or peat biofilter) can convert most of the NH_4^+ to NO_3^- and remove a small portion of the total N by denitrification
  - Porous media biofilters that are designed with a recirculation flow regime (e.g., recirculating sand or manufactured media biofilters) can achieve total N removals of 50 % or more via nitrification and denitrification
    - A proportional splitter diverts some filtrate from the biofilter back to a blend tank or recirculation tank (Fig. 13.13)
    - In the blend tank and recirculation tank, NO_3^- in the filtrate recycled can be denitrified to N_2
    - This flow regime increases denitrification and also adds alkalinity to the influent to the biofilter, which is important for nitrification

[a]*Note*: Porous media biofilters are covered in Chap. 8.

13.44

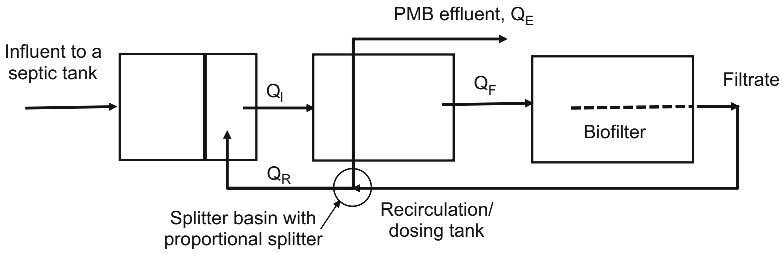


Fig. 13.13 Plan view schematic of an example of a recirculating biofilter designed for enhanced N removal

13.45

- D&I considerations—Constructed wetlands[a]
  - Constructed wetlands are often designed to treat primary or secondary effluent and remove BOD_5 and TSS
    - A conventional constructed wetland can have some incidental removal of N and P by plant uptake assuming the plants are harvested and removed from the site
  - Constructed wetlands can be designed for nutrient removal
    - Nitrogen removal—The wetland can be divided into cells with a long hydraulic retention time (e.g., 2–3 times that needed for just BOD_5 and TSS removal) and the flow regime can include one or more sequences of aerobic and anaerobic zones to achieve biological nitrification and denitrification
    - Phosphorus removal—The wetland can be constructed with P-sorptive media as the aggregate (e.g., lightweight expanded clay aggregate) to achieve high P removal efficiency by media sorption and precipitation

[a]*Note*: Constructed wetlands are covered in Chap. 10.

13.46

- D&I considerations—Land-based treatment operations[a]
  - Land-based treatment operations are widely used in decentralized systems and they can achieve high N and P removal
    - Reactions affecting nitrogen can potentially involve:
      - Plant uptake of NH_4^+ and NO_3^-
      - Sorption of NH_4^+ to soil media surfaces
      - Biological nitrification of NH_4^+ to NO_3^-
      - Biological denitrification of NO_3^- to N_2O and N_2
    - Reactions affecting phosphorus can potentially involve:
      - Plant uptake of PO_4^{-3}
      - Sorption of PO_4^{-3} to soil media surfaces
      - Precipitation of sorbed PO_4^{-3} into a mineral form
  - N and P removal can occur in the treatment operation (Table 13.8) and the subsurface during groundwater assimilation and attenuation

[a]*Note*: Chapter 11 covers subsurface soil treatment units and Chap. 12 covers landscape drip dispersal.

13.47

Table 13.8 Nutrient removal in common types of land-based treatment operations[a]

System type	Nitrogen removal R_E	Nitrogen removal Major processes	Phosphorus removal R_E	Phosphorus removal Major processes
Subsurface infiltration	20–60 %	Denitrification (greater with nitrified effluent applied to finer textured soils)	90–> 99 %	Sorption and precipitation (depends on soil texture and mineralogy with >99 % removals in finer grained soils and potentially <90 % removals in coarse grained soils)
Mound system infiltration	40–60 %	Denitrification (enhanced in the landscape surface zone)		
Landscape drip dispersal	50–70 %	Plant uptake (depends on vegetation and ET) and denitrification (greater in finer textured soils)		

[a]Based on application of domestic STE and R_E determined based on soil pore water after travel througl 3 ft of an unsaturated, aerobic soil profile with conditions conducive to treatment.

13.48

- ○ Treatment efficiency for nitrogen can be complicated
  - * For example, in a subsurface soil treatment unit, total N removal depends on soil profile attributes combined with wastewater loading rate and composition
  - * Results of model simulations and studies illustrate the interactions affecting the removal of NH_4-N and total N
    - – Complete conversion of NH_4^+ by 2 ft depth is common, except at higher HLRs applied to finer-grained soils
    - – By 2 ft depth total N removal is often 30–50 % or more except at higher HLRs (e.g., 1.2 gal/day/ft^2) applied to coarse grained soils
    - – By $\geq$3 ft depth total N removal can be reach 70–100 % at low HLRs (e.g., 0.5 gal/day/ft^2 or less)
  - * If the soil infiltrative surface is ponded with wastewater, the soil below it may be anoxic and nitrification will be hindered
    - – Application of nitrified effluent under this condition could yield high total N removals

13.49

- ■ D&I considerations—Denitrifying biofilters
  - ● Denitrifying biofilters can achieve high efficiency NO_3^- removal
    - ○ Organic or inorganic media can be established in packed beds (Fig. 13.14)
      - * Wood chips are an example of organic media
      - * Beads of elemental sulfur are an example of inorganic media
    - ○ The influent to a denitrifying biofilter has N in the form of NO_3^- (e.g., porous media biofilter effluent)
    - ○ Flow through the bed can be either upflow, downflow or lateral flow as long as the bed remains saturated (Fig. 13.15)
      - * During flow through a bed of media, anoxic conditions must be present
      - * An adequate water column above the top of the bed of media is needed to limit O_2 gas transfer into the bed

13.50

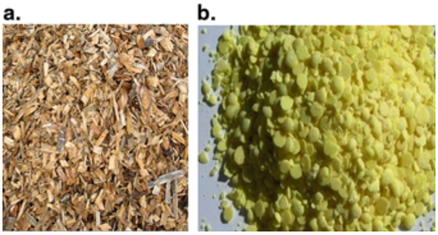


Fig. 13.14 Photograph of wood chips (*left*) and beads of elemental sulfur (*right*) used in denitrifying biofilters (Photographs courtesy of Hazen and Sawyer)

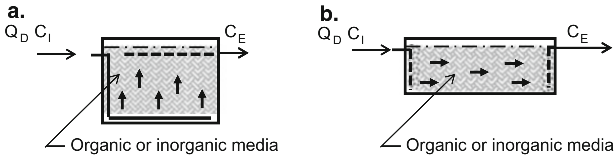


Fig. 13.15 Profile view schematic of a denitrifying biofilter with a flow regime that is saturated upflow (*left*) or saturated lateral flow (*right*)

13.51

- Basic features of organic media denitrifying biofilters
  - Organic media provides a source of organic carbon as an electron donor and substrate in heterotrophic biological reactions
    - Suspended growth and attached biological growth can occur with carbon available in the solid surface and dissolved
  - Different organic materials have been used as media
    - Wood chips (Fig. 13.14a) have been most common
    - Sawdust, straw, corn, compost, seaweed, etc. can be used too
    - Inert media (e.g., coarse sand or pebbles) can also be mixed in to maintain saturated hydraulic conductivity (K_S)
  - Flow through the bed is saturated and conditions become anoxic
    - If DO is present in the influent, aerobic biodegradation of organic matter occurs
    - Once DO is depleted, NO_3^- can be biologically denitrified
    - If NO_3^- is depleted, SO_4^{-2} present can be biologically reduced

13.52

- Characteristics of organic media denitrifying biofilters
  - Media choice depends on properties, availability, and cost
  - Wood chips are effective and readily available at low cost
    - Wood chip size range is typically 0.25- to 2-in. diameter
    - Bed porosity is typically 0.5–0.7 v/v
    - Hard wood vs. softer woods (faster growing)
      - No difference in NO_3^- removal rates
      - Hard wood may maintain structural integrity better
  - Hydraulic conductivity of wood chip beds
    - K_S can be around 2500 gal/day/ft^2
  - Nitrate removal rates within a bed of wood chips
    - Literature rates are 0.2–0.6 lb-N/day/1000 ft^3 (Table 13.9)
    - Lab tests can measure rates with a particular media
      - Data from $\geq$1 year appear representative of long term

13.53

Table 13.9 Nitrate removal rates for wood chip denitrifying biofilters (after Schipper et al. 2010)

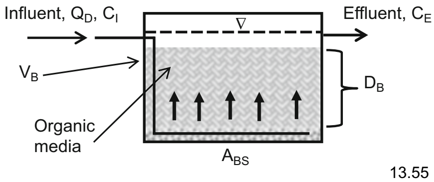
Reference	Bioreactor volume		Typical NO_3^- inputs		Temp. °C	Average N removal rate[a]	
	ft^3	m^3	lb-N/day/1000 ft^3	gN/day/m^3		lb-N/day/1000 ft^3	g-N/day/m^3
Robertson (2010)	70.6	2	0.31	5	10	0.62 0.11[a]	10
Robertson et al. (2005a, b)	318	9	1.06	17	15	0.15[1]	1.8[1]
Robertson et al. (2005a, b)	3813	108	2.37	38	15	0.16[1]	2.4[1]
Robertson et al. (2005a, b)	4237	120	2.18	35	15	0.32[a]	2.5[1]
Robertson et al. (2005a, b)	12,712	360	0.87	14	15	0.13	5.1[1]
van Driel et al. (2006)	24.7	0.7	0.56	9	9	0.23	2.1
van Driel et al. (2006)	7.06	0.2	0.81	13	13	0.20	3.7
Robertson and Merkley (2009)	1412	40	0.31	5	8	0.21 0.087[a]	3.2
Robertson et al. (2009)	600	17	0.62	10	7.7	0–0.69[1]	3.4
Schipper et al. (2010)	2931	83	3.31	53	15–25	0.61	1.4[1]
Schipper et al. (2010)	10,381	294	0.34	5.5	20		0–11[a]
Schipper et al. (2010)	46,609	1320	15.6	250			9.7

[a]Nitrate removal rate is limited by NO_3^- depletion and very low concentrations. *Source*: Table 3 in Schipper et al. 2010. *Note*: 1 $m^3 = 35.3$ ft^3 and 1 g-N/day/$m^3 = 0.0624$ lb-N/day/1000 ft^3.

13.54

- Geometry of an organic media denitrifying biofilter (Fig. 13.16)
  - Biofilter volume (V_B)
    - Volume is critical as it determines HRT (see Eq. 13.20)
    - Need HRT long enough for NO_3^- removal (e.g., >2 days)
  - Biofilter bed depth (D_B)
    - A minimum depth is important to avoiding short circuiting and providing available organic carbon
    - Depths of 2–10 ft have been used
  - Biofilter cross-sectional surface area (A_{BS})
    - HLR to the surface area does not appear to affect N removal

Fig. 13.16 Profile view schematic of an organic media denitrifying biofilter with a flow regime that is saturated upflow



13.55

- Expected effluent quality from wood chip denitrifying biofilters
  - Total N levels depend on N species in the influent and the biofilter HRT
    - Influent N must be in the NO_3^- form to be denitrified
    - With influent N as NO_3^- and a sufficiently long HRT, median total N = <3 mg-N/L is achievable
  - Fecal coliform bacteria
    - Concentrations are reduced >99.9 %
    - Median concentrations can be <200 org/100 mL
  - $cBOD_5$ is elevated after flow through the biofilter
    - Due to organic carbon from the organic media used
    - Biofilter effluent $cBOD_5$ = 25 to 50 mg/L or more is likely
    - This biofilter effluent $cBOD_5$ has to be accounted for in the design of downstream unit operations or discharge plans

13.56

- Longevity of an organic media denitrifying biofilter
  - Two factors can control longevity of wood-chip based biofilters
    - NO_3^- removal rates
    - Hydraulic conductivity of the packed bed of media
  - Experience with operating wood chip biofilters
    - Operation periods of up to 20 year +/− are achievable
    - Generalized observations during this period
      - Little to no change in NO_3^- removal rates
      - Little to no change in hydraulic conductivity
  - Biofilter rejuvenation
    - If at some point NO_3^- removal rates decline or hydraulic conductivity is lost biofilter rejuvenation may be required
    - Rejuvenation can accomplished by media addition and/or replacement

13.57

- Inorganic media denitrifying biofilters
  - Elemental sulfur (S) can be used in a packed bed
  - Sulfur serves as an electron donor for autotrophic bacteria and NO_3^- is reduced to N_2 gas and S is oxidized to SO_4^{-2}
    - Biological processes involve attached growth of microorganisms
  - Elemental sulfur in a pelletized form is available
    - For example, in Florida sulfur beads are available in bags for use as a fertilizer in the citrus industry (see Fig. 13.14b)
  - Flow through the bed is saturated and conditions are anoxic
    - Alkalinity addition is often needed to sustain the process since alkalinity is consumed (see Eq. 13.10)
    - Sources of alkalinity can include limestone chips or oyster shells which are mixed with the elemental sulfur beads

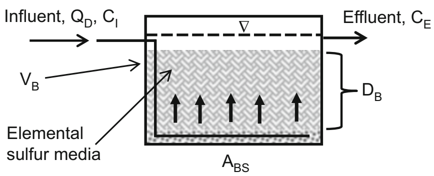
13.58

- Characteristics of packed beds of elemental sulfur media
  - Sulfur beads and other media
    - Sulfur beads
      - Particle size range is typically 2–5 mm
    - Other media can be mixed in as a source of alkalinity
      - e.g., 10–20 % of the packed bed can be oyster shells (3- to 15-mm diameter)
  - Bed characteristics
    - Bed porosity is typically about 0.4 v/v
    - K_S can be about 250 gal/day/ft^2
  - Nitrate removal rates within a sulfur based biofilter
    - Literature data suggests NO_3^- removal rates can be on the order of 4.4 lb-N/day/1000 ft^3

13.59

- Geometry of a sulfur based denitrifying biofilter (Fig. 13.17)
  - Biofilter volume (V_B)
    - Volume is critical as it determines HRT (see Eq. 13.20)
    - Need HRT long enough for NO_3^- removal (e.g., >0.2 days)
  - Biofilter bed depth (D_B)
    - A minimum depth is needed (e.g., >2 ft) to avoiding short circuiting and to provide available organic carbon
  - Biofilter cross-sectional surface area (A_{BS})
    - HLR to the bed inlet surface area (A_{BS}) can affect hydraulic performance if there is clogging of the sulfur beads

Fig. 13.17 Profile view schematic of an sulfur media denitrifying biofilter with a flow regime that is saturated upflow



13.60

- Expected effluent quality from sulfur denitrifying biofilters
  - Effluent levels of total N depend on the biofilter HRT
    - Median total N of <3 mg-N/L is achievable
  - Fecal coliform bacteria
    - Concentrations can be reduced by 99 %
    - Median concentrations can be <200 org/100 mL
  - SO_4^{-2} levels are elevated after the biofilter
    - SO_4^{-2} generation is based on NO_3^- removed
      - According to Eq. 13.10, about 7.5 mg-SO_4^{-2}/L is formed per 1 mg-N/L of NO_3^- removed

        So, removal of 50 mg-N/L of NO_3^- can generate up to 375 mg-SO_4^{-2}/L
      - Sulfate has a secondary drinking water limit of 250 mg/L
    - To control SO_4^{-2} generation, a sulfur denitrifying biofilter has been used as a 2nd stage after denitrification in a wood chip biofilter

13.61

- Longevity of an elemental sulfur denitrifying biofilter
  - Two factors can control longevity of sulfur based biofilters
    - NO_3^- removal rates
    - Hydraulic conductivity of the packed bed of media
  - Experience with operating sulfur based biofilters
    - There is less experience with sulfur biofilters compared to wood chip biofilters
    - Clogging of the inlet to the packed bed is feasible given the bead particle size, particularly if it is used as a 2nd stage following a wood chip biofilter and influent $cBOD_5$ is high
  - Biofilter rejuvenation
    - If at some point NO_3^- removal rates decline or hydraulic conductivity is lost biofilter rejuvenation may be required
    - Similar to an organic media biofilter, rejuvenation can accomplished by media addition and/or replacement

13.62

- Design calculations for a denitrifying biofilter for NO_3^- removal
  - Biofilter volume for uniform saturated flow and zero-order kinetics is calculated using Eqs. 13.20 to 13.22

$$V_B = \frac{Q(C_I - C_E)F}{R_{NR}} \qquad (13.20)$$

$$V_{B'} = (V_B)(n_e) \qquad (13.21)$$

$$V_{B'} = (Q)(HRT) \qquad (13.22)$$

Where:

V_B = total volume of the biofilter (L × W × H) (ft^3)
$V_{B'}$ = empty bed volume for flow in the biofilter (ft^3)
Q = daily flow rate (design or actual) (ft^3/day)
C_I = concentration of NO_3^- in the influent (mg-N/L)
C_E = concentration of NO_3^- in the effluent (mg-N/L)
R_{NR} = rate of NO_3^- removal (lb-N/day per 1000 ft^3) (e.g., range = 0.2 to 0.6 for wood chips (Table 13.9)) and 4.375 for sulfur (R_{NR} is temperature dependent and can be low if NO_3^- is limited (i.e., near 0))
n_e = effective porosity (v/v) (e.g., 0.6–0.7 for a bed of wood chips and 0.4 for a bed of sulfur beads and oyster shells)
HRT = hydraulic retention time in the biofilter (days)
F = 8.34 × 10^{-6} = conversion factor for mg/L to lb/gal

13.63

* Surface area and loading rates are calculated using Eqs. 13.23 to 13.26

$$A_{BS} = \frac{V_B}{D_B} \qquad (13.23)$$

$$HLR = \frac{Q}{A_{BS}} \qquad (13.24)$$

$$Q_{VLR} = \frac{Q}{V_B} \qquad (13.25)$$

$$M_{VLR} = \frac{(Q)(C_I)F}{V_B} \qquad (13.26)$$

Where:

A_{BS} = surface area of the biofilter (ft^2)
V_B = total volume of the biofilter (ft^3)
D_B = depth of the biofilter (ft) (e.g., 2–10 ft)
HLR = hydraulic loading rate to the biofilter surface (gal/day/ft^2)
Q_{VLR} = flow volumetric loading rate (gal/day/ft^3)
Q = flow rate to the biofilter (design or actual) (gal/day)
C_I = concentration in the influent to the biofilter (mg/L)
M_{VLR} = mass volumetric loading rate (lb-N/day/ft^3 or lb-P/day/ft^3)
F = 8.34 × 10^{-6} = conversion factor for mg/L to lb/gal

13.64

- D&I considerations—Two-stage biofilters
  - N removal can be enhanced in a 2-stage biofilter
    - In a 2-stage biofilter nitrification is accomplished in a porous media biofilter and then denitrification is accomplished in a denitrifying biofilter
    - An example 2-stage biofilter is illustrated in Figs. 13.18 and 13.19
      - Stage 1—Aerobic biofilter with intermittent dosing and downward flow in an unsaturated bed of inert media (e.g., sand) or NH_4^+ sorptive media (e.g., expanded clay, zeolites, etc.)
      - Stage 2—Anoxic biofilter with saturated flow through a bed of reactive media for heterotrophic (e.g., wood chips) and/or autotrophic denitrification (e.g., elemental sulfur)
    - Results from a field application of three different 2-stage biofilter configurations are given in Table 13.10

13.65

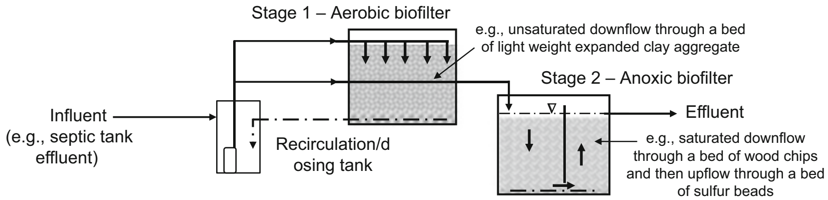


Fig. 13.18 Profile view schematic of a two-stage biofilter for nitrogen removal (Anderson et al. 2014).

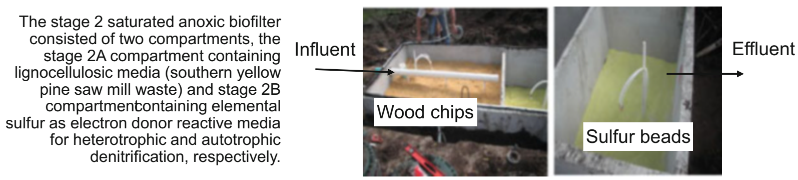


Fig. 13.19 Photograph of a Stage 2 anoxic biofilter within a two-stage biofilter for nitrogen removal (Photographs courtesy of Hazen and Sawyer).

13.66

Table 13.10 Nitrogen removal in three different passive 2-stage biofilter systems (Smith et al. 2008)

Parameter	System 1	System 2	System 3
Media used in Stage 1 and Stage 2	1. Clinoptilolite (C); 2. 75 % sulfur (S) + 25 % oyster shells (OS)	1. Expanded clay; 2. 60 % sulfur + 20 % oyster shells + 20 % expanded shale (ES)	1. Tire crumbs; 2. 45 % sulfur + 15 % oyster shells + 40 % expanded shale
	Particle sizes used: Stage 1 stratified beds with C = 0.3 to 4.76 mm, EC = 0.4 to >5 mm, TC = 0.3 to > 5 mm; Stage 2 nonstratified beds with sulfur = 2 to 5 mm, OS = 3 to 15 mm, EC = 0.4 to 4.5 mm		
Loading rates applied to Stage 1	2.71 gal/ft^{-2}/day $cBOD_5$ = 22.8 g/m^2/day $cBOD_5$ = 22.8 g/m^2/day	2.95 gal/ft^{-2}/day $cBOD_5$ = 24.2 g/m^2/day Total N = 9.23 g/m^2/day	2.51 gal/ft^{-2}/day $cBOD_5$ = 20.6 g/m^2/day Total N = 7.83 g/m^2/day
Total N removal[a]	Avg. = 97.1 % Range = 94.9–97.9 %	Avg. = 97.7 % Range = 96.6–98.6 %	Avg. = 33.0 % Range = 2.2–50.6 %
Effluent Total N (mg-N/L)	2.2	2.1	43.9
Effluent TIN (mg-N/L)	0.14	0.63	42.1

Note: Passive means only 1 pump is used. Media depths = 2 ft. Clinoptilolite is a natural zeolite comprising a microporous arrangement of silica and alumina tetrahedra. Average STE concentrations applied to Stage 1 were: Total N = 77.4 mg-N/L (std. dev. = 6.2 mg-N/L); $cBOD_5$ = 203 mg/L; TSS = 18.7 mg/L.
[a]Removal efficiencies are for the combined Stage 1 plus Stage 2 biofilters.

13.67

- D&I considerations—Specialty P-sorptive media filters
  - Media with high sorption affinity for P can be used, e.g.,:
    - As filter media in packed bed filters
    - As aggregate in constructed wetlands
  - Sorption media can be derived from naturally occurring media, manufactured media, and waste products
  - P removal can occur by
    - Sorption to charged sites on the media surface
      - Fast process that may be reversible
    - Precipitation of phosphate minerals
      - Slower process that enables long-term removal
      - Common mineral form is hydroxyapatite, $Ca_{10}(PO_4)_6(OH)_2$

13.68

- Classification of P sorption media
  - Example classification of sorption level for different media is given in Table 13.11
    - Sorption level is based on the maximum P sorption capacity (PSC) and media particle size

Table 13.11 Phosphorus sorption media types and capacities (after Cucarella and Renman 2009)

Sorption level	Maximum P sorption capacity (g P/kg media dw)	Filter media size and source[a]	
		Fine = <1 mm	Coarse = >1 mm
Very high	>10	BFS[b], Fly ash[b], Polonite, Red mud	No data
High	1–10	BFS[b], Fly ash[b], Fe-coated sand and brick chips	BFS[b], EAF, Filtra P, Filtratlite P, Polonite, Shell sand, UTELITE
Moderate	0.5–1	Bentonite, calcareous soils, Fly ash[b], Spodosol	Bauxite, BFS, Zeolite[b]
Low	0.1–0.5	Sand, soils	LECA, Limestone, Opoka
Very low	<0.1	Soils	Gravels

[a]*BFS* blast furnace slag, *EAF* electric arc furnace slag, *LECA* lightweight expanded clay aggregate.
[b]Depends on chemical composition. *Source*: Table 4 in Cucarella and Renman (2009).

13.69

- Flow diagrams for P-sorptive filters
  - Influent is typically secondary effluent or better quality
  - Flow regimes can be unsaturated or saturated (Fig. 13.20)

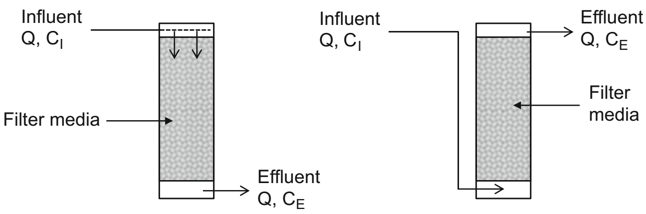


Fig. 13.20 Profile view schematics of packed bed filters for P removal showing an unsaturated downflow (*left*) and a saturated upflow (*right*) flow regime

- Expected effluent quality from P sorptive filters
  - Total P levels depend on effluent contact with the media surface
  - With adequate contact, median total P = <0.5 mg-P/L is achievable

13.70

- Longevity of a P sorptive media filter
  - Two factors control longevity of P sorptive media filters
    - PSC of the filter media
    - Hydraulic conductivity of the packed bed
  - Experience with operating P sorptive media filters
    - Run length depends on the media PSC and the mass of media in the filter
    - Little to no change in hydraulic conductivity should occur with high quality effluent (e.g., secondary effluent) applied to 1–5 mm diameter media
  - Filter rejuvenation
    - After some period of operation, a filter may lose its ability to remove P (e.g., due to saturation of sorption sites)
    - Rejuvenation typically involves media replacement since regeneration is difficult or infeasible

13.71

- Design calculations for sizing a P-sorptive filter
  - Volume and mass of P-sorptive media required for a selected run length is given by Eqs. 13.27 and 13.28

$$V_{SM} = \frac{(V)(C_I)(F)}{(\rho_b)(PSC)} \quad (13.27) \qquad M_{SM} = (V_{SM})(\rho_b) \quad (13.28)$$

Where:
V_{SM} = volume of filter media required (ft^3)
V = volume of effluent processed at media saturation (gal)
C_I = concentration of P in the influent (mg-P/L)
PSC = P sorption capacity of filter media (lb-P/lb dw of media)
Q = flow rate being processed (gal/day)
T = time period desired before filter exhaustion (day)
ρ_b = media dry bulk density (lb/ft^3)
M_{SM} = mass of filter media (lb dw of media)
$F = 8.34 \times 10^{-6}$ = conversion factor for mg/L to lb/gal

13.72

- Surface area and geometry are calculated using Eqs. 13.29 and 13.30 assuming a cylindrical geometry

$$A_{FS} = \frac{V_{SM}}{D_B} \tag{13.29}$$

$$d_F = \sqrt{\frac{A_{FS} \bullet 4}{\pi}} \tag{13.30}$$

Where:
A_{FS} = area of the filter surface (ft^2)
V_{SM} = volume of filter media needed for the run length (ft^3)
D_B = depth of the filter bed (ft) (e.g., 2–10 ft)
d_F = diameter of the filter vessel (ft)

13.73

- D&I considerations—P removal by chemical treatment[i]
  - Chemical treatment for P removal involves chemicals mixed into a wastewater to convert the dissolved PO_4^{-3} in solution into a solid that can be removed by solids separation processes
  - Chemicals that are used for this purpose include:
    - Aluminum sulfate (alum)—$Al_2(SO_4)_3$
    - Ferric chloride—$FeCl_3$
    - Lime—CaO, $Ca(OH)_2$
  - Removal of dissolved P can occur by two major processes
    - Complexation with Al and Fe metal hydroxides that form and are precipitated out of solution
    - Formation of a hydroxyapatite precipitate ($Ca_{10}(PO_4)_6(OH)_2$) that forms when lime is used to raise the pH >10

[i]*Note:* information presented here is based in part on Stensel and Neethling 2010.

13.74

- D&I considerations related to chemical addition
  - What chemical will be used
    - Choice is typically between aluminum sulfate (alum), ferric chloride, and lime
    - Which is best depends on project specifics including cost, suitability for the treatment process, storage and handling
  - What chemical dose will be added at what points
    - Required molar ratios of the particular chemical chosen given the P removal efficiency and residual P level desired
    - Chemical addition points within the treatment train and dose concentrations and feed flow rates (considering the range of flows over design period)
  - Chemical handling and delivery methods
    - Chemical storage
    - Chemical feeding, mixing, flocculation
    - Sludge production

13.75

- Flow diagrams for chemical addition
  - Chemicals can be added at one or more locations as illustrated in Fig. 13.21
  - There are pros and cons to different points of addition as summarized in Table 13.12

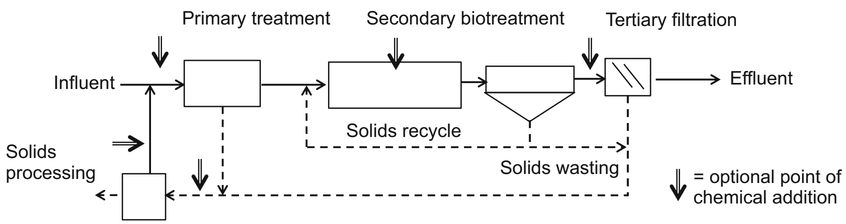


Fig. 13.21 An example treatment train illustrating typical locations where chemical addition can occur (after Stensel and Neethling 2010)

13.76

Table 13.12 Attributes of optional points of chemical addition (after Stensel and Neethling 2010)

Point of addition[a]	Potential benefits	Potential detriments
Primary—Addition prior to or during primary treatment	Increases removal of BOD (up to 65 %) and TSS (up to 90 %) and decreases loading to secondary treatment unit	Higher sludge production
Secondary—Into aeration basin or before secondary clarifier or membrane	For effluent P < 1.0 mg/L good point for dosing; May help improve TSS removal in clarifiers; Helps prevent fouling in MBR systems	Mixed liquor suspended solids have a higher fraction of inert solids; Can remove alkalinity; cannot add lime here due to pH effects
Tertiary—Before filtration or polishing membranes	For effluent P < 0.5 mg/L this is a good control point for dosing; Will help improve TSS removal; Can recycle precipitant to head of train for added P removal	Filtration increases cost and O&M; filter solids breakthrough can cause spikes in effluent P
Multiple points of addition	Can achieve lower effluent P; Can optimize chemical dose to lower requirements; Provides flexibility	Additional cost for chemical feeders at multiple locations; Increased operational complexity

[a]*Note*: Lime cannot be added during secondary treatment due to elevated pH effects. If added during primary treatment pH adjustment may be needed before secondary treatment. Where used, lime is often added after secondary clarification.

13.77

- Estimating the chemical dose generally required
  - Quantity of $Al_2(SO_4)_3$ or $FeCl_3$ based on stoichiometry (Eqs. 13.14 to 13.17) suggests the following
    - 1 mol of P reacts with 1 mol of Al or 1 mol of Fe
    - 1 mol of Al or Fe produces 3 equiv. of acid (H_2SO_4 or HCl_3)
    - Alkalinity used per millimole is 150 mg/L as $CaCO_3$ (0.92 g/g-$FeCl_3$ or 0.50 g/g-$Al_2(SO_4)_3$
  - Specific molar ratios of Al/P or Fe/P depend on the P removal required and certain factors with the application
    - For 80–98 % P removal and soluble P of 4.0–1.0 mg/L, molar ratios of Al/P or Fe/P are in the range of 1.0–1.5
    - For higher efficiency and to achieve soluble P of 0.1 mg/L, molar ratios of Al/P or Fe/P of 6.0–7.0 are required
  - Quantity of $Ca(OH)_2$ required depends on alkalinity and not the PO_4^{-3} present
    - $Ca(OH)_2 \approx 1.4$ to 1.5 times the alkalinity as $CaCO_3$

13.78

- Factors influencing the specific dose required include:
  - Wastewater pH
    - For alum and ferric chloride the highest P removal efficiency is in pH 5.5–7.0 range

      Metal salts consume alkalinity and can lower pH
    - For lime, pH must reach >10
  - Mixing is critical (G values of 200–400 s^{-1})
  - Other wastewater characteristics
    - Colloids and solids can affect P-metal hydroxide complexes
    - Al and Fe can react with humic substances
    - Alkalinity affects the dose requirements for lime

13.79

- Determining the project-specific chemical dose needed
  - Jar test experiments are often needed to compare different chemicals and dosages and their efficiencies in removal of P
  - A jar test apparatus simulates the mixing, flocculation and settling involved in chemical treatment and typically includes:
    - Rapid mix for 1 min
    - Gentle stirring for 20–40 min
    - Settling for 15–60 min
  - Jar testing can help determine the optimum chemical dose and pH to reach a desired residual P level
    - The effects of alkalinity can also be examined
  - Without jar testing, general guidance for Fe and Al salts is:
    - For 1–4 mg/L residual P ➔ 1.0–1.5 mol metal per mol P
    - For 0.5–0.3 mg/L residual P ➔ 2–4 mol metal per mol P

13.80

- Expected effluent quality from chemical treatment
  - Effluent quality will depend on the wastewater characteristics, the type of chemical and dose used, and the point of addition
  - To achieve high P removal efficiency and low residual P concentrations
    - Requires higher chemical doses at one or more points
    - Highly efficient solids separation by single or multiple staged filtration using methods such as:
      - Granular media filters
      - Cloth filters
      - Continuous backwash filters
      - Ultrafiltration or microfiltration membranes
    - With filtration after flocculation and settling, an effluent total P of 0.05–0.10 mg/L is achievable
  - Chemical addition can also aid removal of BOD and TSS

13.81

- Sludge production
  - Sludge production is a major factor related to chemical choice, dose, and point of addition
  - Addition in a primary unit can increase sludge by 50–100 % from that unit and 60–70 % for the system
  - Addition in a clarifier after biological treatment can increase sludge by 35–45 % from that unit and 10–25 % overall
  - Use of alum for secondary effluent can yield the lowest quantity
- Design calculations
  - Design of chemical treatment systems for P removal requires careful consideration of many factors and the design details are beyond the scope of this text
  - Design details can be found in other texts (e.g., USEPA 2010; Neethling 2010; Tchobanoglous et al. 2014)

13.82

13.5. Development-Scale Situations

- Development-scale situations and concerns
  - When larger numbers of decentralized systems are applied at a development or watershed scale, concerns can be focused on N or P fate and effects on groundwater and surface waters
    - Examples of concerns that are often confronted include:
      - NO_3^- loadings to groundwater which may be used as a source of drinking water or which connects to sensitive surface waters
      - PO_4^{-3} loadings to inland surface waters
      - N and P loadings to estuaries and coastal waters
  - Concerns most often arise where there are systems that include release of effluent to the land surface or subsurface environment and count on nutrient attenuation as water is reclaimed and assimilated into a local hydrologic system
    - A prime example of this situation occurs when there are large numbers of land-based systems in a given geographic area

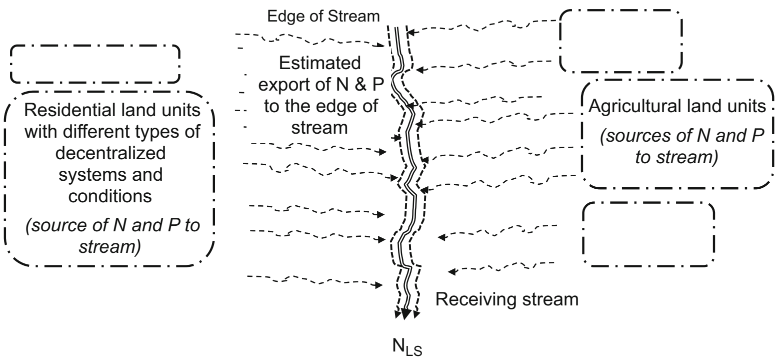
13.83

- Assessments can be made of a given situation
  - With the N and P removal achievable in an existing or proposed set of decentralized systems, the nutrient loadings to the receiving water of concern can be estimated
  - Methods of assessment are varied and can include:
    - Mass balance calculations and GIS modeling
    - Mathematical modeling, for example using:
      - Hydrus-2D (Heatwole and McCray 2007)
      - STUMOD (Geza et al. 2014)
      - WARMF (Siegrist et al. 2005; Geza et al. 2010)
    - Based on the assessment, and considering decentralized system loadings in the context of loadings from other sources (e.g., agriculture drainage tiles or runoff) an evaluation can be made as to what additional treatment is required for N or P removal prior to release of effluent to the land surface or subsurface

13.84

- Conceptual model and framework for simplified mass balance analysis of nutrient loadings
  - Figure 13.22 shows a conceptual model for nutrient inputs to streams
  - Relevant components and processes affecting decentralized system nutrient removal in the context of nutrient loads to the edge of surface waters are illustrated in Fig. 13.23
  - The decentralized system components shown in Fig. 13.23 have potential removal efficiencies for nutrients which affects the nutrient loading as illustrated in Fig. 13.24
  - Estimating the decentralized systems nutrient load (N or P) to an edge of stream (N_{LS}) can be done using Eq. 13.31
    - An example assessment using hypothetical values in Eq. 13.31 is shown in Table 13.13

13.85



N_{LS} = sum of the nutrient loads from individual sources to the edge of stream. (Note N_{LS} can be reduced in riparian zones at the stream edge or during stream flow)

Fig. 13.22 Illustration of a conceptual model for estimating nutrient loads to a stream including the loads from decentralized systems

13.86

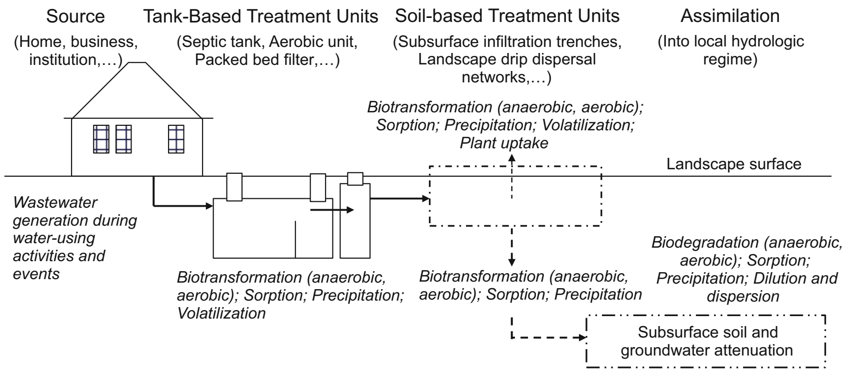
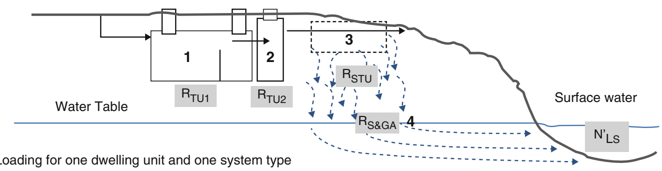


Fig. 13.23 Illustration of the decentralized system components and processes potentially relevant to a particular setting and estimating nutrient loads

13.87



Loading for one dwelling unit and one system type

$$N'_{LS} = N_S\,(1\text{-}R_{TU1})(1\text{-}R_{TU2})(1\text{-}R_{STU})(1\text{-}R_{S\&GA})$$

$$e = (1\text{-}R_{TU1})(1\text{-}R_{TU2})(1\text{-}R_{STU})(1\text{-}R_{S\&GA})$$

e= a combined export coefficient for all compartment for one dwelling unit reflecting factors affecting removal in each compartment

Loading from multiple dwelling unit, multiple system types

$$N_{LS} = \sum_{i=1}^{ST}\left\{\sum_{j=1}^{DU}\left[(N_S)(1\text{-}R_{TU1})(1\text{-}R_{TU2})(1\text{-}R_{STU})(1\text{-}R_{S\&GA})\right]\right\}$$

Component	Parameter	Comment
1. Treatment unit 1 (e.g., septic tank)	R_{TU1}	Literature data is available on removal efficiencies of many confined unit operations
2. Treatment unit 2 (e.g., intermittent sand filter)	R_{TU2}	
3. Soil-based treatment unit (e.g., infiltration trenches)	R_{STU}	STUMOD estimates transformation and removal in an unsaturated soil profile
4. Deeper vadose zone and groundwater	$R_{S\&GA}$	HPS combined with STUMOD estimates removal in the vadose zone and groundwater

Fig. 13.24 Hypothetical illustration of nutrient removal efficiencies in different decentralized system components as it relates to estimating nutrient loads to a surface water

13.88

$$N_{LS} = \sum_{i=1}^{ST} \left\{ \sum_{j=1}^{DU} [(N_S)(1 - R_{TU1})(1 - R_{TU2})(1 - R_{STU})(1 - R_{S\&GA})] \right\} \quad (13.31)$$

Where:

N_{LS} = nutrient load from a land unit to the edge of stream (lb-N or lb-P per day)
N_s = nutrient load from a source (e.g., house) (lb-N or lb-P per day)
R_{TU1} = fractional removal of N or P in a 1st treatment unit (e.g., septic tank)
R_{TU2} = fractional removal of N or P in a 2nd treatment unit (e.g., sand filter)
R_{STU} = fractional removal of N or P in a soil-based treatment unit (e.g., infiltration) trenches
S&GA = fractional removal of N or P in subsurface soil and groundwater by attenuation during water movement from the STU boundary to the edge of stream
ST = system type 1, 2, 3,. . .
DU = dwelling units with system type i
Note: 1—R can be viewed as an "export coefficient"

13.89

Table 13.13 Example of a hypothetical situation and the nitrogen loading from decentralized systems that reaches the edge of a stream

System types[a]	Loading from a single dwelling unit (DU) (lb-N/ year per cap w/2.5 cap/DU)	Dwelling units in the land unit with each system type	Fractional removal of N in treatment unit 1 (TU1)	Fractional removal of N in treatment unit 2 (TU2)	Fractional removal of N in a soil-based treatment unit (STU)	Fractional removal of N by soil & ground water attenuation (S&GA)	N loading to the edge of stream[b] (lb-N/ year)
1	24.7	50	0.1	Not appl.	0.4	0.5	333
2	24.7	20	0.1	0.2	0.4	0.5	107
3	24.7	5	0.1	0.6	0.2	0.5	18
Total		75				N_{LS} = 458[c]	

[a]System 1. Septic tank (TU1)—infiltration trenches (STU); System 2. Septic tank (TU1)—intermittent sand filter (TU2)—drip dispersal (STU); System 3. Septic tank (TU1)—N-removal biofilter (TU2)—infiltration bed (STU).
[b]The N loading to the edge of the stream could be reduced by nitrogen removal in the edge of stream zone or during transport in the stream. [c]For 75 DU the source load = 1852 lb-N/year and the edge of stream load N_{LS} = 458 lb-N/year, which represents an overall average N removal of about 75 % for Land unit 1.

13.90

13-6. Summary

- Reduction of N and/or P in decentralized systems
  - Can be required to meet discharge limitations in some settings
  - Can be used to recover nutrients for their fertilizer value
- Treatment options can be selected for a given goal, e.g.,:
  - N removal of 40–70 % can be achieved by biological nutrient removal, porous media biofilters, or landscape drip dispersal
  - $N \leq 3$ mg-N/L can be achieved by porous media biofilters followed by denitrifying biofilters
  - $P < 0.7$ mg-P/L can be achieved by soil treatment units, landscape drip dispersal, P-sorptive filters, or chemical methods
- Recovery options can yield a fertilizer and soil amendment, e.g.,:
  - Irrigation benefits of N and P in land applied wastewater effluents
  - Recovering N and P by urine diversion, recovering P by struvite precipitation or via P-sorptive media

13.91

13-7. Example Problems

- 13EP-1. Calculating the alkalinity needed for nitrification
  - Given information
    - Convenience store with a design $Q = 1000$ gal/day
    - Septic tank effluent is being treated by extended aeration to achieve removal of BOD_5 (<30 mg/L) plus nitrification of NH_4-N (<5 mg-N/L)
    - STE characteristics: $BOD_5 = 150$ mg/L, NH_4-N = 50 mg-N/L, pH = 6.5, alkalinity = 350 mg/L as $CaCO_3$
  - Determine
    - If the concentration of alkalinity in the STE will be sufficient to sustain nitrification to achieve 90 % removal of NH_4-N if the minimum alkalinity for the process is 35–50 mg/L as $CaCO_3$

13.92

- Solution
  - Consider the stoichiometry of the nitrification reaction

$$NH_4{}^{+} + 2O_2 \rightarrow NO_3{}^{-} + 2H^{+} + H_2O \tag{13.5}$$

    - * For each mol of $NH_4{}^{+}$ converted (14 g N/mol), 2 mol of H+ are produced which requires 1 mol of alkalinity (if expressed as $CaCO_3$ (100 g/mol))
    - * This equates to 7.1 mg/L $CaCO_3$ alkalinity per 1 mg-N/L of $NH_4{}^{+}$ converted (100/14 = 7.1)
  - To convert the STE NH_4-N from 50 mg-N/L to the goal of 5 mg-N/L (90 % removal) requires removal of 45 mg-N/L of NH_4-N
  - According to Equation 13.5, this results in consumption of 320 mg/L of alkalinity expressed as $CaCO_3$
    - * 320 mg/L required is <350 mg/L in the wastewater treated
    - * The residual is low (30 mg/L) given the minimum alkalinity needed is typically 35–50 mg/L

13.93

- 13EP-2. Sizing of a wood chip denitrifying biofilter
  - Given information
    - High school with a design Q = 1000 gal/day
    - A denitrifying biofilter is being designed using saturated upflow through a bed of wood chips with the goal of producing an effluent with total N ≤ 3 mg-N/L
    - Treatment before the denitrifying biofilter is a recirculating textile media filter that should produce an effluent with BOD_5 = 20 mg/L, TSS = 20 mg/L and total N = 40 mg-N/L (in the form of $NO_3{}^{-}$ N)
    - Average temperature during operation will be about 15 °C
  - Determine
    - The volume of the denitrifying biofilter (ft^3)
    - The hydraulic retention time (days)

13.94

- Solution
  - Based on literature data select a NO_3^- removal rate (R_{NR})
    - Assume NO_3^- is not limiting and $R_{NR} = 0.31$ lb-N/day/1000 ft^3
      - 0.31 is a typical rate (see Table 13.9), within the range reported for moderate temperatures if NO_3^- is not limiting

$$V_B = \frac{Q_D(C_I - C_E)F}{R_{NR}} \tag{13.20}$$

$$V_B = \frac{\left(\left(1000\,\frac{\text{gal}}{\text{day}}\right)\left(40 - 3\,\frac{\text{mg-N}}{\text{L}}\right)\left(8.34 \times 10^{-6}\right)\right)}{\left(\frac{0.31 \;\; \text{lb-N}}{\text{d} \bullet 1000 \;\; \text{ft}^3}\right)}$$

$$V_B = \frac{0.3085}{0.31} = 995 \;\; \text{ft}^3$$

  - $V_B = 995$ ft^3 could be provided in a rectangular basin with different geometries, e.g.,:
    - L = 16.5 ft, W = 10 ft, D = 6 ft (=990 ft^3)
    - L = 20 ft, W = 12 ft, D = 4.5 ft (=1080 ft^3)
  - Calculate the empty bed volume and hydraulic retention time
    - Assume the effective porosity of the wood chip bed = 0.6

$$V_{B'} = (V_B)(n_e) = \left(995 \;\; \text{ft}^3\right)(0.6) = 597\,\text{ft}^3 \tag{13.21}$$

$$V_{B'} = (Q_D)(\text{HRT}) \tag{13.22}$$

$$\text{HRT} = \frac{V_{B'}}{Q_D}$$

$$\text{HRT} = \frac{597\,\text{ft}^3}{\left(1000\,\frac{\text{gal}}{\text{day}}\right)\left(\frac{1 \;\; \text{ft}^3}{7.48 \;\; \text{gal}}\right)} = 4.5\,\text{days}$$

- 13EP-3. Sizing of a sorptive filter for PO_4^{-3} removal
  - Given information
    - Apartment complex with a design Q = 1000 gal/day
    - A sorptive filter is being designed to reduce PO_4^{-3} to ≤ 0.5 mg-N/L
    - Media is LWA with density = 0.37 g/cm^3 (23.1 lb/ft^3) and PSC = 3 g-P/kg dry wt. of media
    - Treatment before the P sorptive filter is a aerobic treatment unit that is expected to produce an effluent with BOD_5 and TSS = 20 mg/L, and PO_4^{-} = 10 mg-P/L
    - Average temperature during operation will be about 15 °C
  - Determine
    - The volume (ft^3) and diameter of a cylindrical filter with a depth of 6 ft.
    - The mass of P removal media needed (lb) to provide a 1-year period of operation before media replacement

13.97

- Estimate the volume and mass of P sorptive media required

$$V_{SM} = \frac{(V)(C_I)(F)}{(\rho_b)(PSC)} \tag{13.27}$$

$$V_{SM} = \frac{\left(\dfrac{1000\ \text{gal}}{\text{day}}\right)(365\,\text{days})\left(10\dfrac{\text{mg-P}}{\text{L}}\right)\left(8.34\times10^{-6}\right)}{\left(23.1\dfrac{\text{lb}}{\text{ft}^3}\right)\left(0.003\dfrac{\text{lb-P}}{\text{lb}}\ \text{media}\right)}$$

$$V_{SM} = 440\,\text{ft}^3$$

$$M_{SM} = (V_{SM})(\rho_b) \tag{13.28}$$

$$M_{SM} = \left(440\,\text{ft}^3\right)\left(23.1\frac{\text{lb}}{\text{ft}^3}\right)$$

$$M_{SM} = 10,160\ \text{lb}$$

13.98

- Estimate the diameter of the filter vessel
  * $V_{SM} = 440\ ft^3$ (calculated) with 6 ft depth (given)

$$A_{FS} = \frac{V_{SM}}{D_F} \tag{13.29}$$

$$A_{FS} = \frac{440\,ft^3}{6\,ft} = 73.3\,ft^2$$

$$D_F = \sqrt{\frac{A_{FS} \bullet 4}{\pi}} \tag{13.30}$$

$$D_F = \sqrt{\frac{73.3 \bullet 4}{3.1414}} = 9.7\ \ ft.$$

13.99

Chapter 14

Treatment for Pathogen Reduction

14-1. Scope

Pathogen reduction is often a priority requirement for a decentralized wastewater treatment system, particularly if water reuse is planned. Pathogen reduction can be achieved using disinfection technologies that are designed to destroy pathogenic microorganisms in highly treated wastewaters and reduce the risk of infectious disease transmission that could result from exposure during discharge and reuse applications. Disinfection can also be achieved using removal processes in secondary and tertiary treatment unit operations. This section describes the process principles and design of disinfection methods for pathogen reduction in decentralized applications.

14-2. Key Concepts

- Pathogenic microorganisms in wastewaters can include bacteria, viruses, protozoa and helminthes.
  - These microorganisms can cause infectious disease in humans through direct (e.g., ingestion of drinking water or body contact through bathing) or indirect exposures (e.g., consumption of shellfish raised in polluted waters).
  - Disinfection refers to the process of removing or destroying pathogenic microorganisms in a media like water so that the risk of infectious disease transmission through human contact with (directly or indirectly) that media is reduced.
  - The relative ease of achieving disinfection varies for different microorganisms. For example, protozoa are generally more resistant than viruses, which are relatively more resistant than bacteria.

R.L. Siegrist, *Decentralized Water Reclamation Engineering*,
DOI 10.1007/978-3-319-40472-1_14

- Analyzing for specific pathogens is difficult and costly which inspired the concept of using indicator microorganisms. Indicator microorganisms are non-pathogenic and have been used to assess the likely presence or absence of human fecal contamination of water. The presence of an indicator suggests the possible presence of fecal pathogens and vice versa. Fecal coliform bacteria have been used as indicator microorganisms but their relevance to assess the need for and efficiency of disinfection to enable discharge and reuse options continues to be debated. Other microorganisms have been proposed that are believed to be better indicators of the occurrence and fate of infectious bacteria, viruses and protozoa.

■ Disinfection agent technologies are technologies that have the primary function of destroying pathogens. Common disinfection agent technologies that have been used in decentralized systems include chlorination, ultraviolet light irradiation, and ozonation.

- Destruction mechanisms include: oxidation of cellular materials, destruction of cell walls, nucleic acids and C:N bonds, and disruption of replication or death.
- A number of factors can potentially affect the effectiveness of disinfection agent technologies used for different types of wastewaters
  - Initial mixing is critical to the effectiveness of chemical disinfection methods.
  - Contact time is needed for a minimum period of exposure to the disinfectant agent.
  - Disinfectant dose is required for effective disinfection.
  - Temperature affects reactivity and pH affects ionization of disinfectant chemicals.
  - Type of microorganisms determines their resistance to disinfectants.
  - Number of organisms and their occurrence as dispersed vs. clumped together can affect their susceptibility to disinfection.
  - Wastewater characteristics, the effects of which are significant and vary depending on the disinfectant technology used, can be very important to disinfection effectiveness.

■ Disinfection can also be achieved using membrane filtration or to a certain extent in secondary and tertiary treatment unit operations (e.g., porous media biofilters, soil treatment units).

- Removal and destruction mechanisms can include predation, filtration, sunlight, temperature, salinity, and aging.
- Pathogen reduction that is achieved in this manner is often referred to as “natural disinfection”.

- Disinfection of treated wastewaters is normally slower and can be less effective when compared to that which occurs in clean water. This is due to the interactions of the disinfectant agent with different wastewater characteristics. Prime examples of the adverse effects that wastewater characteristics can have include:
  - Chlorination—The chorine demand of reduced substances and the chlorine reactions with NH_3.
  - Ultraviolet light—The shielding of microorganisms by total suspended solids.

  To avoid these adverse interactions, disinfection technologies that rely on chlorination or ultraviolet light are normally used to disinfect high quality effluents such as produced by an aerobic treatment unit or porous media biofilter.

- Disinfection of higher strength wastewaters such as septic tank effluent or even septage can be achieved using aggressive chemical (e.g., ozone, lime or peracetic acid) or thermal methods (e.g., high temperature drying or incineration). Application of these methods to higher strength wastewaters is often done in batch mode rather than under flow-through conditions.

- Chlorine has been the most common disinfectant agent used in the United States for a long time.
  - Adding Cl_2 gas or $Ca(OCl)_2$ solids or NaOCl liquids to water produces hypochlorous acid (HOCl), which dissociates in water based on pH. Chlorine disinfection is strongly influenced by pH with substantially greater effectiveness at pH < 7, since the relative percentage of HOCl is higher and HOCl is a much more effective disinfectant agent than OCl^-.
  - The chlorine dose required for disinfection is defined by the concentration (C) of chorine residual in the wastewater (typ. 3–5 mg/L) multiplied by the contact time (T) (typ. 15–30 min).
  - The chlorine added for disinfection has to be sufficient to provide a required dose after accounting for the chlorine demand of other constituents in the wastewater plus the chlorine that naturally decays over time. Lower concentrations of chlorine can be added to achieve an effective dose for disinfection of higher quality effluents (e.g., 10 mg/L for porous media biofilter effluent compared to 40 mg/L for septic tank effluent).
  - For many decentralized applications chlorination is often implemented using calcium hypochlorite tablets or sodium hypochlorite liquids. In smaller decentralized systems, it can be difficult to maintain the proper dose. Also, depending on the water quality, chlorination can produce potentially toxic disinfection byproducts (e.g., trihalomethanes).

- Ultraviolet light (UV) has germicidal properties, which have been known for a long time. However, the use of UV for disinfection has evolved over the past 20 years or so.
  - UV disinfection is achieved using special lamps that produce UV light in the proper wavelength range (typically 254 nm) and intensity to ensure transmittance into the wastewater to be disinfected.
  - The UV dose required is defined by the UV intensity (I) and contact time (T) (e.g., 10–40 mW-s/cm^2). Lamp intensity declines with use, often reaching 70 % of the initial output after about 10,000 h. In addition, effective intensity (I_E) can be reduced by deposition of scale on the quartz sleeves used to protect the UV lamps.
  - UV disinfection technologies for decentralized applications are available from several companies. They are self-contained units and often have automatic lamp cleaning and intensity detectors with alarms.

- Ozone is a powerful oxidant that can be used as a disinfectant.
  - The ozone dose required is typically satisfied with ozone concentrations of 1–5 mg/L and contact times of 4–10 min.
  - Ozone is less susceptible to interferences associated with the wastewater characteristics compared to the other disinfectants.
  - Ozone is unstable and must be generated onsite and used immediately. It is also corrosive so materials of construction have to be carefully selected.
  - As a powerful oxidant ozone can destroy dissolved organic matter including many trace organic compounds and it can also remove compounds that cause color, taste and odor.

- Membrane filters do not involve a disinfection agent *per se*, but they can achieve very high pathogen reductions by filtration.
  - Membranes can be manufactured with different pore sizes to reject particles of different sizes, for example: 50–2000 nm by microfiltration, $>$3–100 nm by ultrafiltration, $>$1–6 nm by nanofiltration, and $>$0.1–1 nm by reverse osmosis.
  - Given the typical sizes of pathogenic microorganisms they can be separated from the liquid by membrane filtration. As long as the membrane is intact (i.e., not torn or ruptured) and has a water flux through it, microorganisms should be removed and retained.
  - The ultimate fate of the retained microorganisms depends on operating conditions and die-off and inactivation processes.

- Alternative disinfectant agents (i.e., other than chlorine, UV light, or ozone) exist and they have been, or could be, considered for use in decentralized systems.
  - Alternatives involve different chemicals or energy sources, including: peracetic acid, iodine or bromine, potassium permanganate, calcium

or ammonium hydroxide, urea, electrochemical, gamma radiation, photocatalytic, or heat.

- These agents have different modes of action on microorganisms and different potential benefits and limitations for use in disinfection technologies for decentralized applications.
- As of this writing, most of these agents have experienced relatively limited use in disinfection technologies in decentralized applications.

■ Peracetic acid and hydroxide chemicals are of interest as alternatives for potential application in decentralized systems because they have the ability to achieve disinfection in wastewaters including higher strength wastewaters and sludges and don't appear to produce toxic disinfection byproducts.

- Peracetic acid is an quaternary mixture of peracetic acid, hydrogen peroxide, acetic acid and water. The peracetic acid concentration required for disinfection of high quality secondary effluent (e.g., aerobic treatment unit or porous media biofilter effluents) is similar to ozone (e.g., 1–7 mg/L) but the contact time is relatively longer (e.g., 30 min or more). Disinfection of fecal solids has been achieved with peracetic concentrations of 0.5–1.0 % by wt. and contact times of 1–12 h.
- Calcium hydroxide ($Ca(OH)_2$) is a white powder produced by mixing calcium oxide (lime) with water. Ammonium hydroxide (NH_4OH) is a solution of ammonia in water. When either hydroxide is mixed into a liquid the pH can be elevated. For disinfection purposes, sufficient hydroxide is added to elevate the pH to 12 or higher for a period of at least 30 min. Lime has been used for disinfection of wastewater and for waste solids and sludges. It takes about 20–25 lb. of lime per 1000 gal which equates to a concentration of about 3000–5000 mg/L of $Ca(OH)_2$.
- Potential drawbacks to the use of peracetic acid or hydroxides in decentralized systems include the high dose requirements, high chemical costs, and the safety risks associated with reactive chemical use.

■ If disinfection is required for a particular project based on a planned discharge or reuse option, selection of the optimal disinfection technology to use will be based on the specific features of the project. Design and implementation considerations include:

- Disinfection efficiency required for the planned discharge or reuse option.
- Characteristics of the wastewater (or impaired water) to be disinfected and amenability to different disinfection technologies.
- Attributes of the different disinfection technologies relative to the project circumstances and goals.

- Project location and resources available (e.g., access for delivery, power, operation and maintenance requirements and staff).
- Specific design considerations for a particular disinfection technology (e.g., dose, contact time, contactor, management of unreacted disinfectant).

14-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 14 are presented in the Slides section.

14-4. Terminology

Terminology introduced and used in Chap. 14 is defined below.

Chlorination—Process of adding chlorine or hypochlorite to water to achieve disinfection of pathogenic microorganisms. Breakpoint chlorination refers to the condition where all added chlorine results in a free residual in the water or wastewater to which it is added.

Contact—Refers to the process of bringing a disinfectant agent and microorganism into intimate proximity of each other where the agent can act on or interact with the microorganism.

Die-off—Refers to the death of a microorganism due to natural causes.

Disinfection—Refers to the process of destroying or removing pathogenic microorganisms in a media like water so that the risk of infectious disease transmission through human contact with that media (direct or indirect) is reduced. Example processes that destroy pathogens include chlorination, ultraviolet light irradiation, and ozonation. An example of a process that can remove pathogens from water is membrane filtration. See also Natural disinfection.

Disinfection agent—Refers to a physical or chemical substance or energy source that destroys the ability of a microorganism to cause infectious disease.

Disinfection by products (**DBPs**)—Refers to toxic chemicals that can be formed as a result of chemical reactions that occur when a disinfection technology is applied to a water or wastewater. The nature and concentrations of DBPs depends on the disinfectant and water quality characteristics. Trihalomethanes and haloacetic acids are examples of DBPs that are commonly produced during chlorination of waters that contain dissolved organic carbon.

Dose—Refers to the concentration or intensity of a disinfectant agent in a media to be disinfected and the length of time contact occurs.

Free radical—Refers to an atom or molecule that has a single unpaired electron in an outer shell which causes it to be highly reactive as an oxidant. Hydroxyl radicals (HO•) are produced when ozone is added to water.

Inactivation—Refers to the loss of infectivity of a pathogenic microorganism by one or more mechanisms.

Indicator microorganisms—Refers to a group of microorganisms that are used to indicate the possible presence of human pathogens. Indicator microorganisms are not human pathogens but they are shed by humans in large numbers and are relatively easy to analyze for (e.g., fecal coliform bacteria). Presence of an indicator organism in water suggests that the water may have been impacted by human wastes and could contain human pathogens.

Intensity—Refers to the amount of ultraviolet light energy transmitted into a media (e.g., water, wastewater, etc.) to be disinfected. Effective intensity accounts for the aging of a UV lamp and the transmittance into the media to be disinfected (which is less than 100 %).

Lime—Common name for calcium oxide (CaO).

Natural disinfection—Refers to the destruction of pathogenic microorganisms by die-off and predation mechanisms. This is typically applied to treatment unit operations that are not specifically designed as disinfection agent technologies.

Oxidation—A chemical reaction in which an oxidant species accepts an electron(s) and a reductant species gives up an electron(s).

Ozonation—Refers to the process of dissolving ozone gas in water, which leads to generation of free radicals (e.g., hydroxyl radicals) that are powerful oxidants.

Ozone—In this chapter, refers to ozone (O_3) as a powerful oxidant and disinfectant. O_3 destroys cell walls, nucleic acids, and C:N bonds.

Pathogen—An agent such as a living microorganism or particle that can cause disease. Pathogens that can cause disease in humans include a variety of bacteria, virus, protozoa, fungi, and helminthes.

Peracetic acid (**PAA**)—A quaternary mixture of peracetic acid, hydrogen peroxide, acetic acid, and water.

Predation—Refers to the process of bacteria being killed by protozoa.

Transmittance—In the context of disinfection, transmittance refers to the ability of ultraviolet light to penetrate into water, wastewater, or other impaired waters.

Ultraviolet light irradiation—A disinfection technology used to destroy pathogenic microorganisms. Radiation around 260 nm penetrates the cell wall and is absorbed by cellular materials (DNA, RNA) and prevents replication or causes death of the microorganism.

14-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 14 are listed below.

BOD	Biochemical oxygen demand
Cl_2	Chlorine gas
COD	Chemical oxygen demand
CT	Concentration multiplied by time of contact
DBP	Disinfection byproducts
EPA	Environmental Protection Agency
HOCl	Hypochlorous acid
HRT	Hydraulic retention time
MPN	Most probably number
NaOCl	Sodium hypochlorite
NCl_3	Trichloride
NH_2Cl	Monochloroamine
$NHCl_2$	Dichloroamine
NTU	Nephelometric turbidity units
O_3	Ozone
OCl^-	Hypochlorite ion
PAA	Peracetic acid
ppm	Parts per million
T	Contact time
TSS	Total suspended solids
UV	Ultraviolet light
UVT	Ultraviolet light transmittance

C_A	Concentration of chlorine added to the wastewater being disinfected
$Ca(OH)_2$	Calcium hydroxide
C_{CS}	Concentration of chlorine in the wastewater being disinfected
C_{DC}	Chlorine decay with time
C_{DM}	Chlorine demand of the wastewater being disinfected
C_O	Concentration of ozone in the wastewater being disinfected
C_R	Chlorine residual required in the wastewater being disinfected
D	Dose of a disinfectant agent
I	Intensity of ultraviolet light irradiation
I_E	Intensity of UV radiation accounting for lamp aging and transmittance
N	Number of microorganisms present
N_o	Number of microorganisms present initially
Q_C	Rate of chlorine addition
Q_D	Design daily flow rate
V_C	Volume of contact basin

14-6. Problems

14.1. Of the following statements, which are true concerning disinfection for pathogen destruction? Select all that apply.
(a) Disinfection involves destruction of all microorganisms.
(b) Effectiveness of a particular disinfection method can differ for different pathogens.
(c) Disinfection using UV light can be effectively applied to septic tank effluents.
(d) Shielding of microorganisms can occur in effluents with high TSS levels.
(e) Pathogenic microorganisms can be partially removed in an intermittent sand filter.

14.2. Of the following statements, which are true concerning disinfection for pathogen destruction? Select all that are true.
(a) Bacteria are more difficult to remove by disinfection compared to virus.
(b) Shielding of microorganisms can affect disinfection of effluents with high TSS levels.
(c) The chlorine addition required is affected by the effluent concentration of NH_4^+ nitrogen.
(d) Disinfection using ozone requires onsite generation of O_3 gas.
(e) Disinfection using UV light can be effectively applied to biosolids.
(f) Pathogenic bacteria can be completely removed in a membrane bioreactor.

14.3. Give an example to illustrate what is meant by natural disinfection.

14.4. In general, which of the following pathogens is the most difficult to destroy by a disinfection technology like chlorination: hepatitis A virus, *Salmonella* bacteria, or *Giardia* parasites?

14.5. What wastewater characteristic makes it virtually impossible to achieve high levels of disinfection of septic tank effluents using ultraviolet light (e.g., 99.9999 % destruction)?

14.6. For the following conditions, indicate which disinfectant would be most likely to have the greatest effectiveness for destroying virus: (1) Chlorine, (2) UV light, or (3) Ozone. Use all choices, but use each one only once.
(a) Wetland effluent: (BOD_5 and TSS = 30 mg/L; Total N = 40 mg/L; NTU = 10; pH = 6.5).
(b) Sand filter effluent: (BOD_5 and TSS = 10 mg/L; Total N = 20 mg/L NTU = 3; pH = 8.5).
(c) Aerobic unit effluent: (BOD_5 = 30 mg/L; TSS = 25 mg/L; Total N = 20 mg/L; NTU = 10; pH = 6).

14.7. Accounting for chlorine demand and decay (3 and 2 mg/L, respectively), what residual will be present if the chlorine addition is 6 mg/L and is this normally effective for disinfection purposes?

14.8. A recirculating sand filter was used for a small resort and after disinfection using a liquid feed chlorination unit, the effluent was discharged to a local stream. Based on the results of monthly monitoring during the first year of operation it was discovered that the level of *E. coli*. in the disinfected effluent was above the discharge permit ($>$200 org/100 mL). Which two of the following are the most reasonable explanations for the deficient disinfection performance?

(a) The average daily flow was 50 % higher than the design daily flow.
(b) The RSF effluent temperature was colder than expected (8 °C vs. an expected 10 °C).
(c) The pH of the effluent from the RSF was considerably higher than the anticipated pH 7.
(d) The concentration of nitrate in the RSF effluent was nearly 50 mg-N/L.
(e) The BOD_5 in the influent to the RSF was much higher than expected.

14.9. Packaged media biofilter effluent needs to be disinfected before reuse for turf irrigation at a local apartment complex. Chlorination is being compared to ultraviolet light disinfection to treat a design flow, $Q_D = 3000$ gal/day. Given the following information, provide answers to the questions below.

Given information and assumed values: chlorine residual required = 4 mg/L during a contact time = 30 min, chlorine demand = 3 mg/L and decay = 2 mg/L. Intensity of a new UV lamp (output at the lamp surface) = 40 mW per cm^2 and the hydraulic retention time in the UV tube contactor = 0.010 min.

(a) Chlorine dose = __________ mg min/L
(b) Chlorine addition required = ______ mg/L
(c) Volume of the chlorine contact chamber = ________ gal
(d) Ultraviolet light dose = ________________ mW s/cm^2
(e) Volume of the contact basin after the UV light system = __________ gal

14.10. Name a common problem that can occur with tablet feed chlorinators.

14.11. A local technology developer was working on a new, simple system to reclaim graywater for use in lawn watering. The new system included sedimentation followed by disinfection using a UV lamp unit. If you were asked whether you thought the system would be effective in reducing pathogenic bacteria levels by 99.99 % or more, what would your response be and why?

14.12. Disinfection of a recirculating sand filter effluent with ultraviolet light was required to produce an effluent with $\leq$2.2 coliforms per 100 mL. The UV system chosen was able to achieve this level of disinfection initially, but after about 9 months of routine operation, it was only able to consistently achieve $<$20 coliforms per 100 mL. Give two plausible explanations for the performance of the UV system and give one suggestion as to what might be done to meet the $\leq$ 2.2 requirement.

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Slides of Chapter 14

Decentralized Water Reclamation

Chapter 14: Treatment for Pathogen Reduction

Contents

14.1

14-1. Introduction

■ Pathogen reduction

- Selective destruction of pathogen microorganisms can be achieved by disinfection agents such as chlorine, ultraviolet light, or ozone
- Other unit operations can also achieve high levels of pathogen reduction via engineered or natural processes (Fig. 14.1)

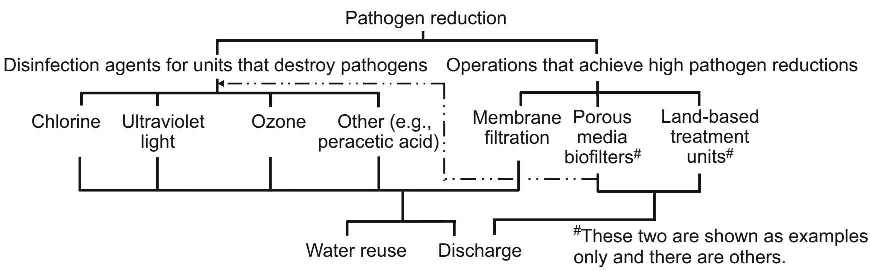


Fig. 14.1 Classification of pathogen reduction technologies and unit operations

14.2

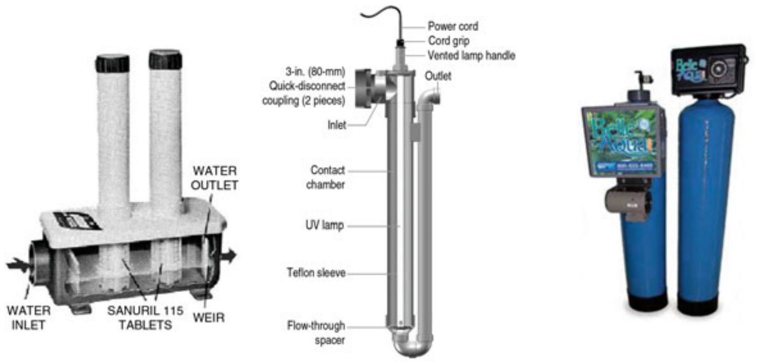
- Basic features of a disinfection agent technology
  - Disinfection agent technologies (hereafter referred to as just disinfection technologies) can include flow-through and batch operating modes
  - Flow-through disinfection technologies
    - Flow-through disinfection technologies that are commonly deployed in decentralized systems are listed in Table 14.1 and shown in Fig. 14.2
    - Wastewater effluent of secondary or better quality is contacted with a disinfectant agent (e.g., chlorine, UV light, ozone) for a relatively short period of time
      - Hydraulic retention times (HRTs) are seconds to minutes
    - During contact the disinfectant agent destroys exposed pathogenic microorganisms by different mechanisms depending on the agent used

14.3

Table 14.1 Characteristics of disinfectant technologies most commonly used in decentralized systems

Disinfectant agent	Available form—Method of contact	Considerations for use in onsite and decentralized system applications
Chlorine	$Ca(OCl)_2$ Solid tablets dissolved into effluent	Corrosive and toxic and can cause formation of by-products Requires a tablet feed system and dose control is difficult Requires a contact chamber or similar basin (e.g., HRT = 10 min) Effectiveness can depend on water quality
	NaOCl Liquid feed into the effluent	Corrosive and toxic and can cause formation of by-products Requires a metered chemical feed system Requires a contact chamber or similar basin (e.g., HRT = 10 min) Effectiveness can depend on water quality
Ultraviolet light	UV light Irradiation of effluent	Non-corrosive, nontoxic, and does not form by products Requires a UV lamp which needs cleaning and replacement Lamp aging and fouling can reduce effectiveness Effectiveness can depend on water quality
Ozone	O_3 Air w/O_3 gas injected into effluent	Corrosive and toxic Requires an O_3 generator onsite and gas injection unit Effectiveness can depend on water quality

14.4



Tablet chlorinator unit
www.wastewaterdepot.com/sanuril_100.html.

Ultraviolet light unit
www.orenco.com/sales/choose_a_system/UV_disinfection/index.cfm

Ozonation unit
www.ozonepurewater.com/cmpltsystm.html.

Fig. 14.2 Examples of commercial options to implement flow-through disinfection technologies for decentralized applications

14.5

- Batch disinfection operations
  - A classic example of batch disinfection is to add bleach (which contains high levels of chlorine) to a diaper pail where soiled diapers are placed for storage prior to laundry washing
  - Batch disinfection has been applied to higher strength wastewater or waste typically based on:
    - Adding a chemical agent (e.g., peracetic acid)
    - At a high concentration (e.g., 1 g/L)
    - For a relatively long contact time (e.g., hours to days)
  - Batch disinfection of primary effluents (e.g., septic tank effluents), septage or fecal sludges has been attempted by this type of chemical addition
    - One motivation for this has been to enable safe land application of partially treated or untreated wastes to achieve beneficial recovery of organic matter and nutrients that can help support agriculture in developing regions

14.6

- Basic features of other treatment operations that can achieve pathogen reductions
  - Other flow-through treatment operations can achieve high pathogen reductions but without using a disinfectant agent
    - For example, very high pathogen reductions can be achieved by membrane filtration (e.g., nanofiltration)
  - During treatment in many widely used unit operations (e.g., porous media biofilters, constructed wetlands, soil treatment units), pathogens can be removed and potentially destroyed
    - Removal and destruction can be due to predation, filtration, sunlight, temperature, salinity, aging
    - Pathogen reduction that occurs in these systems (e.g., constructed wetlands or soil treatment units) is often referred to as "natural disinfection"

14.7

- Where are disinfection technologies used?
  - Disinfection agent technologies (e.g., chlorine, UV, ozone) are often used where a high degree of pathogen reduction is needed before release of a treated wastewater effluent
  - This need is based on the level of potential human exposure and the risk of infectious disease transmission, e.g.:
    - Where there are plans for water reuse of treated effluent
      - Water reuse for landscape irrigation
      - Water reuse for nonpotable functions like toilet flushing, cooling, and ornamental fountains
    - Where treated wastewater effluent may reach a surface water and there is a heightened concern for infectious disease transmission directly or indirectly
      - Directly—e.g., contact with bacteria during swimming
      - Indirectly—e.g., ingestion of virus via eating shellfish

14.8

14-2. Treatment Performance

- Disinfection technology performance
  - Performance = the ability to reliably destroy pathogenic microorganisms in a wastewater effluent to enable it to be safely used for an intended discharge or reuse option
- Use of indicator organisms
  - Analyzing for specific pathogens in disinfected effluent can be difficult and expensive
  - This inspired the concept of using nonpathogenic microorganisms that are easier to analyze for as "indicators"
    - The indicator concept is based on selecting a microorganism that is typically present in wastewaters to use as an indicator of pathogen presence
      - Presence of indicator → possible presence of pathogens
      - Absence of indicator → likely absence of pathogens

14.9

- Historically, coliform bacteria have been used as indicators
  - Coliform bacteria are in the intestinal track of humans and each human discharges 100–400 billion coliforms/day
  - Coliforms can be destroyed during disinfection
- Current thinking on what to use as indicators
  - Continuing debate over which microorganisms are best to use
  - Indicators need to represent not only human infectious bacteria but also viruses and protozoa
  - For assessing disinfection, need indicators that are at least as resistant to disinfection as the pathogens of interest are
  - A suite of potential indicator microorganisms is shown in Table 14.2

14.10

- Disinfection efficiency and performance
  - Disinfection effectiveness can be assessed by two approaches
    - One approach uses the level of reduction in microorganisms as a direct result of disinfection
      - Reductions are often expressed on a log basis
        - 1 log = 90 %, 2 log = 99 %, 3 log = 99.9 % etc.
      - Disinfection that results in the number of *E. coli* dropping from $C_I = 100{,}000$ to $C_E = 100$ org/100 mL yields a 99.999 % reduction or 5-log effectiveness
      - The reduction efficiency (R_E) can be calculated using Eq. 14.1 based on the flow regime in Fig. 14.3
    - As an alternative to, or in addition to, R_E, another approach uses the concentration of microorganisms present in the treated effluent (C_E) after disinfection (Fig. 14.3)
      - For example, disinfection must yield ≤ 2 *E. coli* per 100 mL

14.11

Table 14.2 Examples of indicator microorganisms used for water quality assessment (Ashbolt 2014)

Pathogen group	Example pathogen	Indicator microorganism that represents presence/absence of the pathogen group
Bacteria	Campylobacter	*E. coli*, Coliphages[a]
Viruses	Noro virus	Bacteriophages[b]
Protozoa	Cryptosporidium	*Clostridium perfringens* spores

[a]Coliphages are virus that infect *E. coli* bacteria.
[b]Bacteriophages infect bacteria such as *E. coli* or *Salmonella*.

$$R_E = \left[\frac{C_I - C_E}{C_I}\right] \text{ x } 100\% \qquad (14.1)$$

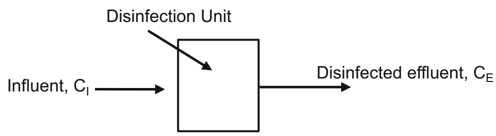


Fig. 14.3 Illustration of an approach to assess pathogen destruction efficiency in a disinfection unit (*Note*: Microorganisms (C_I and C_E) can be indicators or actual pathogens of interest)

14.12

- Disinfection technology efficiencies that are achievable for pathogens of potential concern
  - Targets such as $\leq$2 *E. coli* per 100 mL or 99.999 % destruction can be achieved with one or more disinfection technologies
  - However,… achieving effective disinfection depends on design and implementation of the disinfection technology
    - Choosing a disinfection technology suited to the wastewater effluent quality characteristics
    - Choosing proper values for the disinfectant dose
    - Ensuring proper operation of the disinfectant technology
    - Providing maintenance as required

14.13

14-3. Principles and Processes

- Disinfection targets pathogenic microorganisms (Table 14.3)

Table 14.3 Examples of pathogens that can be present in wastewater effluents

Class (size)	Organism	Disease	Infectious dose
Bacteria (0.5–2 μm)	*Escherichia coli*	Gastroenteritis	
	Salmonella typhi	Typhoid fever	10^4–10^6
	Shigella	Bacillary dysentery	180
	Vibrio cholerae	Cholera	10^3–10^7
Viruses (0.03–0.10 μm)	Andenovirus (31 types)	Respiratory disease	
	Enteroviruses (72 types)	Gastroenteritis, meningistis,…	1–10
	Hepatitus A	Infectious hepatitis	
	Norwalk agent	Gastroenteritis	
Protozoa (4–15 μm)	*Cryptosporidium parvum*	Cryptosporidiasis	
	Giardia lamblia	Giardiasis	<10
Helminthes (20–80 μm)	Ascaris lumbricoides	Ascariasis (round worm)	1–10

14.14

- Disinfection agents and their mechanisms
  - Table 14.4 gives mechanisms of several disinfectant agents

Table 14.4 Disinfection agents and mechanisms of destroying pathogenic microorganisms

Disinfection agent	Mechanisms	
	Type	Description
Chlorine	Chemical	Oxidation. Reactions with available chlorine. Protein precipitation. Modification of cell wall permeability. Hydrolysis and mechanical disruption
Ultraviolet light	Radiation	Photochemical damage to RNA and DNA within an organism. Nucleic acids absorb energy in 240–280 nm range
Ozone	Chemical	Oxidation by O_3 and free radicals. Destruction of cell walls. Damage to constituents of nucleic acid. Breakage of C—N bonds
Peracetic acid	Chemical	Penetration of cell walls and disruption of enzyme systems. Potential direct oxidation of cell materials
Calcium hydroxide	Chemical	$Ca(OH)_2$ can elevate the pH to 12 or above producing hydroxyl ions which are capable of destroying microorganisms

Note: there are other disinfection agents beyond those listed in Table 14.4, but those listed here are mos commonly used.

14.15

- Disinfection dose
  - Disinfection using chlorine, UV light, ozone, or peracetic acid requires a certain dose of the disinfectant agent
  - Dose is determined by the concentration or intensity of an agent and the time of exposure as presented in Eqs. 14.2 and 14.3

$$\text{Dose} = C \times T \tag{14.2}$$

$$\text{Dose} = I \times T \tag{14.3}$$

Where:
C = concentration of a solution-based disinfectant (e.g., mg/L)
I = intensity of ultraviolet light irradiation (e.g., mW/cm^2)
T = contact time (e.g., min)

14.16

- Disinfection dose and relative effectiveness
  - Disinfectant dose required for a pathogen can be compared to the dose for a reference microorganism (e.g., coliform bacteria)
    - This is based on the microorganisms being dispersed and suspended in solution at pH 7–8 and 20–25 °C
  - Different microorganisms are relatively more difficult to destroy by disinfection
    - Total coliform bacteria = 1×
    - Bacteria = 0.5× to 2.5× higher
    - Viruses = 2× to 8× higher
    - Protozoa = 4× to 20× higher

    Increasing resistance to disinfection
  - Relative effectiveness by disinfectant agent varies for the different microorganisms (Table 14.5)
    - e.g., chlorine or ozone are generally more effective than UV for *Cryptosporidium parvum*

14.17

Table 14.5 Relative dose of different disinfectants required for different microorganisms present in wastewater effluents

Microorganism class and examples		Dosage required relative to total coliforms[a]		
		Chlorine	UV	Ozone
Bacteria	Total coliform	1	1	1
	Fecal coliform	0.9–1	0.9–1	0.9–1
	Salmonella typhi	1	0.9–1	
	Staphylococcus aureus	2.5	1.5–2	
Viruses	Andenovirus	0.5–1	0.6–0.8	
	Coxsackie A virus	6–7	0.8–1	
	MS-2 bacteriophage	5–7	6–8	2–4
Protozoa and helminthes	*Cryptosporidium parvum*	8–10	10–20	6–8
	Giardia lamblia	6–8	7–14	4–6
	Nematode eggs		10–12	

[a]Based on single organisms in suspension at pH 7–8 and 20–25 °C.
Source: After Table 12–26 in Crites and Tchobanoglous 1998.

14.18

- Disinfection in clean water
  - 1st-order kinetic models have been used to represent die-off
    - The Chick (1908) model represents simple die-off in clean water (Eq. 14.4)

$$N = N_o e^{-kt} \tag{14.4}$$

    - The Chick-Watson model proposed by Haas and Kara (1984) accounts for a chemical agent being present in the water (Eq. 14.5)

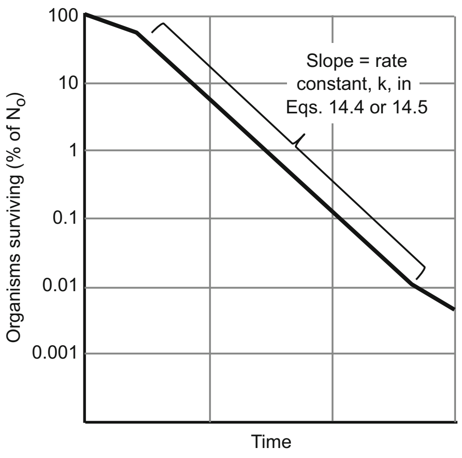
$$N = N_o e^{-kC^n t} \tag{14.5}$$

Where:
N = number of organisms remaining at time, t (#org/L)
N_o = number of organisms present at time = 0 (#org/L)
k = decay constant (either h^{-1} or $mol^{-1}\ h^{-1}$)
C = concentration of chemical agent (mol.)
n = empirical constant (–)
t = contact time (h)

14.19

- The 1st-order die-off of microorganisms in clean water is illustrated in Fig. 14.4

Fig. 14.4 Illustration of a 1st-order die-off of microorganisms in clean water (*Note*: Microorganisms can be indicators or actual pathogens of interest)



14.20

- Disinfection of wastewater
  - Disinfection rate and extent can be very different in wastewater (or other impaired waters) compared to clean water
  - A number of factors can potentially affect the effectiveness of disinfection for different types of wastewaters including:
    - Initial mixing is critical to chemical disinfection
    - Contact time yields period of exposure to the disinfectant agent
    - Disinfectant dose has to be delivered
    - Temperature effects reactivity; ionization of chemicals
    - Type of organisms of varying resistance to disinfectants
    - Number of organisms; occurrence as dispersed vs. clumped
    - The effects of wastewater characteristics are significant, and vary depending on the disinfectant technology used (Table 14.6)

14.21

Table 14.6 Characteristics of wastewater effluents and their potential effects on the efficiency of different common disinfection technologies

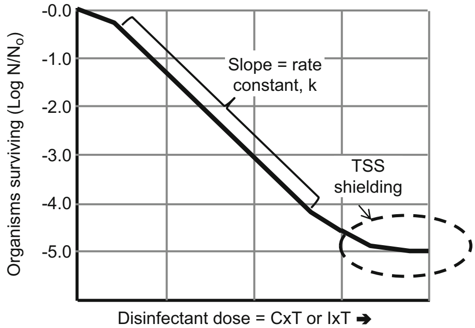
Characteristic[a]	Chlorine	Ultraviolet light	Ozone
TSS	Shielding of microbes	Shielding of microbes; absorption of UV	Shielding of microbes; ozone demand
BOD and COD	Chlorine demand	No or minor effects	Ozone demand
Humic matter	Reduce effectiveness	Strong absorbers of UV	Ozone demand
Ammonium	Combines w/chlorine	No or minor effects	No or minor effects
Nitrite	Oxidized by chlorine	No or minor effects	Oxidized by ozone
Nitrate	No or minor effects	No or minor effects	Can reduce effects
Hardness	No or minor effects	Effects due to precipitation	No or minor effects
Iron	No or minor effects	Strong absorbers of UV	No or minor effects
pH	Affects distribution of hypochlorous acid and hypochlorite ion	Affects solubility of metals and carbonates	Effects rate of ozone decomposition

[a] *TSS* total suspended solids, *BOD* biochemical oxygen demand, *COD* chemical oxygen demand
Source: Based on Table 12–24 (Crites and Tchobanoglous 1998).

14.22

- Shielding of pathogens can inhibit disinfection
  - Disinfection technologies can achieve very high efficiency for "dispersed organisms" in water, but not for "shielded organisms" in wastewaters or other impaired waters
    - TSS can cause shielding and inhibit disinfection (Fig. 14.5)
    - TSS shielding can be overcome by modifying the TSS particle size or removing TSS particles by treatment (e.g., filtration)

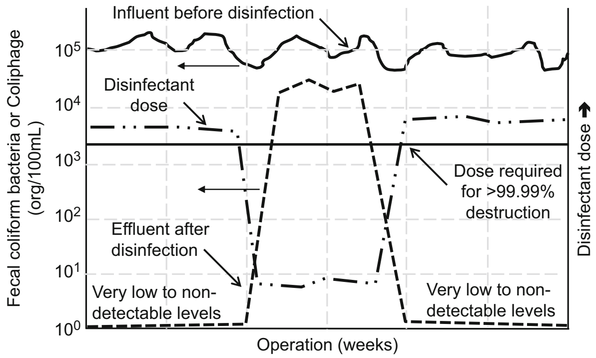
Fig. 14.5 Illustration of a 1st-order die-off of microorganisms in wastewater and shielding that can occur due to TSS (*Note*: microorganisms can be indicators or actual pathogens of interest)



14.23

- Changes in the dose delivered can disrupt disinfection
  - Abrupt changes in dose that can occur (e.g., chlorine feeder malfunction, loss of power to a UV unit, failure of an ozone generator) can reduce disinfection efficiency (Fig. 14.6)

Fig. 14.6 Illustration of an abrupt change in dose that can reduce disinfection efficiency



14.24

- Chlorine as a disinfectant agent
  - Chlorination is the most common disinfectant technology used throughout the world—why?
    - Chlorine possesses many if not most of the desirable attributes of a good disinfectant
    - Chlorine can provide a residual concentration that can sustain disinfection in storage containers and distribution pipelines
    - Chorine and its compounds can destroy pathogens by oxidation of cellular materials
  - Common chlorine compounds used in disinfection
    - Chlorine gas (Cl_2)
    - Chlorine dioxide gas (ClO_2)
    - Calcium hypochlorite solid ($Ca(OCl)_2$)
    - Sodium hypochlorite liquid (NaOCl)

14.25

- Basic chemistry of chlorine disinfection
  - Adding Cl_2 gas or $Ca(OCl)_2$ solids or NaOCl liquids to water produces hypochlorous acid (HOCl)
  - Hypochlorous acid dissociates in water based on pH (Eq. 14.6)

$$\mathrm{HOCl} \leftrightarrow \mathrm{H}^+ + \mathrm{OCl}^- \tag{14.6}$$

    - HOCl does produce acidity, but with the alkalinity in most waters chlorination should not cause a major change in pH
  - The quantity of HOCl plus OCl- present in water equals what is known as the "free" or "available" chlorine
  - The relative % of HOCl vs. OCl^- in water primarily depends on pH
    - At low pH, HOCl predominates
    - At pH 7 and 20 °C, 80 % of free available chlorine is HOCl
  - The germicidal effectiveness of HOCl is 20× to 100× that of OCl^-

14.26

- Chlorine disinfection of wastewater
  - Chlorine added to wastewater reacts in a stepwise fashion
    * HOCl first reacts with readily oxidizable substances (e.g., Fe^{2+}, Mn^{2+}, H_2S, organic matter)
      – In the process, chlorine is reduced to chloride ion (Cl^-)
    * HOCl then reacts with ammonia to form chloramines
      – Chloramines are slow reacting disinfectants

$$NH_3 + HOCl \rightarrow NH_2Cl + H_2O \quad \text{“monochloroamine”} \tag{14.7}$$

$$NH_2Cl + HOCl \rightarrow NHCl_2 + H_2O \quad \text{“dichloroamine”} \tag{14.8}$$

      – Chloramines can be converted to trichloride, N_2, N_2O

$$NHCl_2 + HOCl \rightarrow NCl_3 + H_2O \quad \text{“trichloride”} \tag{14.9}$$

    * After the above reactions occur, the “breakpoint” is reached and continued chlorine addition will yield a residual of free chlorine (Fig. 14.7)

14.27

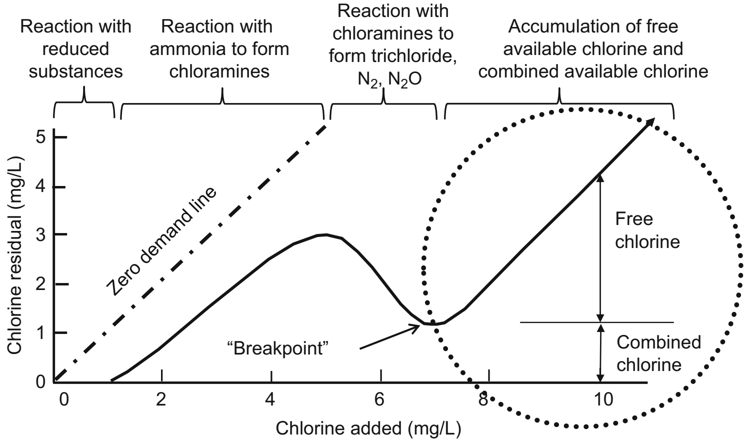


Fig. 14.7 Illustration of breakpoint chlorination during disinfection of wastewater

14.28

- Chlorine dose required for wastewater disinfection

$$\text{Dose} = C_R \times T \qquad (14.10)$$

$$C_A = C_R + (C_{DM} + C_{DC}) \qquad (14.11)$$

Where:
C_R = chlorine residual required in the wastewater (mg/L)
C_A = chlorine addition to the wastewater being disinfected (mg/L)
C_{DM} = chlorine demand of the wastewater (mg/L)
Depends on wastewater type and contact time. e.g.: septic tank effluent = 30–45 mg/L while sand filter effluent = 1–5 mg/L, both during T = 15–30 min
C_{DC} = chlorine decay with time (mg/L)
Decay depends on the time period (e.g., decay can be about 2–4 mg/L during a 60-min period)
T = contact time (min)

14.29

- Chlorine residual required
  - The residual required depends on the microorganisms present, disinfectant agent present, and the contact time
  - Chlorine disinfection effectiveness can be compared
    - * Using the dose required
    - * Using the concentration multiplied by the contact time values
      - – There are referred to as the CT values
  - For example, the CT value required for 99 % destruction of *E. coli* in water varies based on the type of chlorine present
    - * For a 20 min contact time, the CT values range from near 0 to 100 or more:
      - – w/HOCl → CT = 0.15 (e.g., 0.75 mg/L for 20 min)
      - – w/OCl^- → CT = 20 (e.g., 1.0 mg/L for 20 min)
      - – w/NH_2Cl → CT = 100 (e.g., 5 mg/L for 20 min)

14.30

- Approaches for estimating the chlorine residual required
  - Setting the chlorine residual based on past experience
    - Chlorine residual needed is often about 3–5 mg/L for a typical 15–30 min contact time
  - Setting the chlorine residual by modeling specific conditions
    - Example model of White (1992) for secondary effluent[a]

$$\frac{N}{N_o} = \left(\frac{C_R T}{b}\right)^n \tag{14.12}$$

Where:
N = number of organisms remaining at the end of time T
N_o = number of organisms present before disinfection
C_R = chlorine residual remaining at the end of time T
n = slope of experimental curve (typical value = −2.8)
b = value of × intercept when log $N/N_o = 0$ (typical value = 4.0)

[a]*Note:* Based on variability in composition, the coefficients n and b often need to be determined for the particular wastewater to be disinfected

14.31

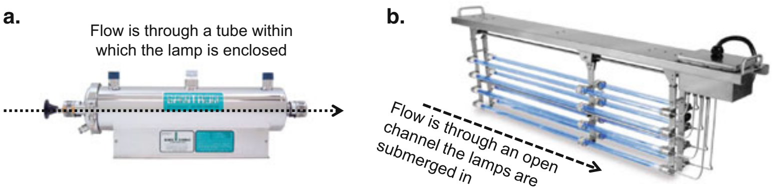
- Dechlorination
  - Residual chlorine can cause problems
    - Taste and odor problems
    - Toxicity to higher life forms
  - To mitigate problems, acceptable levels of residual chlorine are typically set depending on the discharge or water reuse plans
  - Dechlorination involves the removal of chlorine residuals following chlorination
    - Dechlorination can be achieved using various reductants, including:
      - Sulfur dioxide (SO_2)
      - Sodium bisulfite ($NaHSO_3$), Sodium metabisulfite ($Na_2S_2O_5$), or Sodium thiosulfate ($Na_2S_2O_3$)
      - Granular activated carbon
    - Reactions are very rapid so time is not an issue but effective mixing and contact is

14.32

- Ultraviolet light as a disinfectant agent
  - The germicidal properties of ultraviolet light (UV) have been known since the early 1900s
    - UV light is a physical disinfectant
      - Radiation around 260 nm penetrates the cell wall and is absorbed by cellular materials (DNA, RNA)
      - Prevents replication or causes death of the microorganism
  - UV disinfection technologies depend on UV light
    - In the proper wavelength range and at the necessary intensity
    - Effectively transmitted into the water to be disinfected
    - Produced by lamps that can be kept clean and have a long life
  - UV light is typically used to disinfect advanced secondary effluents that have very high clarity to enable transmissivity and virtually no TSS to avoid shielding

14.33

- UV light for disinfection is achieved through special lamps
  - Lamps containing mercury vapor are charged by an electric arc and excitation of mercury results in emission of UV light
  - Lamps include those with:
    - Monochromatic (254 nm) or polychromatic output
    - Relatively lower or higher intensity output (e.g., 1× to 20×)
  - Lamp unit designs vary as shown in Fig. 14.8



Source: www.ultraviolet.com/water/sanitr06.htm.

Source: http://www.calgoncarbon.com/uv/documents/UVC3150Bulletin.pdf

Fig. 14.8 Examples of UV lamp units including (**a**) a single lamp in a flow-through tube reactor and (**b**) a bank of lamp units that can be submerged in a channel through which flow occurs

14.34

- Monochromatic UV lamp systems are commonly used for decentralized system applications
  - * Lower intensity UV systems
    - – Lamps are encased in quartz sleeves and spaced about 3 in. apart (center to center) in the flow field
    - – Lamps are subject to fouling and need to be cleaned periodically, either manually or automatically
    - – Optimal operation occurs at a lamp wall temp. = 40 °C
  - * Higher intensity UV systems
    - – Similar to low-pressure, low-intensity lamp systems, but mercury-indium amalgam is used rather than mercury
    - – Benefits of high intensity UV lamps

      Enable 2–4× more UV output

      Stable operation over a wider temperature range

      Longer lamp life (~25 % longer)
    - – Common for pre-packaged UV units (discussed later)

14.35

- UV dose required for wastewater disinfection
  - UV dose depends on intensity and exposure time (Eq. 14.13)

$$\mathrm{Dose} = \mathrm{I_E} \times \mathrm{T} \tag{14.13}$$

Where:
Dose = disinfectant dose (mW s/cm^2)
I_E = intensity of UV radiation (mW/cm^2) accounting for lamp aging (e.g., 20 % loss) and transmittance into effluent (e.g., 70 %)
T = time of exposure (sec)

  - For a given type of lamp, the intensity (I_E) is determined by the lamp age and the characteristics of the wastewater being disinfected (e.g., turbidity)
  - Contact time (T) is determined by the hydraulic design of the contactor unit in which the UV lamps are housed

14.36

- Estimating the dose required for UV disinfection
  - Dose–response has been studied under lab conditions
    - For example, to achieve 99.999–99.9999 % reduction of dispersed coliform organisms, the UV dose required is 10–40 mW-s/cm^2
  - TSS can cause "tailing" and limit the ability to achieve extremely low levels of viable microorganisms when using UV disinfection
    - Effectiveness of UV systems depends on the fraction of particle-associated organisms
    - TSS of even 10 mg/L can cause shielding of some microorganisms
    - To meet $\leq$2.2 MPN/100 mL for body contact reuse, treated wastewater effluent filtration is required prior to UV disinfection

14.37

- Lamp intensity
  - New lamps have a certain intensity but this declines with use
  - For example, with time, the output from low-pressure, low-intensity lamps can drop from 100 % to about 70 %
    - This is due to two causes
      - Loss of electrons in the lamp
      - Aging of the quartz sleeve enclosing the lamp
  - Useful life before lamp or sleeve replacement
    - Lamp replacement
      - Replacement is needed after about 10,000 h of lamp usage
      - At this point, the lamp output may only be about 70 % of the initial output when it was new
      - Based on continuous usage, replacement could be needed every year or so
    - Quartz sleeve enclosure replacement
      - Replacement can be needed about ever 4–8 years

14.38

- UV intensity within the wastewater being disinfected
  - * UV intensity within the wastewater is affected by:
    - – Lamp configuration
      e.g., depth of flow away from the lamp surface
    - – Transmittance
      e.g., depth of UV penetration into the effluent
  - * Table 14.7 provides example transmittance values for different types of wastewater and drinking water

Table 14.7 Example transmittance values for UV irradiation of treated wastewater effluents

Treated wastewater type	Transmittance (%)
Septic tank effluent	45–67
Secondary effluent	60–74
Sand filter effluent	80–87
Drinking water	80–95

Source: Leverenz et al. (2006).

14.39

- Ozone as a disinfectant agent
  - Ozone (O_3) is a powerful oxidant and disinfectant
    - O_3 destroys cell walls, nucleic acids, and C:N bonds
    - O_3 can destroy compounds that cause color, taste and odors
  - O_3 is unstable and must be generated and used immediately
    - Air or pure O_2 is passed through a gap between closely spaced electrodes and a high voltage causes a corona
      - * 1 O_2 molecule dissociates and recombines with 2 other O_2 molecules to yield 2 O_3 molecules
      - * The O_3 content in the gas stream that is produced:
        - – If air is used: 0.5–3.0 wt.% O_3
        - – If O_2 is used: 1–6 wt.% O_3
    - O_3 is consumed rapidly and must be contacted uniformly in a plug-flow contactor
  - Ozone is typically used for disinfection of secondary effluents

14.40

- Basic chemistry of ozone disinfection
  - Ozone gas is added to water and decomposes following complex reactions illustrated by Eqs. 14.14–14.17

$$O_3 + H_2O \rightarrow HO_3^+ + OH^- \tag{14.14}$$

$$HO_3^+ + OH^- \rightarrow 2HO_2\bullet \tag{14.15}$$

$$O_3 + O_2 \rightarrow HO\bullet + 2O_2 \tag{14.16}$$

$$HO\bullet + HO_2\bullet \rightarrow H_2O + O_2 \tag{14.17}$$

  - Free radicals ($HO_2\bullet$ and $HO\bullet$) have unpaired electrons and are powerful, oxidizing species and are likely responsible for destruction of organisms
  - Ozone and its products can also react with other oxidizable species in water (e.g., Fe^{2+}, H_2S, organic matter)

14.41

- Ozone dose required for wastewater disinfection
  - Ozone dose is equal to:

$$\text{Dose} = C_O \times T \tag{14.18}$$

Where:
$C_O = O_3$ concentration in wastewater (mg/L)
$T =$ contact time (min)

  - Concentration of O_3 in wastewater is determined by:
    - Concentration of O_3 in air being injected into wastewater
      - Ozone generators produce a certain mass of O_3 per hour
    - O_3 transfer efficiency from the gas to dissolved phase
      - Contactors can achieve high transfer efficiency (e.g., 90 %)
  - Contact time is determined by the hydraulic design of the ozone contactor

14.42

- Membrane filtration as a disinfection technology
  - Membrane filtration was discussed in Chap. 9 which covers treatment using membrane bioreactors
  - Membrane filters do not involve a disinfection agent *per se*, but they can achieve very high pathogen reductions by filtration
    - Membranes can be manufactured with different pore sizes to reject particles of different sizes (Table 14.8)
    - As long as the membrane is intact (i.e., not torn or ruptured) and has a water flux through it, microorganisms should be removed and retained
    - The ultimate fate of the retained microorganisms depends on operating conditions and die-off and inactivation processes

14.43

- Membrane filtration typically requires that the water being treated is of very high quality
  - This is important to prevent membrane fouling and sustain a water flux through the membrane
  - Thus, before membrane filtration, a wastewater would need to be treated to at least a secondary effluent quality or higher

Table 14.8 Membrane separation based on particle size and molecular weight

Process	Particle size separated (nm)	Microorganisms potentially filtered out[a]			
		Bacteria	Virus	Protozoa	Helminthes
Microfiltration	>600 nm	Y	N	Y	Y
Ultrafiltration	>10–100	Y	?	Y	Y
Nanofiltration	>1–10	Y	Y	Y	Y
Reverse osmosis	>1	Y	Y	Y	Y

Note: Membrane bioreactors typically utilize microfiltration or ultrafiltration.
[a]Y means the respective microorganisms are filtered out, N means they are not filtered out, and ? means the fate is uncertain.

14.44

- Alternative disinfectant agents
  - There are a variety of alternative disinfectant agents that involve different chemicals or energy sources, including:
    - Peracetic acid
    - Bromine or iodine
    - Potassium permanganate
    - Calcium or ammonium hydroxide
    - Urea
    - Electrochemical energy
    - Gamma radiation
    - Photo catalytic
    - Heat

    These agents have different modes of action on microorganisms and different potential benefits and limitations for use in decentralized applications
  - These agents have experienced relatively limited use in disinfection technologies in decentralized applications
    - Peracetic acid and hydroxide chemicals have potential for use with high strength wastes and sludges and these are briefly described in the following pages

14.45

- Peracetic acid as a disinfectant agent
  - Peracetic acid ($C_2H_4O_3$) is produced by reacting acetic acid (CH_3CO_2H) and hydrogen peroxide (H_2O_2) (Eq. 14.19)

$$CH_3CO_2H + H_2O_2 \rightarrow C_2H_4O_3 + H_2O \tag{14.19}$$

  - Peracetic acid solution is an equilibrium mixture of peracetic acid, hydrogen peroxide, acetic acid, and water
    - Solutions are available commercially with different concentrations of peracetic acid and hydrogen peroxide along with stabilizers to enable storage prior to use
    - Solutions of up to 15 % peracetic acid are used as sanitizers, disinfectants and sterilizers in the food and beverage industry, for water treatment, and in other applications

14.46

- Peracetic acid disinfection mechanisms
  - Peracetic acid kills or inactivates microorganisms by disruption of the sulphydril (—SH) and sulphur (—S—S) bonds within enzymes and cell walls
  - Effectiveness of peracetic acid for virus and some protozoa is not as high as for bacteria
- Disinfection dose required
  - The dose required to achieve a high disinfection effectiveness for different organisms can be quite high (Table 14.9), e.g.:
    * 1 mg/L to 1 wt.% with a contact time of 1–12 h or more
  - Disinfection effectiveness also depends of conditions
    * Effectiveness is higher at low pH and higher temperatures
    * Effectiveness does not strongly depend on TSS or BOD levels but the dose required increases with higher organic matter concentrations

14.47

- Typical operating parameters for peracetic acid disinfection are difficult to provide due to the lack of widespread use
  - Example operating parameters and reductions of different microorganisms in wastewaters are shown in Table 14.9

Table 14.9 Example operating parameters and results of peracetic acid disinfection

Wastewater type	Approx. peracetic acid dose (C × T)	Organism and approx. reduction achieved (%)		References
Raw wastewaters	300 mg-min/L	Total coliforms	99.97	Kitis (2004)
Primary effluent	≥200 mg-min/L	Fecal coliforms	99.99	
Secondary effluent	350 mg-min/L ≥100 mg-min/L ≥250 mg-min/L 300 mg-min/L 500–3000 mg-min/L	Total coliforms Fecal coliforms Fecal streptococci *E. coli* Virus	99.99 99.99 99.99 99.9999 90–99.99999	
Fecal sludge	1.0 % for 1.2 h 0.5–1.0 % for 12 h 1.0 % for 12 h 1.0 % for 12 h	*E. coli* *Enterococcus* spp. *Salmonella* spp. *Clostridia* spp.	99.99999 >99.99 99.99999 99.99	Vinnerås et al. (2003)

14.48

- Peracetic acid disinfection pros and cons
  - Potential benefits
    - Ability to achieve disinfection in the presence of higher organic matter concentrations
    - Normally will not produce toxic disinfection by products
  - Potential limitations
    - Lack of experience and demonstrated results
    - Requires higher concentrations and longer contact times
    - Hazards and safety issues
      - It is unstable and will deteriorate with holding unless there are commercial stabilizers in the solution
      - It can be highly volatile and even explosive at concentrations above 15 %
      - It can be corrosive to many organic and metallic materials (e.g., rubber, iron, copper, brass, bronze)
    - High cost of peracetic acid solutions
    - Acetic acid in the disinfected water can support microbial regrowth

14.49

- Calcium or ammonium hydroxide as disinfectant agents
  - Microorganisms present in wastewaters typically do not survive at extremely high pH
    - Presence of NH_3 at high pH can enhance disinfection effectiveness
  - Calcium hydroxide ($Ca(OH)_2$) or ammonium hydroxide (NH_4OH) can be used to disinfect wastewater and waste solids
    - The alkaline material is added at a concentration that is sufficient to elevate the pH of the mixture to 12 or higher for a period of at least 30 min.
      - Effectiveness is greater at higher temperatures (e.g., >10 °C)
    - Concentrations of 3000–5000 mg/L of $Ca(OH)_2$ with contact times of 30 min or more have been used for disinfection of septage and similar high strength wastes

14.50

- Hydroxide disinfection pros and cons
  - Potential benefits
    - Ability to achieve disinfection in the presence of higher organic matter concentrations
    - Normally will not produce toxic disinfection by products
  - Potential limitations
    - Lack of experience and demonstrated results
    - Doses need higher concentrations and longer contact times
    - Hazards and safety
      - Hydroxides are caustic and can be corrosive
    - High cost of hydroxide chemicals
    - Addition of ammonia to the wastewater can present a high oxygen demand and nutrient content that can limit discharge or use plans

14.51

14-4. Design and Implementation

- Considerations for design and implementation (D&I) of disinfection technologies for pathogen reduction in decentralized applications
  - Disinfection efficiency required for the planned discharge or reuse option
  - Characteristics of the wastewater to be disinfected and amenability to different disinfection technologies
  - Attributes of different disinfection technologies
    - Project location and resources available (e.g., access for delivery, power, operation and maintenance (O&M) staff)
  - Specific considerations for each disinfection technology, e.g.:
    - Dose
    - Contact time
    - Contactor design
    - Management of unreacted disinfectant

14.52

- Disinfection efficiency required
  - Requirements should fundamentally be based on the level of human contact (direct or indirect) and the risk of exposure to pathogens
  - Requirements for wastewater disinfection can be influenced by criteria and standards set for other purposes, e.g.:
    - Recreational water quality criteria (USEPA 2012b)
    - Drinking water standards (USEPA 2015)
  - Requirements can be set for the concentration of microorganisms in an effluent based on the intended discharge or reuse plan in a specific application
    - Example requirements are given in Tables 14.10 and 14.11

14.53

Table 14.10 Example microbiological requirements for treated wastewater effluent discharge and reuse

Discharge or reuse option	Concentration of microorganisms in the treated effluent being discharged or reclaimed for reuse
Surface discharge of treated effluent to a stream in Colorado	*E. coli* maximum = 4000 org/100 mL (7-day geometric mean) or 2000 org/100 mL (30-day geometric mean)[a]
Unrestricted, nonpotable water reuse indoors in California	Total coliforms below 2.2 MPN/100 mL (avg. over a 7-day period) and either (1) 5-log removal of poliovirus (or a virus that is at least as resistant to disinfection as poliovirus) or (2) chlorine disinfection with a total chlorine concentration × time (CT) of at least 450 mg/L-min over ≥ 90 min of contact[b]

Source: [a]Shafer (2014).
[b]Beck et al. (2013).

14.54

Table 14.11 Example requirements for discharge or reuse of reclaimed water (EPA Victoria 2002)

Class	Use of receiving water or reclaimed water	*E. coli* (org/100 mL)	Other pathogens
Receiving waters	• Shellfish harvesting • Primary contact recreation • Other uses (wading, boating)	<14 <200 <1000	None specified None specified None specified
Reclaimed water	• Class A—high risk of direct human contact	<10	<1 enteric virus/50 L <1 viable helminth egg/L <1 protozoa/50 L
	• Class B—irrigation of dairy pasture	<100	Helminth reduction by holding or sand filtration if there is cattle grazing
	• Class C—low to insignificant risk of human contact or livestock access	<1000	
	• Class D—insignificant risk of human or livestock contact with reclaimed water	<10,000	None specified

14.55

- Characteristics of the wastewater to be disinfected and amenability to different disinfection technologies
  - Prior to disinfection, wastewater treatment is normally required to produce a secondary or higher quality effluent
  - In addition there are water quality requirements for an effluent to be disinfected such as those shown in Table 14.12

Table 14.12 Water quality requirements for effluents to be disinfected (EPA Victoria 2002)

Method	TSS (mg/L)	BOD (mg/L)	Turbidity (NTU)	Ammonia (mg/L)	pH
Chlorination	<20	<20	<2[a]	?[b]	6.0–9.0
UV irradiation	<10	<20	<2	Not applicable	Not applicable
Ozonation	<10–15	<20	<2	<1	6.0–9.0

[a]If a significant reduction in pathogens is not required (e.g., *E. coli* <10 org/100 mL), turbidity could be <10 or chlorination and <5 for ozone or UV light.
[b]Reaction of ammonium with chlorine produces chloramines which have lower disinfection power The limit on ammonia is determined by the type of chlorination process used.

14.56

- Attributes of disinfection technologies
  - Table 14.13 lists attributes of three disinfection technologies

Table 14.13 Attributes of common disinfection technologies

Desirable characteristics	Chlorination	UV irradiation	Ozonation
Disinfectant solubility	Slight	n/a	High
Homogeneity in water	Homogeneous	Homogeneous	Homogeneous
Stability of disinfectant	Stable	Generate onsite	Generate onsite
Toxicity to microbes	High	High	High
Penetration through surfaces	High	Moderate	High
Interactions w/extraneous mtl.	Organic matter	Organic matter	Organic matter
Noncorrosive and nonstaining	Highly corrosive	n/a	Highly corrosive
Deodorizing ability	High	n/a	High
Nontoxic to higher life forms	Highly toxic	Toxic	Toxic
Harmful disinfection by products?	Yes	No	No
Safety of transport, handling, use	High concern	Low concern	High concern
Availability	Low—mod. low cost	Mod. high cost	Mod. high cost

Source: After Table 12.21 (Crites and Tchobanoglous 1998).

14.57

- Chlorination disinfection technologies
  - General considerations affecting implementation
    - During design and operation, the greatest control over the disinfection technology exists with respect to:
      - Initial mixing
      - Contact time
      - Chlorine compound type and feed rate
    - Applications to decentralized systems can be challenging due to variability in wastewater flow and composition and the ability to provide for routine and required O&M
    - Disinfection agent technologies that have been used for decentralized systems include:
      - Stack-feed tablet chlorinators
      - Liquid feed chlorinators

14.58

- Typical operating parameters for chlorine disinfection of wastewater are shown in Table 14.14

Table 14.14 Typical operating parameters for chlorine disinfection of wastewater effluents (after Leverenz et al. 2006)

Parameter	Values			
Chlorine dose ($C_R \times T$)[a]	10–100 mg min/L			
Chlorine addition (C_A):	Typical demand (mg/L)	Recommended chlorine addition (mg/L) at pH shown		
		pH 6	pH 7	pH 8
Septic tank effluent	30–45	35–50	40–55	50–65
Activate sludge process effluent	10–25	15–30	30–35	30–45
Packed bed filter effluent	1–5	2–10	10–20	20–35
Contact time (T)	10–30 min			
Disinfectant residual provided?	Yes, if chlorine addition >demand			
Management of un-reacted disinfectant required?	Maybe—dechlorination may be required depending on circumstances			

[a]*Note*: chlorine dose depends on disinfection efficiency required and pH and wastewater quality.

14.59

- Stack-feed tablet chlorinators
  - General description
    - A small feeder unit with tablets of solid $Ca(OCl)_2$ that have about 70 % available chlorine
    - Effluent passes through a feeder and chlorine dissolves into it as it flows through (Fig. 14.9)
    - Effluent passes into a basin for a required contact time

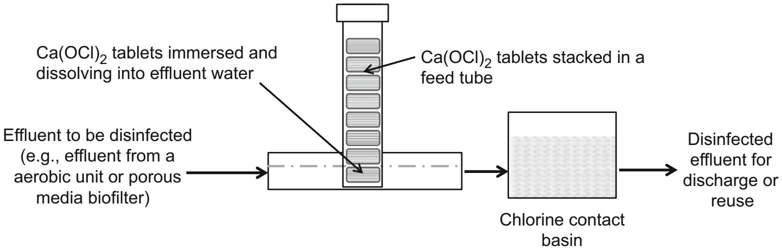
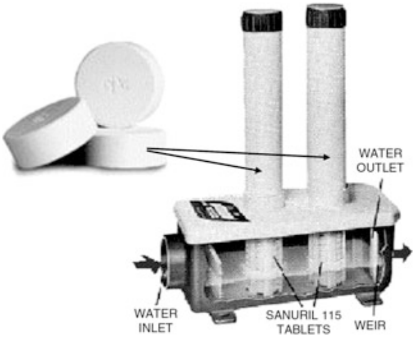


Fig. 14.9 Flow schematic of a tablet feed chlorination unit

14.60

- Stack-feed tablet chlorinators are commercially available
  - * As an example, a Sanuril unit is shown in Fig. 14.10
    - – The Sanuril Model 100 is suitable for design flows up to 10,000 gal/day and peak flows of 6.9 gal/min
    - – The tablet chlorinator holds 29 Sanuril 115 tablets that can last 60 days before reloading

Fig. 14.10 Image of a Sanuril Model 100 tablet feed chlorination unit (*Source*: http://www.tiptonenv.com/sanuril_100.html)



14.61

- Liquid feed chlorinators
  - General description
    - * Sodium hypochlorite (NaOCl) is available in solutions
      - – Household bleach = 3–6 %
      - – Pool sanitizers = 11–15 %
    - * NaOCl solution is fed into a mixing basin using a chemical feed unit as illustrated in Fig. 14.11
      - – Diaphragm pumps or aspirator or suction feeders can be used
    - * The required contact tank is provided in-line or in a tank

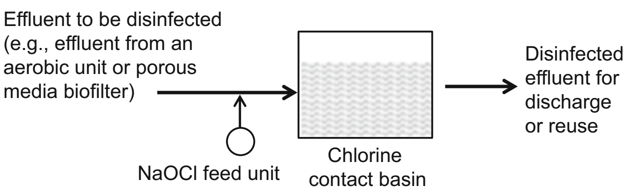


Fig. 14.11 Flow schematic of a liquid feed chlorination unit

14.62

- Calculating the feed rate for a liquid feed chlorination unit can be done using Eq. 14.20

$$Q_C = \frac{C_A(Q_D + Q_C)}{C_{CS}} \tag{14.20}$$

Where:
Q_C = rate of addition of chlorine solution (gal/min)
Q_D = design daily flow rate (gal/min) (e.g., $Q_A \times PF$)
C_A = chlorine addition needed for the dose ($C_R \times T$) plus demand and decay (mg/L)
C_{CS} = concentration of chlorine in the solution added (mg/L)

14.63

- Chlorine contact basin
  * The volume required is given by Eq. 14.21 and basins are designed with baffling to achieve plug flow and minimize short-circuiting through the tank (Fig. 14.12)

$$V_C = (HRT)Q_D \tag{14.21}$$

Where:
V_C = volume of contact basin (gal)
HRT = hydraulic retention time (min) (based on $C \times T$ required)
Q_D = design daily flow rate (gal/day) (e.g., $Q_A \times PF$)

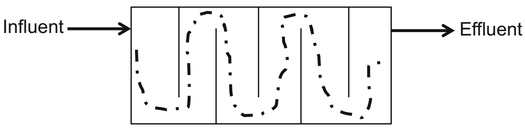


Fig. 14.12 Plan view illustration of baffled flow through a chlorine contact basin

14.64

- General performance attributes of chlorine disinfection
  - Positive attributes
    - Chlorine is a powerful disinfectant
    - Units are readily available, simple to implement and use
    - Flow-through tablet units require no power
  - Potential negative attributes
    - For tablet units:
      - Tablets need to be replaced periodically
      - Tablets can get “stuck” and not drop into the flow-through chamber reliably
    - For liquid feed units:
      - Chlorine solution needs to be refilled periodically
      - Unit requires power and calibration of an injection pump
    - For both units
      - Under- and over-dosing of chlorine can be a problem
      - Disinfection by products (e.g., trihalomethanes) can be formed and pose health concerns

14.65

- Ultraviolet light disinfection technologies
  - General considerations for implementation
    - UV systems require a very high quality effluent
      - Almost drinking water quality clarity is needed
      - Very low TSS and turbidity levels are needed
    - Flow regime for a UV contactor
      - Providing a thin depth of flow to enable UV transmittance is critical to effectiveness
    - UV does not leave a residual
      - Disinfection ceases as soon as the wastewater leaves the UV contactor
    - Commercially available UV units
      - Pre-packaged UV units often used for drinking water can be purchased and installed for wastewater disinfection
      - A few UV units are made specifically for wastewaters

14.66

- Typical operating parameters for UV disinfection are shown in Table 14.15

Table 14.15 Typical operating parameters for UV disinfection of wastewater effluents (after Fedler et al. 2012)

Parameter	Unit	Value
UV dose ($I_e \times T$)[a]	mW-S/cm^2	20–140
Contact time	s	6–40
UV intensity	mW/cm^2	3–12
Typical UV transmittance	%	50–70
Flow rate through a contactor	in./s	2–15
Disinfectant residual provided?	None	
Management of un-reacted disinfectant required?	None required	

[a]*Note*: UV dose depends on disinfection efficiency required and effluent quality.

14.67

- UV flow-through tube contactors
  - Wastewater can be exposed to UV light using different approaches and contactor configurations
  - A common approach involves a flow-through tube contactor such as illustrated in Fig. 14.13

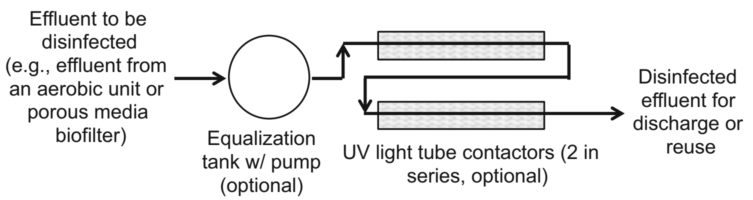


Fig. 14.13 Illustrative example flow schematic for a UV disinfection unit

14.68

- UV tube contactors are designed to achieve high transmittance during a contact time to satisfy the $C \times I$ requirement
  - An example UV tube contactor is shown in Fig. 14.14
  - Equation 14.21 can be used to size the reactor

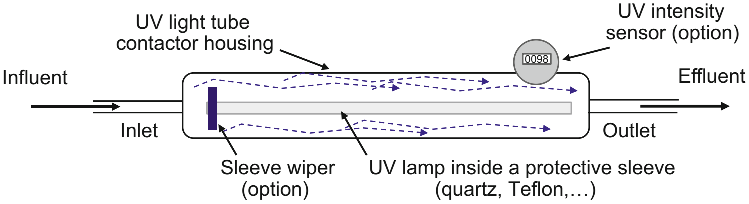


Fig. 14.14 Example of a UV flow-through tube reactor (*Note*: design specifics can vary depending on the manufacturer)

14.69

- UV flow-through tube contactor technologies are commercially available
  - Example of a commercial UV disinfection unit for textile biofilter effluent
    - The Orenco Systems® UV and AXUV are shown in Fig. 14.15
    - According to the manufacturer, the specifications for the UV disinfection unit include:
      - Disinfection of advanced secondary effluents (BOD_5 & TSS $<$ 10 mg/L)
      - Flows up to 500 gal/day
      - AXUV module is integral to a Advantex filter and gravity flow can be used
      - Minimum target dose is 20–38 mW-s/cm^2
      - 99.999 % destruction of bacteria is claimed
      - O&M involves yearly cleaning and lamp replacement every 2 years

14.70

Fig. 14.15 Example of a commercially available UV disinfection unit—the Orenco Systems® UV and AXUV (*Source*: www.orenco.com/sales/choose_a_system/UV_disinfection/index.cfm)

Typical contact chamber UV dose (65% transmittance, 80% lamp degradation)	276,000 μW-s/cm² at 1 gpm (0.06 L/sec) 55,000 μW-s/cm² at 5 gpm (0.32 L/sec) 28,000 μW-s/cm² at 10 gpm (0.63 L/sec)
Minimum target dose	30,000-38,000 μW-s/cm²
Lamp	31 in. (787 mm), 92 VAC, 50 or 60 Hz, 425 mA, 38 W; 254 nm UVC intensity at 1m is 125 μW/cm²

14.71

- Example a commercial UV disinfection unit for water
  - The Trojan UV Max is shown in Fig. 14.16
  - According to the manufacturer, the specifications for the UV disinfection unit include:
    - Low-pressure, high-intensity
    - Units treat flow rates of 7.5–25 gal/min
      - UV dose of $\geq$30,000 μW-s/cm^2
      - Q capacity based on 85 % UV transmittance (UVT) after 9000 h of lamp use
      - Replace lamps yearly (~9000 h)
    - Pretreatment required
      - Pre-filtration to $\leq$5 μm
      - Soften to $<$120 ppm hardness, remove iron to $<$0.3 ppm
      - Use activated carbon if UV transmittance is $<$85 %
    - Accessories
      - Alternative power supplies
      - Optional UV intensity monitor and solenoid shut-off valves

14.72

Fig. 14.16 Example of a commercially available UV disinfection unit—the Trojan UV Max (*Source*: www.home-water-purifiers-and-filters.com/whole-house-ultraviolet.php)

14.73

- General performance attributes of UV disinfection
  - Positive attributes
    - UV is a powerful disinfectant
    - UV units are commercially available in different sizes
    - UV units are small and simple
    - UV dosing can be readily controlled and monitored
  - Potential negative attributes
    - UV generation requires electrical power
    - UV systems can require pressurized flow through the UV lamp unit (e.g., by a pump in a dosing tank)
    - Disinfection effectiveness is dependent on UV irradiation of high quality treated wastewater effluent
      - e.g., very low TSS, hardness, turbidity, color

14.74

- Ozone disinfection technologies
  - General considerations for implementation
    - Ozone is an effective disinfectant and it also destroys organics and contributes DO
    - Ozone gas has to be generated and dissolved into wastewater
      - High ozone transfer efficiencies (e.g., 90 %) are important
    - Ozone is corrosive, presents some safety concerns, is dependent on O&M, and can be relatively expensive
    - Ozone does not leave a lasting residual
      - Ozone ceases as soon as the wastewater leaves the ozone contactor
    - Ozone units are widely available commercially for water (e.g., hot tubs and spas) but less so specifically for applications involving disinfection of wastewaters

14.75

- Typical operating parameters for ozone disinfection are shown in Table 14.16

Table 14.16 Typical operating parameters for ozone disinfection of wastewater effluents (after Fedler et al. 2012)

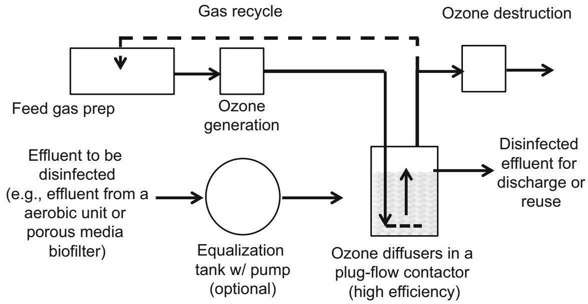
Parameter	Unit	Value
Ozone dose ($C \times T$)[a]	mg min/L	4–20
Contact time	min	4–10
Concentration	mg/L	1–5
Flow rate	in./s	2–15
Disinfectant residual provided?	–	None
Management of un-reacted disinfectant required?	Yes, collection of off gases from the ozone contactor and destruction of fugitive O_3 may be required	

[a]*Note*: ozone dose depends on disinfection efficiency required and effluent quality.

14.76

- Ozone disinfection technology configurations
  - Ozone disinfection can be accomplished using different approaches and contactor configurations
    - * Figure 14.17 illustrates a typical plug-flow contactor
    - * Another approach uses ozonators, which are flow-through tube reactors

Fig. 14.17 Illustration of an example flow schematic for an ozone disinfection unit



14.77

- Ozone generators and contactors
  - * Ozone generators must produce ozone at a rate that can achieve a desired concentration in the effluent being disinfected as given by Eq. 14.22

$$C_O = \frac{(O_P \times O_{TE})(1\,g/1000\,mg)}{(Q_D)(3.785\,L/gal)} \tag{14.22}$$

Where:
C_O = ozone concentration in the contactor (mg/L)
O_P = ozone production rate (g/day)
O_{TE} = ozone transfer efficiency into the effluent (–) e.g., 0.90
Q_D = design daily flow rate (gal/day) (e.g., $Q_A \times PF$)

  - * Ozone contactors are designed to achieve very high O_3 gas transfer efficiencies (e.g., 90 %) and uniform contact
  - * Contactor volume must satisfy $C \times T$ requirements
    - – The volume required can be determined using Eq. 14.21

14.78

- Ozone disinfection technologies are commercially available
  - Ozone units are available as individual ozone generators or complete systems (Fig. 14.18)
    - These are often made for swimming pools, hot tubs, etc.
    - A stand-alone ozone generator has to be configured with a contactor and other components

a. Generators

Model	Ozone rate (g/hr)	Dimensions, wt. amp.
22HD	2	12x12.6; 25 lb.; 1
32HD	3	12x12x6; 30 lb.; 1
42HD	4	14x16x6; 50 lb.; 1
53HD	5	14x16x6; 55 lb.; 1
62HD	6	14x16x6; 60 lb.; 1
72HD	7	20x20x8; 75 lb.; 2
82HD	8	20x20x8; 77 lb.; 2

b. Systems

Fig. 14.18 Commercial ozone generators (*left*) and complete systems (*right*) (*Source*: http://www.ozonepurewater.com)

14.79

- General performance attributes of ozone disinfection
  - Positive attributes
    - Ozone is a powerful disinfectant and also a powerful oxidant
    - Ozone disinfection does not require a high quality effluent such as that required by UV disinfection
    - Ozone units are commercially available and have had wide use in spas and therapeutic pools
  - Potential negative attributes
    - Ozone is highly corrosive so materials of construction need to be carefully selected
    - Ozone generation requires electrical power
    - Ozone disinfection using flow-through ozonators requires pressurized flow through the ozone unit (e.g., by a pump)

14.80

14-5. Summary

- Disinfection is used to destroy or remove pathogenic microorganisms and mitigate health risks associated with discharge and reuse options
  - Disinfection agent technologies destroy pathogens
    - Agents commonly used include chlorine, ultraviolet light, or ozone
    - Alternative disinfectant agents are available but not yet widely used for disinfection in decentralized systems
    - The dose required for effective disinfection varies for different disinfection agents, microorganisms and operating conditions
  - Other treatment operations can remove pathogens, with the levels ranging widely from very low (e.g., septic tank) to very high (membrane filtration)
- Disinfection technologies for pathogen destruction are typically used for secondary or higher quality effluents
  - However, disinfection of higher strength wastewaters or even sludges is possible although it is not widely done

14.81

14-6. Example Problems

- 14EP-1. Design of a chlorine disinfection unit
  - Given information
    - Source = 12 unit apartment building with 2.5 persons/apt.
    - Effluent being disinfected is recirculating sand filter (RSF) effluent with BOD_5 and TSS < 10 mg/L and a pH of 6
    - $Q_A = 69.2 + 37.2N_P$ per apartment with PF = 1.5
    - RSF HLR = 5 gal/day/ft^2; Recirculation ratio = 4:1; 48 doses/day
    - Disinfection using breakpoint chlorination with a liquid feed system to achieve 99.99 % destruction of *E. coli*
  - Determine
    - Chlorine residual and contact time required, chlorine addition concentration and flow rate required, and chlorine contact chamber size

14.82

- Solution
  - Estimate the design daily flow rate

    $Q_A = 69.2 + 37.2\ (2.5) = 162.2$ gal/day per apt.
    $Q_A = 162.2$ gal/day per apt. $\times$ 12 apt. $= 1946$ gal/day
    $Q_D = 1946 \times 1.5 = 2919$ gal/day
  - Estimate the dose required based on Eq. 14.10

$$\text{Dose} = C_R \times T \tag{14.10}$$

* Estimate the chlorine residual (C_R) for a selected contact time
  - Based on pH 6, the dominant species is HOCL
  - For >99.99 % destruction, try $C_R \times T = 100$ mg-min/L
  - Choose $T = 20$ min which is a typical contact time
  - $T = 20$ min yields $C_R = 5$ mg/L for $C_R \times T = 100$

14.83

* Check the chosen dose (100 mg-min/L) to see if it appears able to provide 99.99 % destruction

$$\frac{N}{N_o} = \left(\frac{C_R T}{b}\right)^n \tag{14.12}$$

$$0.0001 = \left(\frac{(C_R)(20\ \text{min})}{4}\right)^{-2.8}$$

$$C_R = 5.4\ \text{mg/L}$$

14.84

* Chlorine dose used to achieve 99.99 % destruction

$$\text{Dose} = C_R \times T \tag{14.10}$$

$$\text{Dose} = (5.4\,\text{mg/L})(20\ \text{min}) = 108\,\frac{\text{mg}\ \text{min}}{\text{L}}$$

○ Determine the chlorine addition required
  * Chlorine addition must provide a chlorine residual after satisfying chlorine demand and decay
  * For RSF effluent select a chlorine demand = 4 mg/L
  * Estimated decay during 20 contact time = 2 mg/L

$$C_A = C_R + (C_{DM} + C_{DC}) \tag{14.11}$$

$$C_A = 5.4 + 4 + 2$$

$$C_A = 11.4\,\text{mg/L}$$

14.85

○ Determine the flow rate of chlorine solution to be added
  * Q_D = 2919 gal/day = 2.03 gal/min
  * Q_C is assumed negligible compared to Q_D
  * Assume chlorine solution is 1 wt.% = 10,000 mg/L

$$Q_C = \frac{C_A(Q_D + Q_C)}{C_{CS}} \tag{14.20}$$

$$Q_C = \frac{11.4\,\text{mg/L}(2.03\,\text{gal/min} + \sim 0\,\text{gal/min})}{10{,}000\,\text{mg/L}}$$

$$Q_C = 0.00232\,\text{gal/min}$$

$$Q_C = 8.8\,\text{mL/min}$$

Note: Q_C = 0.11 % of Q_D, which is in fact negligible

14.86

- Determine the volume of a chlorine contact chamber
  - Volume required to achieve a T = 20 min

$$V_C = (HRT)Q_D \tag{14.21}$$

$$V_C = \left(\frac{20\text{min}}{1440\text{min/day}}\right)(2919 \ \text{gal/day})$$

$$V_C = (0.0139)(2919)$$

$$V_C = 40.6 \ \text{gal}$$

14.87

- 14EP-2. Design of a UV disinfection unit
  - Given information
    - Source = 12 unit apartment building with 2.5 persons/apt.
    - Effluent being disinfected is packaged biofilter (PBF) effluent with BOD_5 and TSS = 5 mg/L and pH = 6
    - $Q_A = 69.2 + 37.2N_P$ per apartment with PF = 1.5
    - PBF HLR = 25 gal/day/ft^2; Recirculation ratio = 4:1; 48 doses/day
    - Disinfection using ultraviolet light (new lamp with I = 10 mW/cm^2) to achieve 99.99 % destruction of *E. coli*
  - Determine
    - UV dose required and UV flow-through tube contactor size

14.88

- Solution
  - Estimate the dose required
    - For $\geq$99.99 % destruction experience suggests a dose of 40 mW-s/cm^2
  - Estimate the contact time required
    - For a new lamp, $I = 10$ mW per cm^2 in clean water
    - Assume 20 % intensity loss due to aging and only 70 % transmittance into PBF effluent being disinfected
    - Thus, $I_E = 5.6$ mW per cm^2
    - Calculate T from the dose using Eq. 14.13

$$\mathrm{Dose} = \mathrm{I_E} \times \mathrm{T} \tag{14.13}$$

$$\mathrm{T} = \left(\frac{40\ ^{\mathrm{mW\,s}}/_{\mathrm{cm}^2}}{5.6\ ^{\mathrm{mW}}/_{\mathrm{cm}^2}} \right) = 7.1\ \mathrm{s}$$

14.89

  - Estimate the daily flow rate

    $Q_A = 69.2 + 37.2\ (2.5) = 162.2$ gal/day per apt.
    $Q_A = 162.2$ gal/day per apt. $\times$ 12 apt. $= 1946$ gal/day
    $Q_D = 1946 \times 1.5 = 2919$ gal/day

  - Estimate the contactor size to achieve a $T = 7.1$ s

$$\mathrm{V_C} = (\mathrm{HRT})\mathrm{Q_D} \tag{14.21}$$

$$\mathrm{V_C} = \left(\frac{7.1\mathrm{s}}{86{,}400\mathrm{s/day}} \right) (2919\ \ \mathrm{gal/day})$$

$$\mathrm{V_C} = (0.0000822\ \ \mathrm{day})(2919\ \ \mathrm{gal/day})$$

$$\mathrm{V_C} = 0.24\ \ \mathrm{gal}$$

14.90

- 14EP-3. Design of an ozone disinfection unit
  - Given information
    - Source = 12 unit apartment building with 2.5 persons/apt.
    - Effluent being disinfected is packaged biofilter effluent (PBF) with a BOD_5 and TSS = 5 mg/L and a pH = 8
    - $Q_A = 69.2 + 37.2N_P$ per apartment with PF = 1.5
    - PBF HLR = 25 gal/day/ft^2; Recirculation ratio = 4:1; 48 doses/day
    - Disinfection using ozonation to achieve 99.99 % destruction of *E. coli*
  - Determine
    - Ozone dose required and concentration and contact time used, ozone generator capacity and contactor size needed

14.91

- Solution
  - Estimate the daily flow rate

    $Q_A = 69.2 + 37.2\ (2.5) = 162.2$ gal/day per apt.
    $Q_A = 162.2$ gal/day per apt. $\times$ 12 apt. = 1946 gal/day
    $Q_D = 1946 \times 1.5 = 2919$ gal/day

  - Estimate the ozone dose required
    * For disinfection, an ozone concentration of 1–5 mg/L in contact for 4–10 min is typically used
      – Choose $C_O = 4$ mg/L and T = 5 min

$$\text{Dose} = C_O \times T \tag{14.18}$$
$$\text{Dose} = 4\,\text{mg/L} \times 5\,\text{min} = 20\,\text{mg-min/L}$$

      – Based on experience, assume this dose will achieve the required $\geq$99.99 % destruction

14.92

- Determine the ozone generator capacity needed

$$C_O = \frac{(O_P \times O_{TE})(1\,g/1000\,mg)}{(Q_D)(3.785\,L/gal)} \tag{14.22}$$

$$O_P = \frac{(4\,mg/L)(2919\,gal/d)(3.785\,L/gal)}{(0.90)(1000\,mg/g)}$$

$$O_P = 49.1\,g-O_3/day = 2.05\,g-O_3/h$$

- Determine the ozone contactor size needed to achieve T = 5 min

$$V_C = (HRT)Q_D \tag{14.21}$$

$$V_C = \left(\frac{5min}{1440min/d}\right)(2919\,gal/d)$$

$$V_C = (0.00347\ \ d)(2919\ \ gal/d)$$

$$V_C = 10.1\ \ gal$$

14.93

Chapter 15

Management of Process Solids, Sludges, and Residuals

15-1. Scope

Solids, sludges, and residuals are generated during decentralized system operations. These materials need to be properly handled, appropriately treated and safely disposed of or beneficially used. This chapter describes the different types of solids, sludges, and residuals that are often generated in decentralized systems and highlights options for treatment and disposal or use.

15-2. Key Concepts

- A variety of solids, sludges and residuals can be generated during decentralized wastewater treatment and water reclamation. The quantity and character of these materials can be very important and may influence the selection, design and implementation of a decentralized system.
- The types of solids, sludges, and residuals that can be generated in decentralized systems include the following.
  - Process sludges and residuals are generated as a product or byproduct of the treatment processes that occur in many unit operations.
    - Septage includes settled sludge and scum that is removed from septic tanks periodically to maintain a desired hydraulic retention time.
    - Waste biological solids include waste activated sludge removed from aerobic treatment units and membrane bioreactors and biomass solids that slough off in recirculating porous media biofilters.

R.L. Siegrist, *Decentralized Water Reclamation Engineering*,
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        - Granular media is used for biofilters and nutrient reduction operations and this media can become clogged or saturated with the constituents being removed and require disposal and replacement or regeneration and reuse.
        - Vegetation grows in constructed wetlands and landscape drip dispersal units and it may be cut and removed from a site for aesthetic or nutrient removal reasons. The vegetation removed requires proper handling and disposition. In addition, vegetation can be grown in sites amended with wastewater solids and sludges and these sites can subsequently be used for grazing or as a food crop.
    - Excreta solids, fecal sludges, and diverted urine can be generated in some situations and they need to be managed.
        - Excreta solids accumulate in waterless toilets and fecal sludges accumulate in vaults to which low-volume water-flush toilets discharge.
        - Urine can be generated in urine diverting toilets where this source separation approach is used.
        - The accumulated waste, and materials with resource values under the specific circumstances, need to be removed periodically.
    - In addition, there can be other materials that may have to be dealt with including:
        - Grease that is removed from grease interceptor units installed in kitchens at restaurants, schools, nursing homes and other food service operations.
        - Untreated wastewaters that are removed from holding tanks that are pumped periodically (e.g., every few days, weeks, or months) at sources where there is only seasonal use or where there is no option for local wastewater treatment and disposal/reuse.
        - Other materials that wear out or break or have service lives and require periodic replacement (e.g., pumps, valves, controls, filter cartridges, screens).

- Effective management of these solids, sludges, and residuals is critical to help ensure protection of public health and environmental quality and to enable beneficial recovery of organic matter, nutrients, and energy content.
    - Management encompasses: (1) proper removal, handling, and transport, (2) appropriate processing, treatment and ultimate disposition, and (3) safe recovery and use of materials that have resource value.
    - Federal and state regulations often govern management and requirements can dictate what treatment and disposal or use options are feasible for a particular situation and set of circumstances.

  - In the United States, many of the materials generated during treatment of domestic/municipal wastewater are regulated through a major Federal regulation, the 40 CFR Part 503 Rule. Some states have primacy for the rule and others rely on the regional office of the U.S. Environmental Protection Agency for authority. This rule is very broad and can cover a wide variety of materials including not only septage and waste activated sludges, but also other media that has come in contact with wastewater (e.g., sand or peat removed from biofilters).

- Septage and waste biological solids.
  - Septage is probably the most common process sludge that is generated in decentralized wastewater systems due to the prevalence of septic tanks as a first unit operation in many treatment trains.
    - Septage pumped out of a septic tank is an anaerobically digested liquid-solid mixture (e.g., 3–10 % by wt. solids content) that typically contains high concentrations of organics, nutrients, and microorganisms as well as grease, hair, stringy materials, and extraneous debris.
  - Waste biological solids are generated in biological treatment unit operations.
    - Waste activated sludge is generated in all aerobic biological treatment units that rely on activated sludge biological processes. This includes aerobic treatment units and membrane bioreactors. Waste activated sludge is a more homogeneous and a less concentrated mixture compared to septage. It is a liquid-solid mixture (e.g., 1–2 % by wt. solids content) that contains high concentrations of organics, nutrients and microorganisms.
    - Waste biological solids can be generated in some recirculating porous media biofilters (PMBs) if attached growth of biomass leads to sloughing. There is limited data on the quantity and composition of these PMB biomass solids but it would seem the quantity per unit volume of wastewater treated would be much lower than that of waste active sludge from an aerobic treatment unit while the composition could be equal or more concentrated.
  - Some degree of treatment is required before disposal or use of septage and waste biological solids.
    - Two options that have been widely used for treatment and disposition of septage and waste biological solids in countries like the United States include: (1) land application and integration with agriculture or (2) discharge to a conventional wastewater treatment plant.
    - Dedicated septage and sludge treatment systems have also been deployed but to a lesser extent. Treatment trains in these systems

can include varied unit operations such as mechanical dewatering, sludge drying beds, alkaline stabilization, stabilization ponds, anaerobic or aerobic digestion, or composting.

- Treatment systems can produce what are referred to as biosolids. In the United States according to 40 CFR Part 503 Rules, biosolids are classified as Class A or Class B, depending on the concentrations of microorganisms in the solids. Class EQ are Class A biosolids of exceptional quality with respect to having low concentrations of heavy metals and a limited attraction for vectors. Biosolids are produced for beneficial use in land reclamation or for incorporation into agriculture. Federal and state site suitability requirements control where and how this is done.

■ Granular media generated as a waste product or residual.

- Granular media is sometimes generated as a waste product or residual during routine operation of a treatment unit such as a porous media biofilter, soil infiltration unit, or nutrient reduction unit.
- Some of this media is typically handled as a waste and used for construction fill or placed in a sanitary landfill (e.g., gravel from a subsurface soil infiltration unit that is being rehabilitated).
- Other media can have value as a soil amendment or fertilizer (e.g., a granular media used for sorption of phosphorus).
- Some media can be sufficiently valuable that it is worthwhile to regenerate and reuse the media (e.g., granular activated carbon).

■ Vegetation can be cut and harvested from treatment operations, land application sites and areas with reclaimed water irrigation.

- Vegetation grows in constructed wetlands and at landscape drip dispersal sites and may be harvested for aesthetic reasons or to achieve removal of nutrients from a site.
  - The quantity and characteristics depend on many factors including the type, location, and growing conditions as well as the frequency and method of harvesting.
  - A common and often appropriate option for management of the harvested vegetation consists of composting to produce a soil amendment.
  - Other management approaches include use of the vegetation to produce fibers or generate bioenergy.
- For many situations, the wastewater solids and residuals generated may end up being applied to the land in settings where vegetation grows. Reclaimed water can also be used to irrigate and promote growth of vegetation.

  - The quantity and characteristics of vegetation that is produced depend on many factors including the type, location, and growing conditions as well as the frequency and method of harvesting.
  - The appropriate method of treatment (if any) of the harvested vegetation is dictated by the intended use of the vegetation (e.g., for grazing or food crops) and requirements in applicable Federal and state regulations. At a minimum, there are normally waiting periods between the time of application and the time of harvesting and use of the vegetation. For example, in the United States, the 40 CFR Part 503 Rules specify waiting periods after biosolids application to land that range from 30 days for grazing and 38 months for crops consumed raw.
  - During land application or irrigation, vegetation can serve as a recipient for the nutrients applied. These nutrients are macronutrients (N, P, K), secondary nutrients (Ca, Mg, S, Fe, Mn), and trace minerals (B, Cu, Zn, Se). Nutrient management during land application or irrigation can be important in some situations and comprehensive nutrient management plans may be required.

- Excreta and fecal sludges.
  - Excreta and fecal sludges may be generated in some situations where alternative toilet systems are used.
  - Excreta commonly need to be managed where waterless toilets are used. This can include use of dehydration or composting toilets that are installed in dwellings or other establishments or in toilet structures provided in day use areas like parks and nature preserves. Use of waterless toilets in dwellings or other establishments is currently limited in the United States and similar developed countries but it is pervasive in many developing regions of the world.
  - Fecal sludges can be generated where ultra low-volume water-flush toilets are used and they are connected to a vault or holding tank that requires periodic emptying. There are a variety of ultra low-volume water-flush toilets including pour flush toilets, vacuum-flush toilets, and minimum flow gravity-flush toilets that can use only 0.2–0.5 gal of water per flush. The number of houses and establishments with fecal sludge generation in the United States and similar countries is relatively limited, but it can be substantial in developing regions of the world.
  - Excreta and fecal sludges require very careful management. These materials have a high resource value with respect to organic matter, nutrients, and energy content but they also pose significant health risks as a result of the high concentrations of pathogens that are typically present.

- Treatment and disposal/use options for excreta and fecal sludges are generally similar to those used for septage. In addition, for small-scale applications in remote areas or developing regions, treatment during long-term storage can occur, mainly associated with die-off and inactivation of pathogens during storage. When properly done, this can be effective for mitigating health risks associated with disposal/use options such as incorporation into soil as a soil amendment.

■ Diverted urine.

- The composition of urine generated through use of urine diverting toilets contains substantial concentrations of valuable nutrients (N, P, K, S). But diverted urine can also contain pathogens and trace organic compounds.
- Urine is normally sterile in the bladder but if an individual has an infection, pathogens can be excreted in urine. The occurrence of pathogens in urine is also highly dependent on the level of fecal contamination of the urine. Fecal contamination of urine is relatively common in urine diverting toilets and can result in fecal pathogens being present in diverted urine including bacteria, viruses and protozoa.
- Urine can contain trace organic compounds including endocrine disrupting substances. These compounds can be biogenic in origin or result from the use of pharmaceuticals and personal care products. The substances present and their concentrations depend on the users generating the urine and their personal behaviors. Examples of the substances that have been reported include: caffeine, carbamazepine, ibuprofen, naproxen, estrone, estriol, 4-acetamidophenol, Triclosan, and diclofenac.
- Diverted urine is normally collected in a container and stored for some period of time. During storage, bacteria degrade the urea and other organics in urine and NH_3 is released and the pH increases to pH9 or higher. Due to the high pH that develops urine can undergo changes in composition. Significant N may be lost by volatilization in open containers, some P will precipitate out as calcium phosphate solids, pathogens can be killed or inactivated, and some trace organics can be degraded but pharmaceutical residues can persist. Storage for 6 months or more at 20 °C (with no further addition of fresh urine) should produce pathogen free urine suitable for fertilizing crops.
- Rather than long-term storage, several physicochemical processes have been tested; however, as of this writing full-scale applications of these processes have been very limited.

15-3. Conceptual and Technical Details

Conceptual and technical details concerning the scope and key concepts covered in Chap. 15 are presented in the Slides section.

15-4. Terminology

Terminology introduced and used in Chap. 15 is defined below.

Biosolids—Biosolids refers to treated sewage sludge that is made suitable for beneficial use through incorporation into soil and agriculture. In the United States the U.S. Environmental Protection Agency sets pollutant and pathogen requirements for biosolids relative to use for land application and surface disposal in Federal regulation 40 CFR Part 503, which sets standards for the use or disposal of sewage sludge.

Composting—The stabilization of organic material through the biological process of aerobic, thermophilic decomposition.

Dehydration—Desiccation process that results from the removal of water by evaporation either naturally or with the aid of heat. Term used to describe a type of waterless toilet that processes feces through dehydration.

Dewatering—Refers to the process of removing water from a solid-liquid mixture (e.g., septage, waste activated sludge, fecal sludge) to increase the total solids content and enable handling and stabilization of the material.

Excreta—In the context of human waste, excreta refers to human urine and feces.

Fecal sludge—For the purposes of this book, fecal sludge is defined as the mixture of human wastes combined with a small volume of water that accumulates in a vault, lined pit, or similar containment structure due to the use of ultra low-volume water-flush toilets. Other definitions of fecal sludge can be broader and encompass combinations of excreta and blackwater, with or without graywater (e.g., pit latrines, septic tanks, aqua privies, and dry toilets).

Fecal sludge management (FSM)—Fecal sludge management encompasses the removal of fecal sludge from a waterless toilet or ultra low-volume water-flush vault toilet (definition varies, see Fecal sludge) followed by the proper management for its treatment and disposal or beneficial recovery.

Pathogen—An agent such as a living microorganism or particle that can cause disease. Pathogens that can cause disease in humans include a variety of bacteria, virus, protozoa, fungi, and helminthes.

Residual—Refers to waste products and materials that result from the routine operation and maintenance of a treatment unit operation.

Septage—The sludge and scum that is separated and retained within a septic tank and requires periodic removal and proper management.

Septage management—Septage management encompasses the removal of septage from a septic tank (typically by pumping) followed by the proper management for its treatment and disposal or beneficial recovery. Options for septage management typically include: (1) land treatment integrated with agriculture, (2) discharge to a local wastewater treatment plant or (3) discharge to a specially designed treatment facility. In the United States, septage is managed as a regulated waste under Federal regulations (40 CFR Part 503).

Sludge—Sludge can have different meanings depending on the context. In general it refers to a liquid-solid mixture (mostly water with 1–10 % by wt. solids). As applied to wastewater it refers to the solids and associated water that are separated during the treatment of wastewater. This definition can include domestic septage and waste activated sludge.

Stabilization—Refers to the set of physicochemical and biological processes that decompose organic matter and reduce odors and destroy pathogenic microorganisms in solid and/or liquid waste material.

Trace organic compounds—Refers to a group of organic compounds that can occur in wastewater and other impaired waters that are derived from biogenic substances, pharmaceuticals, consumer product chemicals, pesticides, and flame retardants. These compounds can be present at very low levels but still be constituents of concern. Trace organic compounds are sometimes referred to as organic micropollutants.

Urine diverting toilets (UDTs)—Refers to a type of water flush toilet that has two separate discharge compartments and drainage lines: one for feces and toilet tissue and another for urine.

Vault toilet—Refers to a waterproof tank, lined pit, or similar containment structure into which human waste is deposited alone or in combination with a small volume of water.

Vector—In the context of water and sanitation, a vector is any insect, rodent or other animal capable of transmitting infectious disease-causing agents.

Waste activated sludge (WAS)—Refers to the excess biomass that is produced during aerobic biological treatment and requires periodic removal from a bioreactor to maintain a desired solids retention time. Waste activated sludge can also include organic and mineral matter that becomes associated with the biological flocs that are separated from the liquid (either by clarification or filtration). The excess solids are removed by pumping out a portion of the aeration compartment, tank or basin or by diverting a portion of the return flow from a clarifier as a sludge with a low solids content (e.g., 1 % by wt. or less).

Waste biological solids—Refers to excess biomass that is removed from aerobic biological treatment operations including waste activated sludge from aerobic treatment units and membrane bioreactors and also excess biomass that sloughs off of media in recirculating porous media biofilters.

15-5. Acronyms, Abbreviations and Symbols

Acronyms, abbreviations and symbols used in Chap. 15 are listed below.

ATU	Aerobic treatment unit
BOD_5	Biochemcial oxygen demand exerted after 5 days
CFR	Code of Federal Regulations (United States)
COD	Chemical oxygen demand
EQ	Exceptional quality
FSM	Fecal sludge management
GAC	Granular activated carbon
K	Potassium
LWA	Lightweight expanded clay aggregate
MPN	Most probable number
N	Nitrogen
P	Phosphorus
PFU	Plaque forming units
PMB	Porous media biofilter
PRFP	Process to further reduce pathogens
S	Sulfur
SRT	Solids retention time
SSWMP	Small Scale Waste Management Project
T_{90}	Time to 90 % inactivation of pathogens
TKN	Total Kjeldahl nitrogen
TN	Total nitrogen
TP	Total phosphorus
TS	Total solids
TSS	Total suspended solids
TVS	Total volatile solids
TVSS	Total volatile suspended solids
UDT	Urine diverting toilet
USEPA	United States Environmental Protection Agency
VAR	Vector attraction reduction
VIP	Ventilated improved pit latrines
WAS	Waste activated sludge
WHO	World Health Organization

15-6. Problems

15.1. In addition to the normal constituents in septage what other pollutants might be of concern in septage generated from a commercial development including a small printing company?

15.2. What are the two most common methods used to manage septage in the United States?

15.3. Name two approaches to dewatering septage or waste activated sludges?
15.4. What is the definition of biosolids?
15.5. Composting can be used to stabilize septage and other wastewater derived solids and produce a product suitable for use as a soil amendment. If the ambient temperature is 20 °C, for an aerated static pile process what temperature (°C) must be reached and for what length of time (days) to ensure the composted material is pathogen free?
15.6. Granular media can be generated as a residual from several unit operations for treatment. Give an example of a type of granular media residual that can be processed and beneficially used in agriculture.
15.7. Give an example of a method to treat vegetation that is cut and harvested from a constructed wetland that can lead to production of a beneficial product.
15.8. How long does it normally take for 90 % inactivation of *Salmonella* after feces are incorporated into soil? If you wanted to be highly confident of complete inactivation how long might it take?
15.9. What is the reason that urine that is generated in urine diverting toilets can contain much higher levels of pathogens after diversion compared to what was in the urine generated by the users?
15.10. Can trace organic compounds (sometimes called organic micropollutants) be present in urine and if so, what is their origination?

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Slides of Chapter 15

Decentralized Water Reclamation

Chapter 15: Management of Process Solids, Sludges, and Residuals

Contents

15.1

15-1. Introduction

- Solids, sludges and residuals are generated during operation and maintenance of decentralized wastewater systems
  - Commonly encountered materials are listed in Tables 15.1 and 15.2 and include:
    - Septage
    - Waste biological solids
    - Granular media
    - Vegetation
    - Excreta (including diverted urine) and fecal sludge
  - Example generation rates are given in Tables 15.3 and 15.4
- The quantity and character of these materials can be very important and may influence the selection, design and implementation of a decentralized system

15.2

Table 15.1 Characteristics of process solids, sludges and residuals that can be generated during operation and maintenance of decentralized wastewater treatment systems

Type	Relative occurrence	Description
Septage	Likely—All systems that include one or more septic tanks	Waste solids consisting of partially digested anaerobic sludge and fats, oils and greases that have to be removed from septic tanks periodically to maintain a desired hydraulic retention time and avoid solids wash out
Waste biological solids	Sometimes—All systems that include suspended and attached growth biological treatment	Excess biomass solids produced during biological treatment (e.g., aerobic treatment units, membrane bioreactors, porous media biofilters) that have to be removed periodically to maintain a desired solids retention time and mixed liquor suspended solids concentration or that slough off of attached growth media
Granular media	Potential—From systems that include porous media when O&M requires removal	Varied types of granular material that become spent or exhausted and require removal from a unit operation. One example is the top few inches of a single pass sand filter that becomes clogged and requires replacement. Another is porous media such as lightweight expanded clay aggregate that is used for P sorption in a porous media biofilter or constructed wetland and becomes exhausted and requires replacement
Vegetation	Potential—Only from systems where plant harvesting is done	Vegetation that is cut and removed from natural systems such as constructed wetlands or landscape drip dispersal units to achieve true removal of nutrients taken up by plants

15.3

Table 15.2 Characteristics of solids and other materials that can be generated during use of decentralized wastewater treatment systems

Type	Relative occurrence	Description
Excreta (including diverted urine) and fecal sludge	Potential in the United States and likely in developing regions—All systems that include waterless toilets or vault toilet systems or urine diverting toilets (UDTs)	Human waste that accumulates in waterless toilets (e.g., pit latrines, vault toilets, composting toilets) or ultra low-volume flush toilets that has is contained and removed periodically to maintain functional capacity or urine that is diverted in UDTs. These waste materials require treatment for safe disposal or use
Oil and grease	Potential—Grease interceptors are used in food service kitchens at restaurants, schools, nursing homes, etc	Grease can be accumulated in a grease interceptor and this can be periodically removed and recycled in various ways
Raw wastewaters in holding tanks	Infrequent—Can occur at sources with short-term occupancy (e.g., seasonal cabin) or more continuous use where there is no option for decentralized treatment	In some situations, holding tanks can be used (often with ultra-low water-using toilets). Liquid wastes can accumulate in the holding tank and need to be periodically pumped out and transported to a treatment facility
Miscellaneous	Likely—Most all systems have materials that are produced intermittently and require disposal or recycling	There can be worn out pumps, broken valves, spent filter cartridges, clogged or torn screens, used containers, etc. that require proper handling and disposal or recycling

15.4

Table 15.3 Estimated quantities of solids produced in three unit operations commonly used in decentralized wastewater treatment systems

Unit operation[a]	Material	Unit generation (lb of solids per 10^3 gal treated)	Conditions and assumptions
Septic tanks	Septage periodically pumped out of a septic tank	Residential: 0.27 Commercial: 0.57	Residential wastewater TSS = 200 mg/L and FOG = 15 mg/L; Commercial wastewater TSS = 400 mg/L and FOG = 60 mg/L; $F_B = 0.15$ and $S_C = 0.07$ kg/L
Aerobic treatment unit	Waste activated sludge periodically removed from a bioreactor	Residential: 0.40 at SRT = 30 days 0.10 at SRT = 180 days Commercial: 1.22 at SRT = 30 days 0.31 at SRT = 180 days	Assume extended aeration ATU; Residential influent $BOD_5 = 120$ mg/L after primary treatment and ATU removal of 100 mg/L of BOD_5; Commercial influent $BOD_5 = 330$ mg/L after primary treatment and ATU removal of 300 mg/L; ATU MLVSS = 4000 mg/L and SRT = 30 or 180 days, at 20 °C $Y_N = 0.39$ or 0.10 lb-VSS/lb-BOD rem., 1 lb VSS = 1.25 lb TSS
Porous media biofilters with recirculation	Waste biological solids periodically pumped out of a recirculation tank	Residential: ≤0.1 Commercial: ≤0.3	Experience has shown little accumulation in residential systems but accumulation can occur in commercial systems where higher strength wastewaters are treated. Net production is assumed to be equal or less than WAS from an ATU with a long SRT

The data provided in this table are only for routine operation of the unit operation shown (not the complete system the unit operation may be part of) and for the conditions and assumptions stated. These data are provided for illustrative purposes only and can vary widely depending on situation specific conditions.
[a]During routine operation, constructed wetlands, soil treatment units, and landscape drip dispersal units can generate vegetation if it is cut and harvested but it is difficult to estimate unit generation values for production.

15.5

Table 15.4 Estimates of the quantities of materials generated in source separated waste streams that can occur in decentralized wastewater treatment systems

Unit operation	Material	Unit generation (per capita values)	Conditions and assumptions
Excreta	Feces alone	80–200 lb/year 9.6–24 gal/year	Feces generation rate depends on a person's diet, health, and activity. Feces produced per day by a person is normally 100–250 g and the normal water content is 50–90 % by wt. Feces specific gravity is typically about 1.0 and that is assumed here
	Diverted urine	648–1626 lb/year 77–193 gal/year	Urine produced per day by a person is normally 0.8–2.0 L (based on a fluid intake of 2 L/day) and this range is used here. Urine specific gravity is typically 1.002–1.03 and 1.01 is used here
Fecal sludge	Feces plus urine	728–1826 lb/year 86.6–217 gal/year	Estimated based on the low and high values in the ranges shown above. Toilet tissue is not included
	Blackwater including feces, urine, and water from water-flush toilets	452–582 gal/year[a] 5562–5692 gal/year[b]	Assume 5 toilet uses per day with either minimum flow toilets[a] (at 0.2 gal/use = 365 gal/year) or contemporary water flush toilets[b] (at 3 gal/use = 5475 gal/year)
	Blackwater including feces, urine, water from water-flush toilets, plus kitchen wastewaters	1547–1677 gal/year[a] 6657–6787 gal/year[b]	Assume 5 toilet uses per day with either minimum flow toilets[a] (at 0.2 gal/use = 365 gal/year) or contemporary water flush toilets[b] (at 3 gal/use = 5475 gal/year) plus 3 gal/day per person for kitchen wastewater generation (1095 gal/year)

The data provided in this table are for illustrative purposes only and are based on the conditions and assumptions stated. Values can vary widely depending on situation specific conditions.
[a]Minimum flow toilets. [b]Contemporary water flush toilets.

15.6

- Management of solids, sludges and residuals is critical
  - Effective management can help ensure:
    - Protection of public health and environmental quality
    - Beneficial recovery of organic matter, nutrients, and energy content
  - Management encompasses:
    - Proper removal, handling, and transport
    - Appropriate processing, treatment and ultimate disposition
    - Safe recovery and use of media with resource value
  - Federal and state regulations can govern management
    - Requirements can dictate what treatment and disposal/use options are feasible for a particular situation and set of circumstances

15.7

- Focus of Chap. 15
  - Chapter 15 is focused on the characteristics and approaches for treatment and disposal or use of:
    - Septage
    - Waste biological solids
    - Granular media
    - Vegetation
    - Excreta (including diverted urine) and fecal sludge
  - For the process principles and design of specific treatment unit operations, the reader is referred to other Chapters in this book and to design manuals and reference literature (e.g., USEPA 2003, WEF 2012, Strande et al. 2014, Tchobanoglous et al. 2014)

15.8

15-2. Septage

- Description of septage
  - Septage can be defined to include the contents removed from septic tanks, portable vault toilets, holding tanks, and similar facilities
  - In this section, the focus is on septage removed from septic tanks
    - Septage generation is discussed in Chap. 6
    - Example generation rates in lb of solids per 1000 gal treated are given in Table 15.3 for a residential versus a commercial source
  - Septage can be described as partially digested anaerobic sludge and fats, oils, and greases, and other materials that have accumulated in a septic tank during routine operation over a period of time (e.g., 3 to 10 yr or more)

15.9

- Characteristics of septage
  - Septage is typically a waste liquid with 3–10 wt.% solids
    - It contains high concentrations of organics, nutrients, and microorganisms as well as grease, hair, stringy materials, and extraneous debris
  - Septage composition
    - Major constituents are listed in Table 15.5 and pathogenic microorganisms are shown in Table 15.6
    - Septage can contain trace organic compounds (Table 15.7)
    - For some commercial or institutional sources, septage can contain higher levels of organics and heavy metals
    - The composition of septage can vary widely between different sources and even from load to load from the same source

15.10

Table 15.5 Concentrations of major constituents in septage removed from septic tanks serving households (USEPA 1984, 1999)

Parameter	Units	Concentration	
		Average	Range
Total solids	mg/L	34,106	1132–130,475
Total volatile solids	mg/L	23,100	353–71,402
Total suspended solids	mg/L	12,862	310–93,378
Volatile suspended solids	mg/L	9027	95–51,500
BOD_5	mg/L	6480	440–78,600
COD	mg/L	31,900	1500–703,000
TKN	mg/L	588	66–1060
NH4-N	mg/L	97	3–116
Total P	mg/L	210	20–760
Alkalinity	mg/L	970	522–4190
Grease	mg/L	5600	208–23,368
pH	(–)	–	1.5–12.6

Source: Table 3-4 in USEPA (1984).

15.11

Table 15.6 Concentrations of microorganisms in domestic septage (USEPA 1984)

Microorganism	Units	Typical range of concentrations
Total coliforms	No./100 mL	10^7–10^9
Fecal coliforms	No./100 mL	10^6–10^8
Fecal streptococci	No./100 mL	10^6–10^7
Pseudomonas aeruginosa	No./100 mL	10^1–10^3
Salmonella sp.	No./100 mL	10^0–10^2
Parasites (Toxacara, Ascaris, Lumbricoides, Trichuris, Trichuiura, Trichuris Vulpis)	Present	–

Source: Table 3-7 in USEPA (1984).

15.12

Table 15.7 Concentrations of ten trace organic compounds observed in septic tank solids from residential dwellings and commercial and institutional establishments (after Conn et al. 2006)

Organic compound	Use	Reporting level (mg/kg)	Samples with detects (%)	Concentration median (maximum) (mg/kg)
Bisphenol A	Plasticizer	0.050	3/5 (60)	0.26 (0.94)[b]
Cholesterol	Animal sterol	0.100	5/5 (100)	63 (200)
Coprostanol	Animal fecal sterol	0.100	5/5 (100)	59 (170)
Indole	Fragrance	0.025	5/5 (100)	1.3 (4.1)
3-Methyl-1H-indole	Fragrance	0.025	5/5 (100)	0.77 (42)
4-Nonylphenol	Surfactant metabolite	0.250	5/5 (100)	410 (1800)[b]
4-Nonylphenolethoxylate	Surfactant metabolite	0.250	5/5 (100)	15 (44)
4-Methylphenol	Disinfectant	0.050	5/5 (100)	1.9 (34)
Triclosan	Antimicrobial agent	0.025	4/5 (80)	5.2 (19)

Results of analyses for one sample of septic tank solids collected in Spring 2004 from each of 1 multi-family residential unit, 1 restaurant, 1 convenience store, 1 veterinary clinic, and 1 elementary school. Analyses were made for 56 different organic compounds and many were infrequently detected (<35 % frequency) or at very low levels.

[a]Concentrations were estimated as they exceeded the maximum value of the standard curve.

15.13

- Treatment and disposal/use of septage
  - In the United States, septage management is specified in Federal (40 CFR Part 503) and State requirements
    - Management programs vary by states and municipalities
    - Tracking or manifest systems help prevent illegal dumping
    - Procedures help ensure proper treatment and disposition
  - Two options are commonly used for treatment and disposal/use of septage in the United States
    - Land application and potential integration with agriculture
    - Treatment at a municipal wastewater treatment plant
    - Tables 15.8 and 15.9 highlight these two options and Fig. 15.1 presents several illustrative photographs

15.14

Table 15.8 Two options commonly used for treatment and disposal/reuse of septage in the United States

Type	Description
Land application[a]	Septage is delivered to a parcel of land that has site conditions that are suitable for distribution (e.g., surface or shallow subsurface incorporation), retention (e.g., <6 % slope with no runoff), and treatment processes (e.g., biodegradation, inactivation). The organic matter in septage can be biotransformed and incorporated into the surface soil horizons along with nutrients. Pathogenic microorganisms can die-off and be inactivated with time. Land application is the most commonly used method of septage management in the United States. Regulations prescribe requirements for site suitability (e.g., slope, depth to groundwater, setback distances) and approved methods for land application
Discharge to a conventional wastewater treatment plant	Septage is delivered to a local municipal wastewater treatment plant that has capacity to receive this high strength waste. Different approaches can be used for feeding septage into a plant to avoid treatment process upsets. Discharge to a wastewater treatment plant is commonly used in areas where suitable land is unavailable for land application and during winter months when land application is not feasible

[a]Three approaches to land application are presented in Table 15.9.

15.15

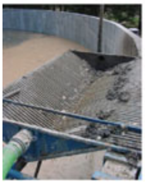
Table 15.9 Land application approaches for septage produced in the United States (USEPA 1999)

Type	Description
Surface application	Application rates depend on slope, soil type, depth of application, drainage, and hydraulic loading. Septage must not be applied during rainfall or on frozen ground. Some states require septage to be disinfected before application. Application can be done by: spray-irrigation for screened septage or by ridge and furrow irrigation for sites with slopes <0.5–1.5 %
Subsurface incorporation	Septage is placed just below the soil surface (e.g., 6–12 in.) reducing health risks and odors while fertilizing and conditioning the soil. Slopes need to be <8 % and there must be >20 in. of soil depth to a seasonal high water table
Burial	Septage burial includes disposal in holding lagoons, trenches and sanitary landfills

15.16

a. Photograph of a pumper truck being used for removing septage from a septic tank. (source: http://inspectapedia.com/septic/SepticTankPumpout036DF.jpg)

b. Photograph of a receiving station that grinds and screens septage as it is discharged from a pumper truck to a wastewater treatment plant. (source: http://www.jwce.com)



c,d,e. Photograph of (c) a receiving storage tank with screening of raw septage, (d) land spreading or (e) subsurface injection into an agricultural field (source: http://www.michigan.gov/deq/0,4561,7-135-3313_51002_3682_3717-101110--,00.html)

Fig. 15.1 Photographs of septage removal and management options

15.17

- Septage can also be treated in dedicated treatment operations to enable safe disposal or use
  - The primary goal is to stabilize the septage by removing excess liquid, decomposing organic matter, and destroying pathogens
    - Once stabilized, the treated septage may be used as a soil amendment, be land applied, or be landfilled
    - Disposition depends on the requirements of applicable regulations and the local situation
  - Examples of treatment operations used for septage in the United States and similar countries are presented in Table 15.10
  - Due the pervasive use of septic tanks in urbanized areas in developing countries, specialized approaches such as outlined in Table 15.11 have been developed and used under these circumstances

15.18

Table 15.10 Example treatment operations used for septage produced in the United States (USEPA 1999)

Operation	Description
Dewatering	Dewatering can be accomplished with plate and frame presses, belt presses, vacuum filters, gravity and vacuum-assisted drying beds, and sand drying beds. Sludge drying reed beds can also be used. Lime and ferric chloride along with polymers have been used to aid dewatering
Lime stabilization	Lime or alkaline material is added to raise the pH to 12 for a minimum of 30 min. Approx. 20–25 lb of lime are used per 1000 gal. Lime slurry can be added to the pumper truck or to a tank where septage is stored
Aerobic digestion	Septage is aerated for 15–20 days to achieve biological decomposition and reduce odors. The time required increases at lower temperatures
Anaerobic digestion	Septage is retained for 15–30 days in a closed vessel, often during co-treatment of sewage sludge. Methane gas can be recovered and used as a biofuel
Composting	Septage is mixed with a bulking agent (e.g. wood chips, sawdust) to decrease the water content, increase porosity, and help assure aerobic conditions during composting. The mixture is aerated either by the addition of air or by mechanical turning and kept at a high temperature for a minimum period of time (Table 15.12). Co-composting septage with the bulking agent provided by food wastes, paper, or yard-wastes is also possible. The compost produced can normally be used for land reclamation or agriculture

15.19

Table 15.11 Example treatment approaches for septage produced in developing regions[a]

Operation	Description
Lagoon treatment	In the Philippines septage is treated in a combination of anaerobic lagoons (3 m deep, 60 days HRT) followed by facultative ponds with the effluent polished in constructed wetlands before discharge to a surface water (Robbins 2007). This approach requires considerable land area in a location away from people
Anaerobic digestion	In Indonesia anaerobic digesters (with biogas generation) followed by sludge drying beds are used to produce biosolids suitable for land application. Effluent is treated in anaerobic bioreactors and ponds before discharge to the sea. This plant is designed to treat 15,800 gal/day of septage from a nearby city and has been in operation since 2006 (Robbins 2007)
Reed beds	Septage is periodically applied to the surface of a reed bed (~5 lb-TS/ft^2/year in cold climates and up to 50 lb-TS/ft^2/year in warm climates). The solids are retained and stabilized while the liquid is removed by gravity drainage and evapotranspiration. Reed beds can operate for months to years and total solids can reach 30–60 % by wt. (Koottatep et al. 2005, Vincent et al. 2010, Pandey and Jenssen 2015)

[a]There are other options (e.g., Table 15.9) that have also been applied in developing countries and some of the options shown here have been applied in developed countries as well. For example, sludge drying reed beds are used in Denmark.

15.20

- Treatment of septage and other wastewater solids can generate biosolids
  - "Biosolids are the nutrient-rich organic materials resulting from the treatment of sewage sludge (the name for the solid, semisolid or liquid untreated residue generated during the treatment of domestic sewage in a treatment facility)"[a]
  - Biosolids are categorized into levels based on the residual levels of pathogens and heavy metals after treatment (Tables 15.12 and 15.13)
  - Biosolids are intended to be used as a soil amendment or integrated into agricultural applications for recovery of organic matter and nutrient content (Fig. 15.2)

Fig. 15.2 Class A biosolids produced from wastewater treatment plant sludge in Chicago, Illinois (*source*: Chicago Tonight, 11 Aug 2014)

[a]*Source:* http://water.epa.gov/polwaste/wastewater/treatment/biosolids/

15.21

Table 15.12 Classification of biosolids generated from sludge treatment in the United States

Class	Description
EQ	Class A biosolids which also meet one of Part 503 VAR options 1–8 and meet the metals limits (Part 503 Table 3) are designated as "Exceptional Quality (EQ)". These products are exempted from the Part 503 General Requirements, Management Practices and Site Restrictions, and may be generally marketed and distributed
A	Class A biosolids typically are treated by a "Processes to Further Reduce Pathogens" (PFRP) such as composting, pasteurization, drying or heat treatment, advanced alkaline treatment, or by testing and meeting the pathogen density limits in Part 503 (see Table 15.13). Class A pathogen reduction reduces the level of pathogenic organisms in the biosolids to a level that does not pose a risk of infectious disease transmission through casual contact or ingestion
B	Class B biosolids typically are treated using a "Process to Significantly Reduce Pathogens" (PSRP) such as aerobic digestion, anaerobic digestion, air drying, and lime stabilization. As an alternative, producers may document compliance by analyzing the material for fecal coliform levels. When Class B requirements are met, the level of pathogenic organisms is significantly reduced, but pathogens are still present. In this case, other precautionary measures required by the Part 503 rule, i.e., site and crop harvesting restrictions, are implemented to protection of public health Class B biosolids are treated but still contain detectible levels of pathogens. There are buffer requirements, public access, and crop harvesting restrictions for virtually all forms of Class B biosolids

Source: http://water.epa.gov/scitech/wastetech/biosolids/upload/2002_06_28_mtb_biosolids_fsguide_chapter4.pdf.

15.22

Table 15.13 Pathogen density standards for biosolids classification in the United States (CFR 40 Part 503)

Classification	Pathogen or indicator	Standard density limit (dry wt. basis)
Class A biosolids	Salmonella	<3 MPN/4 g total solids or
	Fecal coliforms	<1000 MPN/g and
	Enteric viruses	<1 PFU/4 g total solids and
	Viable helminth ova	$<1/4$ g total solids
Class B biosolids	Fecal coliforms	$<2 \times 10^6$ MPN/g total solids

Biosolids Pathogen Standards can be satisfied by determining the geometric mean of seven samples of biosolids after treatment.
Source: http://water.epa.gov/scitech/wastetech/biosolids/upload/2002_06_28_mtb_biosolids_fsguide_chapter4.pdf.

15.23

- Composting as a treatment operation to produce biosolids
  - Compositing is an aerobic thermophilic biological process involving bacteria, fungi, and actinomycetes
    - Composting can be accomplished using within-vessel, static aerated pile, or open windrow operations (Figs. 15.3 and 15.4)
    - Composting at high temperature for a minimum period of time can stabilize organic solids and reduce pathogen levels (Table 15.14)

Table 15.14 Time and temperature requirements for composting of wastewater solids in the United States according to 40 CFR Part 503

Process	Temperature[a]	Time at temperature
Within-vessel, static pile	55 °C (131 °F)	3 days
Open windrow	55 °C (131 °F)	15 days with 5 turnings

[a]Vector attraction reduction (VAR) also required to meet Class A biosolids requirements. In composting operations, VAR is achieved by maintaining 45 °C (114 °F) for 10 days.

15.24

Fig. 15.3 Photographs of septage composting using (**a**) in-vessel or (**b**) windrow composting. (Photographs courtesy of Robert Rubin)

Fig. 15.4 Photograph of aerated bin composting of septage along with fats, oils and greases. (Photograph courtesy of Robert Rubin)

15.25

15-3. Waste Biological Solids

- Description of waste biological solids
  - Waste biological solids are produced in different unit operations for wastewater treatment
    - Waste activated sludge (WAS) is produced during suspended or attached growth in aerobic treatment units (Chap. 7) and membrane bioreactors (Chap. 9)
    - Waste biological solids are also produced due to attached growth and sloughing that can occur in recirculating porous media biofilters (Chap. 8)
  - Example generation rates in lb of solids per 1000 gal treated are given in Table 15.3 for a residential versus a commercial source

15.26

- Characteristics of waste biological solids
  - WAS consists of biomass and organic and inorganic solids in a wastewater liquid matrix at a concentration of 1–2 % by wt.
    - The generation rate and composition of WAS varies widely depending on the wastewater being treated and the type and operational features of the unit operation in which it is generated, including:
      - Influent wastewater composition and BOD removal efficiency desired
      - Suspended growth versus attached growth
      - Solids retention time, temperature, and mixed liquor volatile suspended solids concentration in the bioreactor
    - Tables 15.15 and 15.16 present basic composition data for WAS
    - Heavy metals and trace organic compounds (see Table 15.7) can also be present, particularly during treatment of commercial and institutional wastewaters

15.27

Table 15.15 Concentrations of major constituents in waste activated sludge removed from activated sludge systems receiving domestic wastewaters[a]

Parameter	Units	Tyagi and Lo (2013)	Crites and Tchobanoglous (1998)
pH	–	6.5–8.0	6.5–8.0
Alkalinity	mg$CaCO_3$/L	580–1100	–
Total dry solids (TS)	wt.%	0.8–1.2	0.83–1.16
Volatile solids (VS)	% of TS	59–68	59–88
Nitrogen	% of TS	2.4–5.0	2.4–5.0
Phosphorus	% of TS	0.5–0.7	2.8–11.0[b]
Greases and fats	% of TS	5–12	–

[a]This table presents basic composition data for waste activated sludge which may or may not be representative of a specific type of aerobic treatment unit or membrane bioreactor used in decentralized systems. Also, depending on the source of wastewater treated, waste activated sludge can contain levels of heavy metals and trace organic chemicals.
[b]Phosphorus is considered to be in the form of P_2O_5.

15.28

Table 15.16 Concentrations of microorganisms in waste activated sludge

Type	Organism	Density (no. per gram dry wt.)
Bacteria	Total coliforms	7×10^8
	Fecal coliforms	8×10^6
	Fecal streptococci	2×10^2
	Salmonella spp.	9×10^2
Virus	Various enteric viruses	3×10^2
Protozoa	*Giardia* spp.	10^2–10^3
Helminthes	*Ascaris* spp.	1×10^3
	Trichuris trichivra	$<10^2$
	Toxacara spp.	3×10^2

Source: Table 11.8 in Gerba and Pepper (2006).

15.29

- Waste biological solids also can be generated in some recirculating porous media biofilters
  - The microorganisms attached to the porous media can grow to the point where there is excess biomass that intermittently or continuously sloughs off the media
  - There is limited data on the quantity and composition of these biomass solids as they are often retained and removed in other solids handling operations
    - e.g., pumped out and removed along with septage as it is pumped from an upstream septic tank, the effluent from which is the influent to the PMB
  - It would seem the quantity of PMB biomass that sloughs off per unit volume of wastewater treated would be much lower than that of WAS while the composition could be equal or more concentrated

15.30

- Treatment and disposal/use of waste biological solids
  - In the United States, management of waste biological solids is specified in Federal (40 CFR Part 503) and State requirements
  - Options for management are similar to those used for septage and include:
    - Land application and potential integration into agriculture
    - Treatment at a wastewater treatment plant
    - Treatment in a specialized sludge treatment facility (e.g., a composting operation)
  - Tables 15.8, 15.9, 15.10, 15.11, and 15.14 provide a description of each of these methods

15.31

15-4. Granular Media

- Description of granular media solids and residuals
  - Varied types of granular material used in unit operations can become waste products or treatment residuals, e.g.:
    - The top few inches of a single pass sand filter that becomes clogged with a biomat and requires replacement
    - Filtralite P or Polonite® mineral media from a packed bed that is used for phosphorus sorption and the media becomes exhausted and requires replacement
    - Media from a peat biofilter that is spent and requires replacement
- Characteristics of granular media solids and residuals
  - The quantities and characteristics of granular media waste products and treatment residuals is highly varied
    - Table 15.17 and Fig. 15.5 provide some insight through a few examples

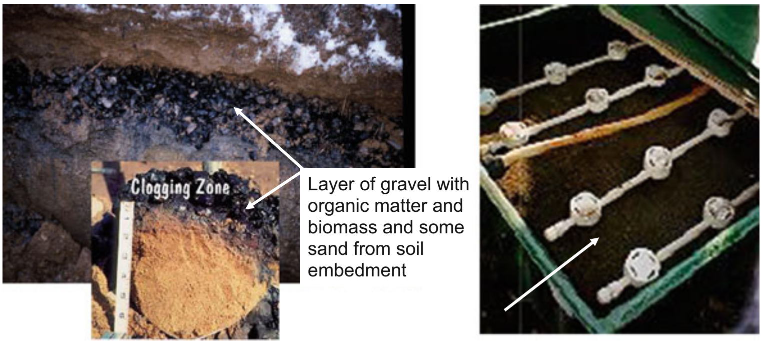
15.32

Table 15.17 Examples of granular media solids and residuals generated during decentralized wastewater treatment and water reclamation[a]

Examples	General characteristics	Example management option[b]
Media such as gravel from a soil absorption system or constructed wetland that becomes clogged and requires replacement	Stones (e.g., 1–2.5 in. diam.) that can include organic and mineral matter and potentially biomass and plant parts. Pathogenic microorganisms could be present	Dispose of as a solid waste or use as subsurface fill material
Media such as the top few inches of a single pass sand filter that becomes clogged and requires replacement	Medium sand (e.g., 0.25–1.0 mm diam.) that can include organic and mineral matter and potentially biomass. Pathogenic microorganisms are likely present	Dispose of as a solid waste or via land spreading to a non-agricultural field
Media such as Filtralite P (LWA) that is used for P sorption in a porous media biofilter or a constructed wetland or granular activated carbon (GAC) that is used in a column for trace organics removal that becomes exhausted and requires replacement	Particles (2–4 mm diam.) with phosphorus (LWA) or trace organics (GAC) sorbed to the external and internal surfaces. Pathogenic microorganisms may be present	Use the LWA with its sorbed phosphorus as a slow release fertilizer amendment in gardening and agriculture; regenerate the GAC using thermal methods so it can be reused

[a]*Note*: The information presented here is for illustrative purposes only and this compilation is not comprehensive.
[b]Federal and state rules may regulate what management options can be used.

15.33



a. Gravel removed from a soil absorption system that has hydraulically failed.

b. A layer of sand and clogging materials removed from the infiltrative surface zone of a single pass sand filter to rejuvenate its hydraulic capacity.

Fig. 15.5 Illustration of granular media solids and residuals that might be generated during maintenance functions for an old soil absorption system or a single pass sand filter

15.34

- Treatment and disposal/use of granular media solids and residuals
  - Feasible options depend on the type of media involved (Table 15.17) and applicable Federal and state regulations
    - Some media will most often be disposed of as a fill material for construction or land development or as a solid waste for landfilling, e.g.:
      - Gravel removed from a hydraulically failed soil absorption system
      - Sand removed from the clogged infiltrative surface zone of a single pass sand filter
    - Some media can be regenerated and reused, e.g.:
      - Filtralite P or Polonite® granules that were used as a sorbent for P could be processed (e.g., storage for a period of time) and then used as a source of P in agriculture
      - GAC media that was used as a sorbent for trace organic chemicals could be regenerated and reused as a sorbent

15.35

15-5. Vegetation

- Description of vegetation
  - Vegetation grows during certain types of wastewater treatment
    - Vegetation grows in constructed wetlands and other plant-based systems as well as during landscape drip dispersal
    - Vegetation may or may not need to be cut and removed from the site to enable normal operation
    - However, for true long-term removal of organic matter and nutrients from a site, vegetation generally needs to be cut and removed periodically
  - Vegetation is often present and important to processes that occur during land application of different wastewater solids and sludges
    - Vegetation that grows in a wastewater solids or sludge amended setting is often used for beneficial purposes (e.g., animal grazing, food crops)

15.36

- Characteristics of vegetation
  - The types of vegetation vary depending on the system
    - Vegetation that grows in constructed wetlands can include various types of macrophytes
    - Vegetation that grows in some plant-based systems and areas with landscape drip dispersal can include various types of grasses, shrubs, and trees (e.g., willow trees)
    - Vegetation that grows in areas that are amended with waste solids and sludges or irrigated with reclaimed water can include various types of grasses and crops
  - The quantity and characteristics of vegetation generated during routine operation varies based on situation-specific factors include growing conditions and harvesting methods
    - In general the quantity of vegetation harvested for common unit operations (e.g., constructed wetlands, soil treatment units, landscape drip dispersal units) will be limited

15.37

- Treatment and disposal/use of vegetation
  - Trees that are removed from a site can be chipped and used as a source of bioenergy in chip burning systems
  - Grasses and small woody species can be processed through composting in a fashion similar to yard waste
    - Processing and treatment can occur at the household or small business scale or in a more centralized location
    - Composting is a common method that can be used and includes:
      - Cutting and chopping to reduce the particle size of the materials
      - Mixing the vegetation with an organic matter
      - Composting and aerobic biodegradation
        - Need to provide for proper moisture content, carbon: nitrogen ratio, and oxygen levels over a sufficient period of time
      - Use of the compost product as a soil amendment

15.38

- Vegetation that grows at a site that is amended with wastewater solids or sludges or irrigated with reclaimed water
  - Methods of treatment and disposal/use depend on the situation and what, if any, beneficial outcome there is for the vegetation
    - Example beneficial outcomes include:
      - Where vegetation can be used for grazing or feeding animals
      - Where vegetation is a food crop for humans
  - Treatment of the vegetation generated may or may not be required based on the intended use and the composition of the vegetation (heavy metals, trace organics)
  - At a minimum, there are normally waiting periods before the vegetation is used
    - In the United States, the 40 CFR Part 503 Rules specify waiting periods between biosolids application ends and the crop can be harvested that range from 30 days for animal grazing to 38 months for crops consumed raw

15.39

15-6. Excreta and Fecal Sludge

- Description of excreta and fecal sludge[a]
  - Excreta including diverted urine
    - Excreta consists of the human wastes generated during use of waterless toilets (Table 15.18, Fig. 15.6)
    - Urine diverting toilets can be used to capture urine separately and enable its processing for nutrient recovery (N, P, K, S)
    - Waterless toilets and urine diverting toilets are widely used in some regions of the world but so far they have been infrequently used in houses and businesses in the United States and many other developed countries
    - Example generation rates for excreta and diverted urine are given in Table 15.4

[a]*Note:* Chapter 4 describes source separation and the use of alternative toilet systems including waterless toilets and urine diverting toilets.

15.40

Table 15.18 Features and functions of several waterless toilet options[a]

Type	Features	Treatment during accumulation[b]	Character of solids produced[b]
Commercial composting toilets	Waste is deposited into a self-contained unit or central unit in which composting occurs over long holding times. Electric fans and heaters can aid evaporation of excess liquid and drying	Yes, aerobic composting and/or evaporation of liquid	Depending on use and operating conditions, the solids produced *can be* a stable humus-like organic material that is free of *most* pathogens and safe for careful use as a soil amendment or fertilizer
Dehydration toilets	Dehydration toilets are similar to composting toilets but have the goal of evaporating liquid and drying out the waste rather than composting it. Water and urine should be diverted from the dehydration chamber	Yes, evaporation of liquid and drying of the feces	
Aerated vault latrines	Waste is deposited into a lined vault (e.g., 500–1000 gal volume) and accumulates over time. The contents of the vault are aerated intermittently or continuously	Yes, some biological decomposition and evaporation of liquid	High strength liquid waste with high concentrations of solids, organic matter, nutrients, and pathogens
Ventilated improved pit (VIP) latrines	Waste is deposited into a chamber that is ventilated. The chamber is typ. unlined so liquids are removed by seepage. In a dual compartment unit, decomposition can occur in the accumulated waste in one chamber during an extended period of no use (1–2 years)	Depends on design, but there can be some biological decomposition and seepage of liquid	

[a]*Note*: There are many types of waterless toilets but the four shown here are common and offer insights into contrasting features and functions.
[b]Depends on level of use and conditions during use relative to design.

15.41

a. Photograph of a vault toilet and it being emptied by a pumper truck. (source: http://www.rvsws.com/Vaults.htm)

b. Photograph of a self-contained composting toilet. (source: Sun-Mar http://sun-mar.com/prod_self.html)

c. Photograph of a composting toilet system with a centralized composter suitable for 65,000 uses per year. (source: clivusmultrum inc. www.clivusmultrum.com/green-building-projects.php)

Fig. 15.6 Photographs of a few waterless toilet options

15.42

- Fecal sludge
    - One definition is broad and encompasses more than just excreta:

      "Faecal sludge (FS) comes from onsite sanitation technologies, and has not been transported through a sewer. It is raw or partially digested, a slurry or semisolid, and results from the collection, storage or treatment of combinations of excreta and blackwater, with or without greywater. Examples of onsite technologies include pit latrines, unsewered public ablution blocks, septic tanks, aqua privies, and dry toilets. Faecal sludge management (FSM) includes the storage, collection, transport, treatment and safe enduse or disposal of FS. FS is highly variable in consistency, quantity, and concentration."—Strande et al. (2014)
    - In Chap. 15, the definition of fecal sludges is restricted as follows:
        - Fecal sludge consists of human wastes combined with a very small volume of water and it is generated when flush water is used and the toilet is connected to a vault or tank
        - Example toilets include pour-flush toilets and ultra low-volume gravity- or vacuum-flush toilets that can use only 0.2–0.5 gal of water per flush

15.43

- Characteristics of excreta
    - Excreta
        - The quantity and composition of excreta depends on the age, health status, diet, and activity of the individual
            - Example generation rates are given in Table 15.4
        - Tables 15.19 and 15.20 present data on the basic characteristics of excreta as well as graywaters
    - Feces alone
        - An individual can excrete about 80–200 lb/year/cap of fecal solids with a moisture content of 50–90 % by wt. (Table 15.4)
        - Feces is typically made up of microorganisms, undigested food and fiber, fats, proteins, and inorganic matter
        - Microorganisms in feces are mostly high numbers of nonpathogenic bacteria but pathogens can be present (Table 15.21)

15.44

Table 15.19 Water and nutrient loads contributed in separated sources with no dilution for urine and feces (Otterpohl et al. 2003)

Parameter	Units	Approximate annual per capita contributions			
		Total	Graywater	Urine only	Feces only
Volume	gal/year/cap kL/year/cap % of total	6750–26,660 25–100 100 %	6,600–26,420 25–100 99 %	132 0.5 <1 %	13.2 0.05 ≪1 %
Nitrogen	kg N/year/cap % of total	4–5 100 %	0.12–0.15 3 %	3.5–4.4 87 %	0.4–0.5 10 %
Phosphorus	kg P/year/cap % of total	0.75 100 %	0.08 10 %	0.38 50 %	0.30 40 %
Potassium	kg K/year/cap % of total	1.8 100 %	0.61 34 %	1.0 54 %	0.22 12 %

Note: This table also appears in Chap. 4 as Table 4.13.

15.45

Table 15.20 Mass loadings per capita for organic matter and nutrients in blackwater (urine and feces) as measured in the United States and Norway and reported elsewhere in Europe

Location	Organics (g COD/day/cap)	Nitrogen (g N/day/cap)	Phosphorus (g P/day/cap)
United States[a]	16.7[b] 6.9–23.6	8.7 4.1–16.8	1.2 0.6–1.6
Norway[c]	68–83	10–12	1.1–1.4
Sweden[c]	85	4.6	1.5
Sweden[c]	–	12	1.4
Germany[c]	40	7.5	0.9
Netherlands[c]	57–119	11.4	0.7–1.7
Typical Europe[c]	75	11.9	1.5
Turkey[c]	90	19.6	3.7
Range of above values	40–119	4.6–19.6	0.9–3.7

[a]Data compilation completed by Siegrist (1977): average of study averages over the range of individual study averages reported by Siegrist et al. (1976) and five other U.S. studies.
[b]Results are for BOD_5.
[c]Reported by Todt et al. (2015).

15.46

Table 15.21 Incidence and concentration of enteric viruses and protozoa in feces in the United States

Pathogen	Incidence (%)	Density in stool (no. per gram)
Enteroviruses	10–40	10^3–10^8
Hepatitus A	0.1	10^8
Rotavirus	10–29	10^{10}–10^{12}
Giardia	3.8	10^6
	18–54[a]	10^6
Cryptosporidium	0.6–20	10^6–10^7
	27–50[a]	10^6–10^7

Source: Table 26.3 in Pepper et al. (2006).
[a]Children in day care centers.

15.47

- Urine alone
  - An individual can excrete about 80–200 gal/year/cap of urine each year (Table 15.3)
  - Urine is an aqueous solution (>95 % water) of dissolved constituents including: urea, Cl, Na, K, and other dissolved ions plus inorganic and organic compounds (proteins, hormones, metabolites)
  - Microorganisms can be present in urine depending on the health status of the individual generating the urine
    - Urine in the bladder is normally sterile
      - However, during passage through the urinary tract urine can pick up microorganisms
    - If an individual has an infection his/her urine can contain infectious microorganisms
    - Table 15.22 presents the estimated level of pathogens in urine and the importance of urine as a transmission route for infectious disease

15.48

Table 15.22 Pathogens that may be excreted in urine (WHO 2006)

Pathogen	Urine as a transmission route	Importance
Leptospira interrogans	Usually through animal urine	Probably low
Salmonella typhi and *Salmonella paratyphi*	Probably unusual, excreted in urine in systemic infection	Low compared with other transmission routes
Schistosoma haematobium (eggs excreted)	Not directly but indirectly, larvae infect humans in fresh water	Needs to be considered in endemic areas where snail intermediate hosts are present
Mycobacteria	Unusual, usually airborne	Low
Viruses: cytomegalovirus, polyomaviruses JCV, BKV, adenovirus, hepatitis virus, others	Not normally recognized other than single cases of hepatitis A and suggested for hepatitis B; more information needed	Probably low
Sexually transmitted pathogens	No, do not survive for significant periods outside the body	Insignificant
Urinary tract infections	No, no direct environmental transmission	Low to insignificant

Source: Table 3.3 in WHO (2006, vol. 4).

15.49

- Diverted urine
  - The composition of urine generated through use of urine-diverting toilets (UDT) contains substantial concentrations of valuable nutrients (N, P, K, S) (Table 15.19)
    - * But urine from UDTs can also contain pathogens and trace organic compounds
  - ○ Pathogens in urine
    - * The composition depends on the users health status and the potential for fecal contamination of the urine generated
    - * If an individual has an infection, pathogens can be excreted in his/her urine (Table 15.22)
    - * The occurrence of pathogens in urine is also highly dependent on the level of fecal contamination of the urine
      - – Fecal contamination of urine is relatively common in UDTs and can result in fecal pathogens being present in diverted urine

15.50

- Urine can contain trace organic compounds including endocrine disrupting substances
  - * Substances can be biogenic in origin or result from the use of pharmaceuticals and personal care products
  - * The compounds present and their concentrations depend on the users generating the urine and their personal behaviors
  - * The following are examples of the substances that have been reported (Mitchell et al. 2013)
    - – Caffeine—stimulant
    - – Carbamazepine—anticonvulsant
    - – Ibuprofen, Naproxen—analgesic anti-inflammatory
    - – Estrone, Estriol—estrogens
    - – 4-acetamidophenol—analgesic
    - – Triclosan—anti-bacterial and anti-fungal
    - – Diclofenac—anti-inflammatory

15.51

- Fecal sludge
  - The composition of fecal sludge generated from ultra low-volume water-flush toilets depends on two major factors
    - * The type of excreta contributed (feces alone or feces plus urine) and its composition
    - * The volume of water used per flush
  - An estimate of fecal sludge composition can be made using the data given in Tables 15.19, 15.20, and 15.21
    - * An adjustment to the concentrations in fecal sludge can be made for the volume of dilution water added from the use of a specific type of water-flush toilet

15.52

- Treatment and disposal/use of excreta or fecal sludge
  - Treatment during accumulation and storage
    - In some waterless toilet designs human waste can undergo varying degrees of decomposition during periods of accumulation and holding
      - For example during holding in a ventilated or aerated vault there can be evaporation of water, a degree of biological degradation, and some inactivation of pathogens
    - In some waterless toilet designs heat can be added to help evaporate excess liquid
      - For example, some types of composting toilets can have an electric heating element that is used to heat the human wastes during holding
    - Table 15.23 presents data on die-off of pathogens in feces during storage and in soil and Table 15.24 presents guidance on the required storage prior to use of excreta and fecal sludges

15.53

Table 15.23 Estimated pathogen survival times during storage of feces and in soil (WHO 2006)

Microorganism	Survival at 20–30 °C (days)		Time needed for 90 % inactivation (T_{90}) (days)		Absolute max./normal max. survival in soil[a]
	Feces/sludge	Soil	Feces	Soil	
Thermotolerant coliforms	<90, usually <50	<70, usually <20	*E. coli*: 15–35	*E. coli*: 15–70	1 year/ 2 months
Salmonella	<60, usually <30	<70, usually <20	10–50	15–35	
Viruses	<100, usually < 20	<100, usually < 20	Rotarvirus: 20–100 Hepatitis A: 20–50	Rotavirus: 5–30 Hepatitis A: 10–50	1 year/ 3 months
Protozoa	<30, usually < 15	<20, usually <10	Giardia: 5–50 Crypto.: 20–120	Giardia: 5–20 Crypto.: 30–400	?[b]/2 months
Helminthes	Several months	Several months	Ascaris: 50–200	Ascaris: 15–100	7 years/ 2 years

Source: Table 3.9 in WHO (2006, vol. 4).
[a]Absolute maximum for survival is possible under unusual circumstances, such as at constantly low temperatures or well protected conditions. [b]"?" indicates unknown.

15.54

Table 15.24 Recommendations for storage treatment of dry excreta and fecal sludge before use at the household or municipal levels (WHO 2006)

Storage treatment[a]	Criteria	Comments
Storage—Ambient temperature of 2–20 °C	1.5–2 years	Will eliminate bacterial pathogens but regrowth of *E. Coli* and *Salmonella* may need to be considered if rewetted; will reduce viruses and parasitic protozoa below risk levels. Some soil-borne ova may persist in low numbers
Storage—Ambient temperature >20–35 °C	>1 year	Substantial to total inactivation of viruses, bacteria and protozoa; inactivation of schistosome eggs (<1 month); inactivation of nematode (roundworm) eggs, e.g., hookworm and whipworm; survival of a certain percentage (10–30 %) of *Ascaris* eggs (≥4 months) with more or less complete inactivation within 1 year
Alkaline treatment	pH > 9 for > 6 months	If temperature is >35 °C and moisture <25 %, lower pH and/or wetter material will prolong the time for absolute elimination

Source: Table 4.5 in WHO (2006, vol. 4).
[a]During storage treatment there is no addition of new material.

15.55

- Treatment of exreta or fecal sludge after removal
  - For most toilet system designs, excreta and fecal sludge are typically removed from a self-contained waterless toilet or an associated vault or holding tank
    - For example, a vault used with dry toilets may be emptied mechanically by pumping (or manually by excavation in developing regions) when the vault becomes full of human waste
  - Depending on the toilet design and storage conditions further treatment may be required before safe disposal or use
    - Tables 15.8, 15.9, 15.10, 15.11, and 15.14 present several treatment and disposal/use options for septage and waste activated sludge and under some circumstances these same options can be appropriate for excreta and fecal sludges
    - Table 15.25 presents several low cost options for treatment of fecal sludges that have been recommended for applications in developing regions

15.56

Table 15.25 Low cost options for treatment of fecal sludge in developing regions (WHO 2006)[a]

Type	Description
Solids-liquid separation	Settling-thickening tanks or primary ponds have been used for solids separation
Unplanted and planted drying beds	Drying beds can be used to reduce the water content below 20–30 %. The dewatered solids and liquid are removed very week or two and typically require further treatment before disposal or reuse. Planted drying beds can have improved performance. One approach uses a multimedia filter (e.g., gravel, sand, soil) that is planted with wetland plants like bullrushes or cattails. Fecal sludge is periodically discharged to the surface. Solids accumulate on the surface of the bed and require periodic removal every few years. The solids may be sufficiently stabilized with pathogens inactivated that they would not require further treatment before disposal or reuse
Stabilization ponds	Stabilization pond systems are comprised of solids-liquid separation in tanks or ponds followed by one or more anaerobic ponds and a facultative pond
Composting	Fecal sludges or blackwater (after dewatering) is mixed with an organic bulking agent (e.g. wood chips, sawdust) to support thermophilic composting (water content = 50–60 %; C:N ratio = 30–35; and mixing to enable aeration) where the temperatures can rise to 50–65 °C. Well-operated thermophilic composting can achieve near complete inactivation of pathogens after 3–4 weeks or less (see Table 15.14 for time and temperature guidance)

[a]*Note*: These options, while low cost, are generally more applicable for fecal sludge collected from multiple locations and treated at a centralized location rather than at the site of a single household or small business.

15.57

- Safe use of treated excreta and fecal sludge
  - What is considered “safe” and suitable for a specific end use depends on the situation and risks as well as applicable regulatory requirements
    - e.g., WHO recommendations for safe use of treated excreta and fecal sludge in agriculture are given in Table 15.26

Table 15.26 WHO guidelines for use of excreta and fecal sludge in agriculture (WHO 2006)

Step	Description
Treatment prior to soil	Excreta and fecal sludge should be treated before they are applied to the soil and used as a fertilizer (see Tables 15.8, 15.9, 15.10, 15.11, 15.14, or 15.25)
Soil application	Precautions related to handling of infectious material should be taken when applying excreta or fecal sludge to soil. Treated excreta and fecal sludge should be worked into the soil as soon as possible and not left on the surface
Crops	Improperly sanitized excreta or fecal sludge should not be used for vegetables, fruits or root crops that will be consumed raw, excluding fruit trees. There should always be a period of at least 1 month between the application of treated excreta or fecal sludge and the harvesting of crops. This period helps ensure die-off of pathogens

15.58

- Treatment and disposal/use of diverted urine
  - Treatment during accumulation and storage of urine
    - Diverted urine is normally collected in a container
      - During storage, urea and other organics in urine are degraded by bacteria
      - NH_3 is released and the pH increases to 9 or higher
    - Due to the high pH that develops during storage, urine can undergo changes in composition
      - Significant N may be lost by volatilization in open containers
      - Some P will precipitate out as calcium phosphate solids
      - Pathogens can be killed or inactivated
      - Some trace organics can be degraded but pharmaceutical residues can persist

15.59

  - Treatment after urine is collected
    - Treatment during simple longer-term storage
      - Recommended storage times for urine based on pathogen content and use as a fertilizer are given in Table 15.27
      - It is also recommended that urine be carefully incorporated into soil as a fertilizer to avoid volatilization and loss of NH_3
    - Treatment by different processes prior to disposal/use
      - Physicochemical processes have been tested including:
        - Combined electrodialysis, microfiltration, ozonation
        - Phosphorus recovery by struvite precipitation
        - Ammonia stripping
        - Acidification and solar evaporation
        - Nitrification/distillation
        - Electrolysis
      - However, as of this writing full-scale applications of these processes have been very limited

15.60

Table 15.27 Recommended guideline for storage times for urine mixture based on estimated pathogen content and recommended crop for larger systems[a]

Storage temperature (°C)	Storage time (month)	Possible pathogens in the urine mixture after storage	Recommended crops
4	≥1	Viruses, protozoa	Food and fodder crops that are to be processed
4	≥6	Viruses	Food crops that are to be processed, fodder crops[b]
20	≥1	Viruses	Food crops that are to be processed, fodder crops[b]
20	≥6	Probably none	All crops[c]

[a]*Note:* Urine or urine and water. When diluted, it is assumed the urine mixture is at least pH 8.8 and a nitrogen content of at least 1 g/L. Gram-positive bacteria and spore-forming bacteria are not included in the pathogen content and underlying risk assessment, but are not normally recognized as causing any infections of concern. A larger system is a system where the urine mixture is used to fertilize crops that will be consumed by individuals other than members of the household from which the urine was collected.
Source: Table 4.6 in WHO (2006, vol. 4).
[b]Not grasslands for production of fodder.
[c]For food crops that are consumed raw, it is recommended that the urine be applied at least 1 month before harvesting and that it be incorporated into the ground if the edible parts grow above the soil surface.

15.61

15-7. Summary

- Solids, sludges and residuals are generated during operation and maintenance of decentralized systems and include:
  - Septage and waste biological solids
  - Granular media from various unit operations
  - Vegetation that is cut and harvested
  - Excreta and fecal sludge and diverted urine
- Effective management is critically important to ensure:
  - Protection of public health and environmental quality
  - Recovery of organic matter, nutrients, and energy content
- Treatment can occur during accumulation and storage and by an array of unit operations and systems to enable safe disposal or beneficial use

15.62

Appendix A

Unit Conversion Table

The units used throughout the book are based on the U.S. customary units. This appendix provides a set of conversions from U.S. customary units to SI units (International System of Units, also known as the metric system) (Table A1).

Table A1 Table of conversion factors for U.S. customary units

To convert the unit in this column (Column 1)	Multiply Column 1 by this conversion factor	To obtain the unit in this column (Column 2)
acres (ac)	0.4047	hectares (ha)
acres (ac)	43,560	square feet (ft^2)
Btu	0.0002931	kilowater-hour (kWh)
Btu/hour (Btu/h)	0.2931	watts (W)
centimeters (cm)	0.3937	inches (in.)
centimeters (cm)	0.03281	feet (ft)
cubic feet (ft^3)	0.0283	cubic meters (m^3)
cubic feet/second (cfs)	448.8	gallons/minute (gal/min)
cubic meters (m^3)	35.314	cubic feet (ft^3)
cubic meters (m^3)	1.3079	cubic yards
cubic yards (yd^3)	0.7646	cubic meters (m^3)
feet (ft)	0.305	meter (m)
feet/second (fps)	0.6818	miles/hour (mph)
gallon (U.S.) (gal)	3.785	liter (L)
gallon (U.S.) (gal)	0.003785	cubic meter (m^3)
gallon (U.S.) (gal)	0.1334	cubic foot (ft^3)
gallons/minute (gal/min)	3.785	liters per minute (L/min)

(continued)

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Table A1 (continued)

To convert the unit in this column (Column 1)	Multiply Column 1 by this conversion factor	To obtain the unit in this column (Column 2)
hectares (ha)	10,000	square meters (m^2)
hectares (ha)	2.47	acres (ac)
kilograms (kg)	2.205	pounds (lb)
kilometers (km)	0.6214	miles (mi.)
kilowatt-hour (kWh)	3412	Btu
liters (L)	0.2642	gallons (U.S.)
liters (L)	1.057	quarts liquid (qt.)
meters (m)	3.2808	feet (ft)
meters (m)	0.000621	miles (mi.)
meters (m)	1.0936	yards (yd.)
miles (mi.)	1.6093	kilometers (km)
miles (mi.)	5280	feet (ft)
miles/hour (mph)	88	feet/minute (fpm)
millimeters (mm)	0.0394	inches (in.)
pound (lb)	454	gram (g)
pound (lb)	0.454	kilogram (kg)
square feet (ft^2)	0.0929	square meter (m^2)
square feet (ft^2)	0.1111	square yard (yd^2)
square feet (ft^2)	2.296E-5	acres (ac)
square feet (ft^2)	9.29E-6	hectares (ha)
square kilometers (km^2)	0.3861	square miles (mi^2)
square meter (m^2)	10.76	square feet (ft^2)
square meter (m^2)	1.196	square yards (yd^2)
square meter (m^2)	0.0001	hectares (ha)
square miles (mi^2)	2.59	square kilometers (km^2)
square yards (yd^2)	0.8361	square meters (m^2)
watts	3.4121	Btu/hour (Btu/h)
watts	0.001341	horsepower
yards (yd.)	0.9144	meters (m)
yards (yd.)	0.000568	miles (mi.)

Appendix B

Glossary

This glossary was prepared by the author to define terms that are commonly used in the context of decentralized wastewater treatment and water reclamation and reuse. In the context of other applications there could be other valid definitions for some or many of the terms listed in Appendix B.

Activated sludge—A biological process where microorganisms are grown under aerobic conditions using organic matter in the influent wastewater as a source of food and energy.

Aeration zone—A term that describes the physical system within which active aeration occurs. Examples include an aeration chamber or compartment in a larger tank, a stand-alone tank of basin devoted to aeration, and so forth.

Aerobic—Refers to a biochemical state where microorganisms require oxygen to survive and function by using oxygen as an electron acceptor.

Aerobic treatment unit (ATU)—Refers to a physical system of compartments, tanks or basins used to establish an aerobic bioreactor and the supporting components and appurtenances used to achieve aerobic biological treatment of wastewater. Aerobic treatment unit (ATU) may also be used to refer to a small-scale packaged plant used for aerobic biological treatment of wastewater.

Aggregate—Refers to stones (typically 0.5–1.5 in. diameter) that are used to establish a storage volume within a subsurface infiltration trench or bed.

Alternative sewers—Sewer systems that convey untreated or treated wastewaters utilizing gravity or pressure (positive pressure or vacuum) in smaller diameter pipelines with watertight joints that can be laid on variable grades with few access points and low infiltration and inflow. Alternative sewers include: septic tank effluent gravity (STEG), septic tank effluent pressure sewers (STEP), raw wastewater grinder pump sewers, and raw wastewater vacuum sewers.

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Anaerobic—Refers to a biochemical state where microorganisms do not require oxygen and utilize organic matter or hydrogen as electron donors and inorganic (e.g., nitrate, sulfate) or organic matter as electron acceptors. Some anaerobic organisms may react negatively or even die if oxygen is present.

Anammox—The name of a biological process that involves the simultaneous oxidation of ammonia nitrogen combined with denitrification of nitrite nitrogen.

Appliance—A water-using piece of equipment that requires power to function properly (e.g., a dishwasher or clothes washer).

Appurtenance—Devices and equipment that are not essential to the basic function of a septic tank but can improve its function or enhance its operation in one way or another.

Area-based reaction rates (k_A)—An approach to expressing reaction rates for constructed wetlands that is based on the horizontal surface area of the wetland and has units of length per time (e.g., ft/day).

Areal loading rate—An approach to determining the horizontal surface area of a constructed wetland based on empirical data concerning a loading rate specification such as mass of constituent per unit area per time that yields a particular effluent quality (e.g., the ALR in lb-BOD per ft^2 per day that yields a wetland effluent BOD of 30 mg/L).

Assimilation—Refers to the ability of subsurface soil and groundwater to accept and integrate water reclaimed from wastewater treated in a land-based treatment operation into the hydrologic cycle.

Attenuation—Refers to a set of soil and groundwater processes (e.g., biological and chemical reactions along with dilution and dispersion) that can reduce the concentrations of constituents of potential concern in water as it moves from a depth below a soil-based treatment operation (e.g., subsurface soil treatment unit or landscape drip dispersal unit) and recharges groundwater and moves away from the recharge location.

Attached growth—Refers to a aerobic biological process where the microorganisms involved in treatment are attached to, and grow on, physical surfaces such as rocks or plastic honeycombs.

Autotrophic—Refers to a group of microorganisms that use an inorganic material as an electron donor (e.g., elemental sulfur) and acceptor (e.g., nitrate nitrogen).

Background concentrations (C*)—Refers to the concentrations of constituents that are present within a wetland based on additions through means other than the influent wastewaters being treated (e.g., by plant decay, wildlife presence, atmospheric deposition).

Biofilter—See porous media biofilter.

Biogas—Methane gas that evolves during anaerobic biological processes in treatment unit operations (e.g., a septic tank).

Biomass—Biological material derived from living microorganisms involved in biological wastewater treatment.

Biosolids—Biosolids refers to treated sewage sludge that is made suitable for beneficial use through incorporation into soil and agriculture. In the United States the U.S. Environmental Protection Agency sets pollutant and pathogen requirements for biosolids relative to use for land application and surface disposal in Federal regulation 40 CFR Part 503, which sets standards for the use or disposal of sewage sludge.

Biozone—Term that refers to the region at and around the soil infiltrative surface where wastewater-induced changes occur involving biofilms, a biomat, and pore-filling agents.

Blackwater—Wastewaters from water-flush toilets and potentially including wastewaters from kitchen sink and dishwasher uses.

BOD$_5$—Oxygen demand exerted over 5 days due to biological degradation of organic matter plus potentially bio-oxidation of ammonia.

Bottom area—The horizontal soil infiltrative surface that is used for infiltration of wastewater until wastewater ponding causes infiltration to occur through vertical sidewall infiltrative surfaces.

Building—A structure (such as a house, school, restaurant, etcetera) with a roof and walls that is used for a given purpose (e.g., living, working, storage, etcetera).

Building drainage—Piping within a building that conveys wastewaters generated by usage of fixtures and appliances and typically connects to a building sewer that conveys the combined wastewater out of the building.

Building sewer—A sewer line that is connected to the building drainage piping and is used to convey wastewater to a treatment system located onsite or into a sewer system for collection and conveyance to the site of a nearby decentralize system or further away to a more remote centralized system.

Buried—(1) A term used to describe a pipeline, tank, or other component that is established below ground surface and covered with earthen materials. (2) Refers to porous media biofilter, soil treatment unit or similar unit operation that is established in the landscape with its wastewater infiltrative surface below the ground surface. Buried unit operations need to be designed to account for the fact that operation and maintenance functions can be difficult and rejuvenation may require excavation.

Carbonaceous BOD$_5$ (cBOD$_5$)—Oxygen demand exerted over 5 days due to biological degradation of organic matter.

Cell—(1) Refers to the structural, functional and biological unit of a living organism. (2) Refers to a portion of a constructed wetland that is intentionally established to help support plug-flow like conditions so flow occurs through the wetland from the inlet to the outlet with minimized short-circuiting.

Cesspool—An older form of land-based waste disposal that was used for direct release of untreated wastewater into the subsurface to keep wastewater away from direct contact with humans. Cesspools have caused soil and groundwater contamination and are no longer used in most locations.

Chamber—A compartment within a single tank that is designed to allow supernatant movement out of it to a downstream compartment while retaining sludge and scum solids within it.

Chlorination—Process of adding chlorine or hypochlorite to water to achieve disinfection of pathogenic microorganisms. Breakpoint chlorination refers to the condition where all added chlorine results in a free residual in the water or wastewater to which it is added.

Clarifier—Typically refers to a unit operation that receives the effluent that exits an activated sludge aerobic bioreactor and achieves biomass and associated solids separation during quiescent conditions and upward flow out of the clarifier over a weir into an outlet channel. This type of clarifier is often referred to as a secondary clarifier.

Clay—A naturally occurring granular material composed of finely divided mineral particles. Clay can be further defined as particles with a diameter of <0.002 mm. Clay is also a textural class of soil along with silt and clay.

Clean in place (CIP)—Refers to a method to maintain a high flux rate through a membrane without having to remove it from the membrane bioreactor.

Clogging—(1) In the context of a constructed wetland, clogging refers to the filling of porosity within a porous media like gravel by the deposition of suspended solids, accumulation of biomass and microbial byproducts, and the growth of plant roots. Clogging can reduce the saturated hydraulic conductivity of the porous media from a value for the clean media (K_S) to an effective value that accounts for the loss in porosity and permeability due to clogging (K_E). (2) In the context of a soil treatment unit, clogging refers to the blocking and filling of soil pores at and near the soil infiltrative surface that is caused by a set of physicochemical and biological processes that occur during infiltration of wastewater into soil. (3) In the context of a landscape drip dispersal unit, clogging refers to the accumulation of material that plugs up dispersal tubing or the drip emitters in it or blocks and fills soil pores in the soil the tubing is installed in. Clogging can cause hydraulic problems and necessitate operation and maintenance actions.

Cluster—Term that refers to combining the wastewater flows from more than one building (e.g., multiple houses or several businesses) using a collection system so the combined flow can be treated for a chosen discharge or water reuse option.

Cluster system—A term used to describe decentralized infrastructure that is used to serve a group of buildings or other sources. A cluster system is often comprised of an alternative sewer system connected to a decentralized treatment system for treated effluent discharge or water reuse.

Collection—A term that refers to a sewer system that is used to receive wastewater (raw or treated) and convey it (typically in buried pipelines) to a location where treatment and discharge or reuse can occur. See Conveyance.

Combined sewer overflows (CSO)—Discharge of untreated wastewater combined with stormwater to a surface water or land surface. CSOs typically can occur during storm events when hydraulic overloads occur in collection systems or treatment plants that handle wastewater plus stormwater.

Compartmentalization—Providing the required effective liquid volume for septic tank treatment by using two or more separate chambers within one tank or multiple tanks in series through which wastewater must flow from the inlet to the outlet.

Complexation—In water chemistry complexation refers to a chemical process that involves the combination of individual atom groups, ions or molecules to create one large ion or molecule. Complexation can also involve reactions that occur at surfaces that carry a charge that depends on pH and composition of the solution. The charge enhances sorption of ions with a charge opposite to the surface and repels ions with the same charge as the surface. Complexation of phosphate to aluminum or ferric oxides and hydroxides can contribute to P removal from wastewater by soil-based and chemical treatment operations.

Compliance point—See Point of compliance.

Composition—The character and concentrations of dissolved, suspended and colloidal substances in water, the nature and degree of which determine the level of impairment of the water and its quality.

Composting—The stabilization of organic material through the biological process of aerobic, thermophilic decomposition.

Configuring decentralized systems—The engineering process of selecting and combining compatible strategies and unit operations to form a system that is considered viable and sustainable for a particular project application.

Confined unit operation—Refers to treatment units that can be established in containers (e.g., a tank or basin) and can be isolated from the effects of environmental processes such as precipitation, evaporation, and temperature fluctuations. Aerobic treatment units, porous media biofilters, and membrane bioreactors are examples of confined unit operations.

Constituent of concern (COC)—Constituents of concern include dissolved and suspended inorganic and organic substances and biological organisms that can cause undesirable human health effects or degraded environmental conditions under a given water reclamation plan for discharge or reuse.

Constructed (treatment) wetland—Refers to a physical system that is designed and implemented to mimic and exploit the processes occurring in natural wetlands to accomplish treatment of an impaired water such as residential, commercial or other wastewaters.

Contact—Refers to the process of bringing a disinfectant agent and microorganism into intimate proximity of each other where the agent can act on or interact with the microorganism.

Contemporary—A term used to describe the water use and wastewater generation characteristics of a dwelling unit or other building for a particular period of time (e.g., 1990s) based on the fixtures and appliances present and the water use behaviors typical of that period of time.

Continuously stirred tank reactor (CSTR)—A type of reactor that is constantly mixed during a chemical or biochemical reaction.

Conventional—A term used to described a device, product, process, operation or system that is well established and has been commonly used for a stated purpose.

Conventional sewers—Sewers that include larger diameter pipes that are used to convey untreated wastewaters from multiple buildings under gravity flow (aided as needed by pumping stations if excavation depths get too great, to lift wastewater up for continued gravity flow) to a centralized facility for treatment and discharge or reuse.

Cross-sectional area—(1) Refers to a vertical or horizontal plane view through a physical object or a part of it. (2) Refers to a vertical or horizontal plane that is perpendicular to flow in a treatment unit operation (e.g., constructed wetland).

Decentralized water reclamation—Wastewater treatment and discharge or reuse occurs on the same or nearby property close to the location(s) where the source(s) of wastewater generation is (are) located.

Dehydration—Desiccation process that results from the removal of water by evaporation either naturally or with the aid of heat. Term used to describe a type of waterless toilet that processes feces through dehydration.

Delivery—Refers to the process of transporting wastewater effluent from a treatment unit to a downstream unit operation. For example, membrane bioreactor could be delivered to a downstream disinfection unit by gravity of pressurized flow. Delivery can also be referred to as transport. See also Distribution.

Denitrification—Denitrification involves the reduction of nitrate to N_2O and N_2 gas under anoxic conditions (DO < 0.5 mg/L) by heterotrophic bacteria (different anaerobic and facultative bacteria) that utilize organic matter as a source of energy and organic carbon. Denitrification can also be carried out under anoxic conditions by autotrophic bacteria (*Thiobacillus denitrificans* and *Thiomicrospira denitrificans*) that can use sulfur as an electron donor and NO_3^- as an electron acceptor.

Deployment viable systems—Decentralized systems that are technically viable for a particular application and are also compliant with applicable regulations and codes.

Developing region—A term that refers to a geographic location where infrastructure is being established or improved to protect public health and preserve environmental quality while increasing the overall standard of living. This term can be used for locations within a country such as the United States, Australia or Germany, which have a high human development index or in more underdeveloped regions of Asia, Africa, South

America and elsewhere where countries have a low human development index.

Development—A term that typically refers to a group or cluster of buildings such as a subdivision of houses or a commercial center comprised of several businesses.

Development scale—Refers to a geographic location where larger numbers of decentralized systems are used and there is potential for cumulative effects on groundwater or surface water quality.

Dewatering—Refers to the process of removing water from a solid-liquid mixture (e.g., septage, waste activated sludge, fecal sludge) to increase the total solids content and enable handling and stabilization of the material.

Die-off—Refers to the death of a microorganism due to natural causes.

Disinfection—Refers to the process of destroying or removing pathogenic microorganisms in a media like water so that the risk of infectious disease transmission through human contact with that media (direct or indirect) is reduced. Example processes that destroy pathogens include chlorination, ultraviolet light irradiation, and ozonation. An example of a process that can remove pathogens from water is membrane filtration. See also Natural disinfection.

Disinfection agent—Refers to a physical or chemical substance or energy source that destroys the ability of a microorganism to cause infectious disease.

Disinfection by products (DBPs)—Refers to toxic chemicals that can be formed as a result of chemical reactions that occur when a disinfection technology is applied to a water or wastewater. The nature and concentrations of DBPs depends on the disinfectant and water quality characteristics. Trihalomethanes and haloacetic acids are examples of DBPs that are commonly produced during chlorination of waters that contain dissolved organic carbon.

Dispersal event—As applied to landscape drip dispersal, refers to the delivery and distribution of wastewater into a network of drip tubing from which it exits through drip emitters and infiltrates into shallow soil below the ground surface.

Distal orifice—The orifice in lateral of a pressure distribution system that is furthest from the point at which the transport piping connects to the manifold to which the lateral and orifice are connected.

Distribution—Refers to the process of dispersing flow to all portions of a treatment unit according to the design of that unit. For example, effluent from a septic tank could be delivered to the location of a porous media biofilter and then distribution of the influent within the biofilter would normally be done to achieve uniform application of the influent over the biofilter surface area.

Distribution box—Refers to a physical unit that is supposed to distribute flow between two or more laterals or infiltration units in a soil absorption system. In practice the distribution box (also known as a D-box) does not achieve this since settling occurs and the outlets occur at different elevations. A distribution box is a very poor way to achieve uniform delivery between multiple laterals or infiltration units.

Dose—(1) A volume of influent that is delivered and distributed to a treatment unit (e.g., a biofilter or soil treatment unit) or a zone within it. (2) Refers to the concentration or intensity of a disinfectant agent in a media to be disinfected and the length of time contact occurs.

Drainage fixture unit (DFU)—A unit of measure used to size drainage piping in buildings. One drainage fixture unit is defined as equal to a discharge flow rate of 7.5 gal/min and various fixtures and appliances are allocated a certain number of DFUs based on their respective discharge flow rates.

Drip dispersal—refers to a method of intermittently distributing wastewater into shallow soil just below the ground surface. Drip dispersal of wastewater is an adaptation of drip irrigation for agronomic purposes.

Drop box—Refers to a physical unit that is supposed to deliver flow to an infiltration unit (e.g., trench) that is situated along the same topographic contour until soil clogging results in continuous ponding of that unit and the liquid level reaches a height where it overflows and is transported downslope to the next drop box and infiltration unit in the soil absorption system. This process continues until all of the infiltration units are utilized. By design, a drop box causes localized overloading to the upslope infiltration units as they are progressively used and soil clogging develops.

Dwelling unit (DU)—A single unit of residential occupancy for one person or one family such as a house, apartment unit, condominium unit, etcetera. A building can contain one dwelling unit (e.g., a house) or many (e.g., multiple apartments in a single building).

Dysfunction—See Failure.

E. coli—*Escherichia coli* is a bacterium found in the gut that is used as an indicator of fecal contamination of water.

Effective hydraulic conductivity (K_E)—In the context of a VSB or FWS constructed wetland, effective hydraulic conductivity (K_E) is a term that is associated with the ability of a constructed wetland to transmit water through it. K_E (e.g., gal/day/ft^2) is less than the K_S of the new wetland due to clogging processes that reduce the hydraulic conductivity of the wetland, particularly in the inlet zone and initial section of the treatment zone. The K_E is multiplied by the hydraulic gradient (e.g., 0.001 ft/ft) and cross-sectional area (e.g., ft^2) to determine the actual hydraulic capacity (e.g., ft^3/day or gal/day) through a zone of the wetland that has been in operation for a period of time.

Effective hydraulic retention time (HRT_E)—The average time the liquid entering a constructed wetland remains in the wetland during horizontal flow from the inlet to the outlet accounting for the presence of porous media and inactive flow zones.

Effective liquid volume—The liquid volume provided by a treatment unit operation involving a tank, basin or compartment (e.g., septic tank, chlorine contact chamber) that yields a required hydraulic retention time. The effective liquid volume is equal to the liquid surface area multiplied by the liquid depth below the outlet from the tank, basin or compartment.

Effective size (ES)—Term used for a mixture of particles that describes the diameter that 10 % by wt. of the particles present is smaller than.

Effective water volume (V_{WE})—The volume of water in a wetland that is actually involved in flow through it, accounting for the presence of porous media and inactive flow zones.

Effluent—The liquid that is discharged from a treatment unit. For example, the effluent from a biofilter is the filtrate that is discharged (not recycled) and transported to a next treatment unit or discharged to the environment. Effluent can become the influent to another treatment unit operation. For example, in the context of landscape drip dispersal (LDU) effluent is produced by an upstream treatment unit (e.g., aerobic unit) and becomes the influent to the LDU.

Effluent screen—A coarse screening device (e.g., ⅛ to ¼ in. openings) that is inserted in the flow path near the outlet from a tank, basin or compartment and is used to prevent larger solids from being discharged. Effluent screens are typically considered in the context of septic tanks and similar unit operations.

Energy grade line (EGL)—The energy grade line represents the total head available to the liquid that is flowing in a pipe. For a liquid that is flowing without any energy losses due to friction (major losses) or components (minor losses), the energy grade line would be at a constant level. In practice, however, the energy grade line decreases along the pipeline due to friction and component losses.

Enhanced biological nutrient removal (EBNR)—Refers to biological treatment systems that are specifically designed to achieve high levels of N or P removal.

Equivalent dwelling unit (EDU)—An equivalent dwelling unit is a construct used to normalize the discharges from different types of sources connected to a sewer. An EDU is based on a selected average daily flow rate. Designers or authorities can decide on the gal/day per EDU and values of 150–250 gal/day per EDU are typical.

Emitters—Generally refers to devices placed in tubing to control the rate of water flow out of the tubing into the space surrounding the tubing. Emitters can be pressure compensating or turbulent flow. Emitters are used in small diameter specialty tubing that is manufactured and used for irrigation, wastewater distribution, and wastewater drip dispersal into the shallow subsurface.

Excreta—In the context of human waste, excreta refers to human urine and feces.

Extended aeration—A term that can be used to refer in general to wastewater being aerated over a long period of time or to a suspended growth flow-through aerobic treatment system that has a long solids retention time.

Evapotranspiration (ET)—The amount of water removed by evaporation and transpiration under the current soil moisture and environmental conditions.

Factor of safety (FOS)—Factors of safety can be used to account for uncertain or unknown attributes, such as usage at a commercial establishment, while peaking factors account for known variability.

Failure—A term that is used to describe the inability of a product, process, operation, or system to achieve the performance expected based on the specifications or engineering design and implementation. For example, a porous media biofilter that was designed to process 5 gal/day/ft^2 but can no longer process 1 gal/day/ft^2 could be considered to have suffered a hydraulic failure. Failure normally implies that rejuvenation would be required to restore the performance to that expected during the design and implementation. Dysfunction is a measure of the degree of failure, but which does not normally require rejuvenation. For example, a porous media biofilter that was designed to process 5 gal/day/ft^2 but can only process 4 gal/day/ft^2 could be considered dysfunctional but not yet a failure.

Fecal sludge—For the purposes of this book, fecal sludge is defined as the mixture of human wastes combined with a small volume of water that accumulates in a vault, lined pit, or similar containment structure due to the use of ultra low-volume water-flush toilets. Other definitions of fecal sludge can be broader and encompass combinations of excreta and blackwater, with or without graywater (e.g., pit latrines, septic tanks, aqua privies, and dry toilets).

Fecal sludge management (FSM)—Fecal sludge management encompasses the removal of fecal sludge from a waterless toilet or ultra low-volume water-flush vault toilet (definition varies, see Fecal sludge) followed by the proper management for its treatment and disposal or beneficial recovery.

Field flushing—Refers to a process done where flow through drip dispersal tubing is high enough to cause scouring of solids and this flushing flow is captured and returned to a treatment tank or basin.

Filtrate—The liquid that exits the bottom of a porous media biofilter. The filtrate can be discharged as biofilter effluent or it may be recycled back to a recirculation/dosing tank for blending with the incoming wastewater (e.g., septic tank effluent) before dosing to the biofilter.

Filtration—In the context of treatment of wastewater or other impaired waters, filtration refers to a physicochemical process that removes colloidal and particulate solids from the water during its movement through a membrane or porous media that have certain pore size and chemical properties that prevent the solids from passing through.

First-order reaction rate—The rate of a reaction that is dependent on the concentration of one reactant (e.g., BOD_5). Zero-order reaction rates are only dependent on time and second-order reaction rates are dependent on the concentration of two reactants (e.g., O_2 and BOD_5).

Fixture—A water-using piece of equipment that does not require power to function properly (e.g., a sink faucet or toilet).

Flotation—The physical process by which solids that are less dense than a liquid are separated from the liquid (e.g., wastewater) by rising to the liquid surface due to buoyant forces.

Floc—Refers to the clustering of microorganisms and biomass solids that develops during flocculant settling in a secondary clarifier of an aerobic treatment unit operation.

Flow—(1) Water or wastewater liquid movement in a pipeline, basin or unit operation. (2) Water used or wastewater generated as a result of use of an appliance or fixture in a building.

Flow controls—Refers to physical barriers (typically vertically oriented but they could be horizontal) that are used to help enable plug-flow like conditions and avoid short-circuiting during flow through a constructed wetland from the inlet to the outlet end.

Flow rate (Q)—(1) A measure of the volume of liquid that flows through a pipe during a certain period of time (e.g., gal/min, ft^3/s). (2) A measure of the volume of water used or wastewater generated during use of an appliance or fixture in a building (e.g., gal per laundry load, gal per toilet flush).

Flux—(1) The rate of liquid or mass flow through a membrane given in units of volume per time per surface area (e.g., gal/day/ft^2, lb/day/ft^2). (2) The rate of liquid or mass flow through a horizontal or vertical plane in the subsurface given in units of volume per time per surface area (e.g., gal/day/ft^2, lb/day/ft^2).

Foam media—A type of media used in package biofilters that is comprised of open cell foam typically configured in small cubes that are packaged in cylindrically containers.

Food to microorganism ratio (F:M)—A design parameter that is defined as the substrate entering an aeration zone compared to the concentration of microorganisms in the aeration zone.

Footprint area—Refers to the landscape area encompassed within the perimeter surrounding the area occupied by the entire treatment unit (e.g., membrane bioreactor, constructed wetland, soil treatment unit or landscape drip dispersal unit).

Forward flow—The flow that is applied to the surface of a biofilter. For a single pass biofilter this is equal to the incoming daily flow. For a multiple pass biofilter (with recirculation) this flow is the equal to the incoming flow plus the recycled flow.

Fouling—A term that refers to the process of pore plugging or surface coating of a membrane or other filter material that reduces the rate of liquid flow through the membrane.

Free access—A term used to describe a biofilter that has an infiltrative surface that is easily accessible which facilitates operation and maintenance functions and rejuvenation if needed.

Free radical—Refers to an atom or molecule that has a single unpaired electron in an outer shell which causes it to be highly reactive as an oxidant. Hydroxyl radicals (HO•) are produced when ozone is added to water.

Free water surface wetland—A type of constructed wetland for treatment of wastewater or other impaired waters that has the distinguishing feature of having a major portion of the horizontal surface of the wetland that is comprised of an open water surface.

Gravity distribution—Refers to a method of distributing wastewater into different parts or zones of a soil treatment unit (e.g., different trenches or different portions of a bed).

Graywater—Wastewaters produced by water use in basins, sinks and appliances in residential and nonresidential buildings. Mixed graywater includes food preparation related wastewaters (e.g., kitchen sink and dishwasher) while light graywater excludes food preparation wastewaters and possibly laundry wastewaters. All types of graywater exclude toilet wastewaters, which contain human excreta. Graywater can also be spelled as greywater.

Grinder pump sewer—A type of wastewater collection and conveyance system that uses a grinder pump in a small vault near each building to grind up untreated wastewater and discharge it under pressure into a pressurized pipeline for conveyance to another location.

Heterotrophic—Heterotrophic bacteria (different anaerobic and facultative bacteria) utilize organic matter as a source of energy and organic carbon.

High rate process—A term that refers to an aerobic biological treatment process with respect to how fast it achieves biodegradation of organic matter in wastewater. A high rate process is one that only requires a relatively short period of aeration of the wastewater to achieve a secondary quality effluent.

Horizon—A term used to describe a layer in a soil profile that has developed through a set of soil-forming processes and is distinguishable from a layer above and below it.

Horizontal flow—(1) Refers to a flow path that is near level from one location to another. (2) Refers to constructed wetlands in which the wastewater being treated flows in a horizontal direction from an inlet to an outlet.

Human Development Index (HDI)—A statistical tool developed by the United Nations used to measure a country's overall achievement in its social and economic dimensions. The social and economic dimensions of a country are based on the health of people, their level of education attainment and their standard of living.

Hydraulic capacity (Q_C)—The volume of water or wastewater that can be processed through a unit operation such as a porous media biofilter from the inlet to the outlet while achieving a performance the unit operation was designed for.

Hydraulic gradient—A unit-less measure of the force causing water flow to occur through a channel or bed of porous media defined as the change in elevation head over a unit length of the flow path.

Hydraulic grade line (HGL)—The hydraulic grade line represents the total head available to a liquid flowing in a pipe minus the velocity head. In STEG and STEP systems, the velocity head is usually negligible so the HGL is approximately equal to the EGL.

Hydraulic inefficiencies—(1) Refers to the departure from true plug flow that occurs in treatment unit operations that include tanks, basins, compartments (empty or filled with porous media). (2) Refers to the departure from true plug flow that occurs in constructed wetlands by virtue of the inlet and outlet hydraulics, wetland geometry, and heterogeneities within the wetland. Hydraulic inefficiencies are accounted for during wetland surface area sizing using the P-k_A-C* modeling of constituent removal in a wetland.

Hydraulic loading rate for design (HLR_D)—The areal loading rate applied to the surface area of a treatment unit such as a porous media biofilter or soil treatment unit that is used for design of the surface area required for a design daily flow rate.

Hydraulic retention time (HRT)—(1) In the context of confined treatment operations the hydraulic retention time is a design parameter that describes how long liquid remains in a specified chamber, basin, or tank. HRT is defined as the volume of the chamber, basin, or tank (e.g., gal) divided by the flow rate passing it through it (e.g., gal/day). (2) In the context of land-based treatment operations, hydraulic retention time refers to the length of time that water remains within a volume. Within a soil treatment unit this means between the location of the soil infiltrative surface and some depth of soil below it or distance in groundwater away from the point of recharge.

Imhoff Tank—A tank that combines solids separation and anaerobic digestion to achieve advanced primary treatment of wastewater. An Imhoff Tank has a settling chamber that is physically separated from the chamber in which anaerobic digestion occurs.

Impaired water—Refers to water that has been used or impacted in a manner as to have quality characteristics that make it unsuited for one or more uses. Examples of impaired waters include: residential and commercial wastewater, municipal wastewater, graywater, stormwater, acid mine drainage, etc.

Inactivation—Refers to the loss of infectivity of a pathogenic microorganism by one or more mechanisms.

Inactive flow zones—Generally used in the context of constructed wetlands to refer to volumetric portions within it that are not involved in advective movement of water from the inlet to the outlet.

Indicator microorganisms—Refers to a group of microorganisms that are used to indicate the possible presence of human pathogens. Indicator microorganisms are not human pathogens but they are shed by humans in large numbers and are relatively easy to analyze for (e.g., fecal coliform bacteria). Presence of an indicator organism in a water suggests that the water may have been impacted by human wastes and could contain human pathogens.

Indoor water use—Water use that occurs through use of fixtures and appliances within a building. Indoor water use generates wastewaters. Indoor water use is also referred to as interior water use.

Infiltrability—The infiltration rate when water is made freely available at a soil infiltrative surface (at the ground surface or within the subsurface).

Infiltration and inflow (I&I)—Infiltration is due to groundwater seepage into conveyance piping and tankage through holes, cracks, joint failures, and faulty connections. Inflow is due to stormwater flow directly into conveyance piping or tankage via roof drain downspouts, foundation drains, storm drain cross-connections, and through holes in covers.

Infiltration rate (IR)—The rate at which water passes through the infiltrative surface area of a bed of porous media.

Infiltrative surface (IS)—The horizontal surface area that comprises the top of a biofilter to which influent is distributed during a dose or the location in a soil profile to which wastewater is distributed and becomes the influent to a soil treatment unit.

Infiltrative surface architecture (ISA)—Refers to the physical characteristics at and around the soil infiltrative surface encompassing the geometry of the infiltration unit (e.g., narrow trench or bed) and the characteristics of the space through which wastewater moves once it is released from the delivery piping and moves over and infiltrates into the pore network of the native soil (e.g., gravel filled vs. chamber outfitted). ISA can be difficult to grasp but it is analogous to the architecture of a building.

Infiltrative surface utilization (ISU)—Refers to the fraction of the infiltrative surface area determined during design that is actually used during startup and as operation continues.

Infiltration unit—Refers to an individual physical unit (e.g., a single trench, a narrow bed, a chamber within a larger bed) to which wastewater is applied within a soil treatment unit.

Influent—The effluent from an upstream treatment unit becomes the influent to a downstream treatment unit. For example, septic tank effluent is often used as the influent to a porous media biofilter.

Infrastructure—The basic physical and organizational structures and facilities needed for a given function such as water treatment and supply or wastewater treatment and discharge or water reuse.

Intensity—Refers to the amount of ultraviolet light energy transmitted into a media (e.g., water, wastewater, etc.) to be disinfected. Effective intensity accounts for the aging of a UV lamp and the transmittance into the media to be disinfected (which is less than 100 %).

Interior water use—See Indoor water use.

Intermittent—A term that is used to describe a method of applying an influent to a treatment unit (e.g., a porous media biofilter) where there are periods of dosing and no dosing.

Ion exchange—A process that involves the exchange of ions between a solution and a solid polymer or mineral resin.

Kinetics—A term that refers to the process concerned with measuring and studying the rates of reactions.

Kjeldahl nitrogen—See Total Kjeldahl nitrogen.

Land-based treatment system—Refers to unit operations and systems that are used to treat wastewater or other impaired waters by exploiting processes that naturally occur on the land surface and in the soil profile underlying it. Groundwater is often involved as the receiving environment where reclaimed water is ultimately assimilated. Land-based treatment systems can also be referred to as soil-based treatment systems. Examples of soil-based treatment operations covered in this book include subsurface soil treatment units and landscape drip dispersal units.

Lateral—A small diameter pipe with orifices in it or spray nozzles attached to it that is used for distribution of the influent uniformly over the infiltrative surface of a porous media treatment unit (e.g., porous media biofilter or soil treatment unit). In the context of landscape drip dispersal, a lateral is a length of drip dispersal tubing that connects from a supply manifold to a return manifold.

Layer—Refers to a thickness of soil, rock, water, or other matter that may or may not be affected by soil-forming processes.

Leachfield—A term that was used in the 20^{th} century for a soil-based wastewater system that primarily involved a subsurface means of infiltration that was used for disposal of septic tank effluent.

Life-cycle assessment (LCA)—Life cycle assessment is an analytical technique for evaluating environmental demands and impacts associated with a product, operation, or system from the cradle to grave. LCAs typically compile an inventory of relevant energy and material inputs and releases and evaluate the potential impacts associated with the inputs and releases.

Lightweight aggregate (LWA)—Lightweight aggregate is made by heating clay to a high temperature (e.g., 1200 °C) in a rotary kiln causing gases to expand the clay and form a microporous structure when cooled. LWAs have a high phosphorus sorption capacity (PSC) and can be used as a reactive porous media in constructed wetlands and phosphorus removal filters. LWA can be produced in different spherical size ranges (e.g., 0.1–4 mm, 4–10 mm diameters). Forms of LWA are manufactured in several countries and carry trade names such as LECA® or Filtra P.

Limiting condition—Refers to a characteristic of the subsurface that can interfere with proper function and performance of a soil-based treatment system. Three common types of limiting conditions include: a shallow perched zone of saturation or shallow groundwater table, a layer of soil materials that has low permeability and low hydraulic conductivity, and a layer of bedrock.

Lime—Common name for calcium oxide (CaO).

Linear loading rate (LLR)—Refers to the rate of flow of wastewater that is applied as influent to a cross-section of a soil treatment unit that is perpendicular to the landscape contour and flows downgradient in the subsurface. The landscape linear loading rate has dimensions of gal/day per ft of soil treatment unit length along a slope.

Liner—Refers to a water impermeable material (e.g., a geomembrane) that is used to contain a treatment unit operation like a recirculating sand filter or constructed wetland.

Long-term acceptance rate (LTAR)—The pseudo steady-state rate at which wastewater is transmitted through the infiltrative surface of a bed of porous media (e.g., within a porous media biofilter or a soil treatment unit) after a long period of operation and in the absence of continuous ponding of wastewater on top of the soil infiltrative surface.

Low rate process—A term that refers to an aerobic biological treatment process with respect to how fast it achieves biodegradation of organic matter in wastewater. A low rate process is one that typically has an extended period of aeration of the wastewater in order to achieve a secondary quality effluent.

Macrophyte—A class of vegetation that is used in constructed wetlands. Examples of macrophytes include emergent plants (e.g., bulrush, common reed, cattail), floating plants (e.g., water lily, duckweed, water hyacinth), and submerged plants (e.g., pondweed, American shoreweed).

Management systems—Management systems involve entities and activities, often organized within a jurisdiction, to ensure decentralized systems are properly considered during infrastructure and land use planning, and if selected they are properly designed, constructed, and operated so performance is satisfactory over a long-term planning period.

Manifold—(1) A small diameter solid wall pipe that is used to evenly distribute the influent to two or more laterals in a treatment unit (e.g., porous media biofilter or soil treatment unit). (2) In the context of a landscape drip dispersal unit, manifolds are small diameter solid wall pipe that include a supply manifold that is used to deliver a dose to a network of drip dispersal tubing and a return manifold that is used to capture the flow during a field flushing event.

Maximum Contaminant Level (MCL)—The highest level of a contaminant that is allowed in drinking water in the United States under the Safe Drinking Water Act.

Maximum day—Compared to an average day at a particular building or development, the maximum day can be defined as one that recurs periodically (e.g., recurring maximum day that occurs 1 day every month) or very rarely (e.g., extreme maximum day that occurs only 1 day every year).

Mean cell residence time (MCRT)—A design parameter that describes the average length of time that a microorganism remains in the aeration zone of an aerobic treatment unit.

Membrane bioreactor (MBR)—A unit operation for wastewater treatment that combines an aerobic bioreactor with a membrane unit for biomass separation and retention.

Methaemoglobinemia—A blood disorder that involves the presence of an elevated level of methemoglobin, a form of hemoglobin, that is useless for carrying oxygen in a human body. Since hemoglobin is the key carrier of oxygen in the blood, its replacement by methemoglobin can cause a slate gray-blueness of the skin (cyanosis) and potentially cause more serious symptoms due to insufficient oxygen.

Microfiltration—Filtration though a membrane that is used for removal of micro-particles and macromolecules that are in the size range of 0.05–2.0 μm (e.g., bacteria, asbestos, paint pigments).

Minimum flow fixtures and appliances—Fixtures and appliances that use little or no water but still function properly.

Mixed liquor—A term used to refer to the contents of the aeration zone (compartment, tank, basin) within an activated sludge biological treatment system.

Mixed liquor volatile suspended solids (MLVSS)—A measure of the biomass cells in the mixed liquor within an aeration zone (i.e., a compartment, tank, or basin).

Monomedia—A term that describes a filter bed that is characterized by having a single layer of the same media.

Nanofiltration—Filtration though a membrane that is used for removal of molecules and constituents in the size range of 0.0008–0.006 μm (e.g., viruses, sugars, dyes).

Natural disinfection—Refers to the destruction of pathogenic microorganisms by die-off and predation mechanisms in unit operations that are not specifically designed as disinfection agent technologies. See also Disinfection.

Natural Resources Conservation Service (NRCS)—An organization of the U.S. Department of Agriculture concerned with the description, mapping, and preservation of natural soils, waters, and other resources associated with the landscape.

Natural system unit operation—Refers to treatment systems that involve natural processes and are typically open in the environment and rely on natural environmental processes for wastewater treatment and water reclamation. Constructed wetlands, subsurface soil treatment units, and landscape drip dispersal units are examples of natural systems.

Nitrification—Nitrification involves the conversion of ammonia nitrogen to nitrite and nitrate nitrogen under aerobic conditions by autotrophic bacteria that utilize O_2 as an electron acceptor and CO_2 as a carbon source.

Nominal hydraulic retention time (HRT_N)—(1) The average time the liquid entering a tank, basin or compartment remains in it during flow from the inlet to the outlet not accounting for the presence of porous media and inactive flow zones. (2) In a constructed wetland the average time the liquid entering it remains in the wetland during horizontal flow from the inlet to the outlet not accounting for the presence of porous media and inactive flow zones.

Nominal volume (V_N)—(1) The volume defined by the bottom, sides, and top of a tank, basin or compartment not accounting for any objects within it. (2) In a porous media biofilter or a constructed wetland, nominal volume does not account for any porous media, vegetation, or inactive flow zones.

Nonpotable—Water that has a quality that makes it unsafe for use as a source of safe drinking water but suitable for other purposes such as toilet flushing or landscape irrigation.

Nonresidential—Buildings that are used for purposes other than providing residency for day-to-day living by individuals or families. Buildings can be used for commercial, institutional, recreational, or other purposes. Examples of nonresidential buildings include: hotels, motels, restaurants, laundromats, schools, veterinary clinics, gasoline service stations, highway rest stops, and recreational park facilities.

Onsite water reclamation—In the context of decentralized infrastructure, onsite refers to wastewater treatment and discharge or reuse that occurs on the same property as the source of the wastewater generation (e.g., house, business, institution).

Operation and maintenance (O&M)—Refers to the set of activities and events involved in ensuring proper function of a unit operation (e.g., a septic tank, aerobic treatment unit, soil treatment unit, etc.).

Organic loading rate (OLR)—The mass of organic matter (typically measured as lb-BOD_5/day/ft^2) that is applied to the surface of a treatment unit (e.g., porous media biofilter, constructed wetland, soil treatment unit).

Orifice—A perforation (typ. 1/8-in. diameter +/−) in the wall of a lateral pipe in a pressure distribution system through which the pressurized influent is discharged onto a porous media in a treatment unit.

Orifice shield—Refers to a cup or other protective capping device that helps disperse the discharge from an orifice while protecting the orifice from blockage by porous media used in a biofilter.

Oxidation—A chemical reaction in which an oxidant species accepts an electron(s) and a reductant species gives up an electron(s).

Overflow rate—The rate at which liquid in a secondary clarifier flows over the weirs within it to exit the aerobic treatment system.

Oxidation ditch—A type of aerobic treatment system that involves wastewater being aerated as it flows around an oval channel.

Ozonation—Refers to the process of dissolving ozone gas in water, which leads to generation of free radicals (e.g., hydroxyl radicals) that are powerful oxidants.

Ozone—In this chapter, refers to ozone (O_3) as a powerful oxidant and disinfectant. O_3 destroys cell walls, nucleic acids, and C:N bonds.

Packaged media biofilter (PBF)—Packaged media biofilters are commercially manufactured in modular units containing porous media that is lightweight and suitable for shipping and has a high surface area per unit volume and weight. Examples of PBF media include: peat, foam, textile, and polystyrene beads.

Pathogen—An agent such as a living microorganism or particle that can cause disease. Pathogens that can cause disease in humans include a variety of bacteria, virus, protozoa, fungi, and helminthes.

Peaking factor—A multiplier used to estimate a peak flow rate compared to an average flow rate (e.g., maximum day flow compared to average day flow).

Peat media—A type of media used in packaged media biofilters. Peat is a soil-like material that is **a** heterogeneous mixture of decomposed plant material that has accumulated in a water-saturated environment and in the absence of oxygen. When used in a PBF, peat media is containerized in manufactured modules or pods.

Peracetic acid (PAA)—A quaternary mixture of peracetic acid, hydrogen peroxide, acetic acid, and water.

Percolation—Refers to water movement that occurs in a downward direction from below a soil infiltrative surface through a soil profile under unsaturated flow conditions. Can also refer to unsaturated water movement through the media in a porous media biofilter.

Percolation test—Refers to a crude test procedure to measure a soil's infiltration capacity for clean water that is based on ponding clean water in a 6–12 in. diameter borehole for a period of time and measuring the rate of decline in the water level to determine the 'perc rate' in min/in. Perc rates have been used in the past (and still are in some locations) to judge site suitability and guide selection of a hydraulic loading rate for design but the test procedure is crude and the rate measured is dependent on test conditions and operator behaviors. Use of perc rates is generally not recommended for siting and design of a STU.

Performance-based design—An explicit approach to achieving performance that allows designers to develop solutions to achieve a numerical performance requirement (e.g., 10 mg-N/L) that can provide for flexibility and innovation in design, but can require monitoring to verify performance.

Peri-urban—A term that is used to refer to a residential area or mixed-use area that exists between suburban areas and the countryside. See Suburban.

Permeate—Refers to the water that passes through a membrane.

Phosphorus sorption capacity (PSC)—Refers to the ability of a porous media to remove phosphorus from wastewater or other impaired waters by sorption processes. The PSC is typically expressed in terms of the weight of P sorbed to a unit weight of dry porous media (e.g., 1 g-P per kg media dry wt.).

Physicochemical—A term used to refer to processes and reactions that have both physical and chemical characteristics. Sorption is an example of a physicochemical process.

P-k_A-C* modeling—An approach to modeling constituent removal in a constructed wetland that is based on representing the flow regime as plug-flow like using a number of tanks in series but which accounts for a departure from plug flow due to hydraulic inefficiencies as well area-based reaction rate constants that decline with distance from the inlet to the outlet of the wetland.

Plug flow—A flow regime where the velocity of the fluid is constant across any cross-section of the tank, basin or other unit perpendicular to the axis of the inlet to the outlet flow path.

Plume—A term that refers to the extent of measurable concentrations of one or more constituents contained in groundwater that are derived from wastewater that recharges groundwater under a site used for land-based treatment. The nature and extent of the plume is different for reactive constituents (e.g., BOD, NH_4^+), which are retarded compared to nonreactive constituents (e.g., Cl^-), which are not.

Point of compliance—Refers to the location in space associated with a decentralized system(s) where a water quality criteria must be satisfied (e.g., NO_3-N concentration $\leq$10 mg-N/L). An example point of compliance is the effluent discharged from a confined unit operation such as a textile biofilter. Another example for a soil-based treatment operation is the groundwater quality measured in a groundwater observation well placed at the downgradient property line or in the groundwater as it reaches the edge of a local stream.

Polonite®—A media ($CaO \cdot SiO_2$, $CaSiO_3$) that is derived from mining and processing calcium silicate rock and has a high porosity and specific surface area. It has a high phosphorus sorption capacity and has been used in phosphorus sorptive filters.

Porous media biofilter (PMB)—A term used to describe a wastewater treatment unit operation that involves media placed in a container through which a wastewater flows by gravity and receives treatment, primarily by attached growth biological processes. There is porosity within the bed of media by virtue of spaces between adjacent particles, filaments, or media objects. There can also be internal porosity within some types of media used in PBFs (e.g., foam cubes).

Potable—Water that has a quality that makes it safe to use as a source of safe drinking water.

Potential evapotranspiration (PET)—The maximum amount of water removed by evaporation and transpiration under the current environmental conditions if water supply to plants is unlimited.

Precipitation—Refers to forms of water (e.g., rain, sleet, snow) that fall from the sky toward the land surface. A chemical process that involves reactions where a dissolved substance is removed from solution by conversion to a solid substance that can be physically separated from the solution. An example of chemical precipitation involves the removal of phosphate from solution by addition of lime ($Ca(OH)_2$) to create a hydroxyapatite solid ($Ca_{10}(PO_4)_6(OH)_2$) that forms when lime is used to raise the $pH > 10$.

Preliminary treatment—A term used to encompass processes and unit operations that are used to accomplish the initial processing of raw wastewaters generated in buildings, which often includes the removal of debris and fats, oils, and greases. Examples of preliminary treatment include: grease interceptors, coarse screening units, grinders and comminutors.

Prescriptive design—An implicit approach to achieving performance where regulatory requirements dictate the steps and methods to be adhered to in system planning, design, and operation and satisfactory performance is presumed to be achieved if the prescribed code requirements are met.

Predation—Refers to the process of bacteria being killed by protozoa.

Pressure-compensating emitters—A drip emitter that is designed to function where the flow rate dispersed is largely constant across a wide range in pressure within the drip tubing.

Pressure distribution—Refers to a method of distributing wastewater over the horizontal infiltrative surface within a treatment unit operation such as a porous media biofilter or a soil treatment unit. Pressure distribution is often used to help achieve more uniform application of wastewater to all portions of the infiltrative surface from startup through longer-term operation.

Primary treatment—A term used to encompass processes and unit operations that remove suspended solids (organic and inorganic) from wastewater by sedimentation or flotation processes. Advanced primary treatment includes some treatment of the separated solids (e.g., by anaerobic biodegradation of settled organic solids). Examples of primary treatment operations include: settling basins, septic tanks, and upflow anaerobic sludge blanket reactors.

Quality—Quality is a qualitative term used to describe the degree of “impairment” of a water due to use and changes in composition (e.g., low vs. high quality).

Reaction rate—For a reaction, the measure of the change in concentration of the reactants or the change in concentration of the products per unit time.

Recirculating sand filter (RSF)—A type of biofilter that is characterized by a bed of coarse sand or gravel to which primary or better quality effluent is intermittently dosed and filtrate is recycled for several passes through an unsaturated aerobic filter bed during which advanced secondary treatment can be achieved.

Recirculation—The process of directing a portion of the filtrate from a multiple-pass biofilter back to a recirculation/dosing tank where it is blended with the incoming wastewater (e.g., septic tank effluent) for dosing of the biofilter.

Recirculation ratio—The ratio of the daily filtrate flow that is recycled compared to the daily incoming flow. Recirculation ratios are typically 3–5.

Reclaimed water—Reclaimed water is wastewater that has been treated to remove inorganic and organic substances and pathogenic microorganisms to a degree that the wastewater can be considered reclaimed water with a quality that is fit for the purpose (i.e., appropriate for and of a necessary standard) of an intended discharge or water reuse plan.

Redox zones—Refers to a condition in a constructed wetland where based on sources and sinks for dissolved oxygen there can be zones that are aerobic, anoxic, or anaerobic.

Rehabilitation—See Rejuvenation.

Rejuvenation—A term that refers to correcting a deficiency in the function of a treatment unit that is responsible for a poor hydraulic or purification performance. Rejuvenation is often used in the context of porous media biofilters, soil treatment units or nutrient reduction filters. Rehabilitation is often considered to be synonymous with rejuvenation.

Residential—buildings that are used for individuals or families to live in over extended periods. Examples of residential buildings include single houses, apartments buildings, and condominium buildings.

Residual—(1) Refers to the presence of a disinfectant agent in disinfected wastewater (or other impaired waters) after the wastewater exits the disinfection unit operation. (2) Refers to waste products and materials that result from the routine operation and maintenance of a treatment unit operation.

Responsible management entity (RME)—Refers to an individual, company, organization or government agency that has responsibility for the design and implementation and the sustained operation and successful performance of a decentralized wastewater treatment system.

Risk factors—In the context of decentralized wastewater treatment and water reclamation, risk factors are engineering and environmental attributes relevant to an approach, unit operation or system that contribute to the likelihood that the approach, unit operation or system will experience a performance dysfunction or failure that could potentially cause an adverse and undesirable effect on public health or environmental quality.

Rotating biological contactor (RBC)—A type of aerobic treatment system that consists of a tank through which wastewater flows and a cylindrical unit containing closely spaced disks that are supported on a rotating shaft just above the surface of the wastewater in the tank. Microorganisms grow on the disks and contact substrate as the disks are rotating through the wastewater and receive aeration through passive means via exposure to the atmosphere as they rotate above the liquid level.

Run—(1) Refers to the length of time that a filter or other unit operation functions before maintenance or rejuvenation is needed. (2) Refers to a length of drip dispersal tubing that leads away from or returns to a supply manifold.

Sand—A naturally occurring granular material composed of finely divided rock and mineral particles. Sand can be further defined as particles with a diameter of 0.05–2.0 mm. Sand is also a textural class of soil along with silt and clay.

Sanitation—A term that refers to the processes, systems and services used to prevent human contact with the hazards of wastes and wastewaters and provide for effective treatment and proper disposal of wastewater. According to the World Health Organization, inadequate sanitation is a major cause of disease worldwide and improving sanitation is known to have a significant beneficial impact on health both in households and across communities.

Saturated hydraulic conductivity (K_S)—A term that is associated with the ability of a porous media to transmit water through it. K_S (e.g., gal/day/ft^2) is multiplied by the hydraulic gradient (e.g., typ. 1.0 for biofilters or soil treatment units) and cross-sectional area (e.g., ft^2) to determine the hydraulic capacity (e.g., gal/day) through the porous media.

SCADA—An acronym for supervisory control and data acquisition systems that are used to gather and analyze real-time data to monitor and control a unit operation or system.

Scouring—A term that refers to the removal of solids that could accumulate in a sewer pipe during wastewater flow through it. Scouring velocity is that velocity which is sufficient to transport the solids and mitigate their deposition and accumulation. In a conventional gravity sewer for untreated wastewater a scouring velocity is typically 2 ft/s. In a STEG or STEP sewer system a scouring velocity can be near zero since these sewers convey septic tank effluents that have no gross solids or debris and only very low levels of suspended solids.

Secondary treatment—A term used to encompass processes and unit operations that follow primary treatment and are designed to remove biodegradable dissolved and colloidal organic matter by aerobic biological processes. Advanced secondary treatment includes transformation and removal of nutrients (e.g., a nitrifying extended aeration bioreactor).

Examples of secondary treatment operations include: extended aeration bioreactors, porous media biofilters, and constructed wetlands.

Sedimentation—The physical process by which settleable solids are separated from liquid wastewater by gravity forces.

Seepage pit—An older form of land-based waste disposal that was used for direct release of partially treated wastewater (e.g., after treatment in a septic tank) into the deeper subsurface to keep wastewater away from direct contact with humans. Seepage pits have caused soil and groundwater contamination and are no longer used in most locations of the United States. Seepage pits are also referred to as dry wells or soakaways.

Septage—The sludge and scum that is separated and retained within a septic tank and requires periodic removal and proper management.

Septage management—Septage management encompasses the removal of septage from a septic tank (typically by pumping) followed by the proper management for its treatment and disposal or beneficial recovery. Options for septage management typically include: land treatment integrated with agriculture, discharge to a local wastewater treatment plant or discharge to a specially designed treatment facility. In the United States, septage is managed as a regulated waste under Federal regulations (40 CFR Part 503).

Septic tank—A watertight tank with an inlet and outlet that combines solids separation and anaerobic digestion to achieve advanced primary treatment of wastewater. A septic tank has one or more compartments within which settling and flotation can occur and where the sludge and scum that is separated can undergo anaerobic digestion.

Septic tank effluent (STE)—The liquid that is discharged from a septic tank under gravity flow or by intermittent pumping.

Sequencing batch reactor (SBR)—A type of aerobic treatment system that consists of a single tank within which multiple steps occur in sequence. The steps typically include: fill, aerate, settle, decant and idle (waste biosolids).

Settleability—A term that refers to the tendency of a biomass suspended in wastewater to settle and compact by gravity under quiescent conditions.

Sewer—A pipeline that is typically located below ground and used to convey wastewaters (untreated or treated) from one location to another.

Sewer system—A network of sewer lines that collect and convey wastewaters (untreated or treated) from one or more sources to the site(s) where treatment and discharge or reuse will occur. Depending on the type of sewer system, there can also be pumps, pump basins, controls, valves, cleanouts and other components that are part of the system and needed for the system to function properly.

Shielding—Refers to the process where suspended solids can prevent ultraviolet light from reaching and destroying pathogenic microorganisms during disinfection of treated wastewater.

Short-circuiting—Refers to the process where flow through a treatment unit operation is not uniform and some portion of the influent reaches the outlet from the unit much sooner than the balance of the influent does.

Sidewall area—The vertical sides of an infiltration unit (e.g., trench or bed) through which wastewater can infiltrate in a horizontal direction. Sidewall area is only used for infiltration when the trench or bed it is part of is has intermittent or continuous ponding of the bottom area soil infiltrative surface.

Silt—A naturally occurring granular material composed of finely divided mineral particles. Silt can be further defined as particles with a diameter of 0.002–0.05 mm. Silt is also a textural class of soil along with sand and clay.

Single pass sand filter (SPSF)—A type of biofilter that is characterized by a bed of medium sand to which primary or better quality effluent is dosed and advanced secondary treatment can be achieved during a single pass through an unsaturated aerobic filter bed.

Siphon—A siphon is a hydraulic device that enables intermittent discharge from a tank or basin based on hydraulic processes as a liquid level rises in a tank or basin and that is equipped with an inverted bell, vent tube, and discharge leg. An automatic siphon does not require power to operate and represents an alternative to a submersible pump in some situations where intermittent discharge from a tank or basin is required (e.g., dosing a porous media biofilter or soil treatment unit).

Slope (S)—A measure of the change in elevation along segments of pipe within a STEG system or the change in the elevation of the energy grade line along segments of pipe within a STEP system. A measure of the change in elevation of a surface with distance (e.g., land surface where a soil treatment unit is installed, water surface in a constructed wetland).

Sloughing—Refers to the process by which biomass attached to support surfaces in an attached growth system such as a trickling filter (e.g., rocks or plastic honeycombs) separates from the surfaces and is carried out of the system.

Sludge—Sludge can have different meanings depending on the context. In general it refers to a liquid-solid mixture (mostly water with 1–10 % by wt. solids). As applied to wastewater it refers to the solids and associated water that are separated during the treatment of wastewater. This definition can include domestic septage and waste activated sludge.

Sludge age—A term that is sometimes used as a measure of how long biomass solids remain in an aerobic treatment system. Sludge age is defined as the total mass of MLVSS that are in the aeration tank divided by mass of VSS influent to the system.

Sludge volume index (SVI)—A crude measure of the settleability of mixed liquor solids resulting from aerobic biological treatment. It is defined as the volume of one gram of settled solids in one liter of solids containing wastewater (e.g., mixed liquor) after 30 min of quiescent settling.

Soil—Soil is a natural body comprised of solids (minerals and organic matter), liquid, and gases that occurs on the land surface, occupies space, and is characterized by one or both of the following: horizons, or layers, that are distinguishable from the initial material as a result of additions, losses, transfers, and transformations of energy and matter or the ability to support rooted plants in a natural environment.

Soil absorption system—A term that was used in the latter part of the 20th century for a soil-based wastewater system that primarily involved a subsurface means of infiltration that was used for disposal and treatment of domestic septic tank effluent.

Soil-based treatment system –Refers to a treatment unit operation or system that involves the use of soil as a treatment medium. Soil-based treatment systems may also be referred to as land-based treatment systems.

Soil treatment area (STA)—See Soil treatment unit.

Soil treatment unit (STU)—A term that was coined in the early part of the 21 st century in the United States to refer to a land-based wastewater treatment system that primarily involved a subsurface means of infiltration that was used for dispersal and treatment of wastewater including primary and secondary effluents. This definition is also used for a soil treatment area (STA).

Solids—(1) Earth media such as soil, sand, gravel, rock, and stones. (2) Scum and sludge that is separated from raw wastewater during treatment using solids separation methods (e.g., a septic tank or screening device). (3) Solid (and excess solids) that result from biological growth in a bioreactor used for aerobic biological treatment of wastewater (biomass) plus nonvolatile suspended solids that get entrained within flocs of biomass. (4) Porous media of various types used for different purposes during wastewater treatment and water reclamation.

Solids retention time (SRT)—A term that describes the length of time that solids produced during biological wastewater treatment are retained in the aeration zone. SRT is equal to the total mass of mixed liquor volatile suspended solids in the aeration zone divided by the mass of volatile suspended solids wasted from the system.

Soluble microbial products (SMP)—A mix of organic materials that are produced by microorganisms during growth when they are degrading organic materials in an activated sludge process. microbial growth.

Sorption—A general term used to refer to the process or processes that cause a substance in solution to become attached to a solid. Sorption is generally used to include absorption where a substance is incorporated into another substance (e.g., NH_3 gas absorbed into a basic solution) and adsorption where a substance is bound to the surface of another phase (e.g., PO_4^{-3} adsorbed to a soil mineral surface).

Source—Source is defined as the origin of the wastewaters that are generated and will be treated for discharge or reuse. A source can include an individual dwelling unit, an apartment building, a cluster of dwelling units, a commercial or institutional building, a development of residential and/or commercial buildings, a portion of a city-wide service area, etc.

Source separation—In decentralized systems, refers to the separation and separate management of individual wastes and waste streams. For example using dual plumbing systems, blackwater comprised of toilet wastes and kitchen sink wastewaters can be separated from graywater produced by basins, other sinks, and appliances. Another example is the diversion of urine from fecal wastes using a urine-diverting toilet to enable urine processing and use as a fertilizer.

Stabilization—Refers to the set of physicochemical and biological processes that decompose organic matter and reduce odors and destroy pathogenic microorganisms in solid and/or liquid waste material.

Stage—In wastewater and water treatment, can refer to a major component in a treatment train (e.g., first stage is primary treatment and a second stage is secondary treatment) or to parts of a component (e.g., a stage could be a single sequence of aerobic and anaerobic zones within a biological treatment component).

Stratified media—(1) A term used to describe a filter bed that has multiple layers of media that have different physical and/or chemical properties. (2) A term used to describe a soil profile that has layers of soil media that have different morphology and physical and/or chemical properties.

STUMOD—Acronym for a spreadsheet-based analytical flow and transport model that was developed to simulate the treatment of wastewater during subsurface soil infiltration and percolation in a soil treatment unit.

Styrene media—A type of media used in packaged media biofilters that is comprised of uniform plastic beads that are packaged in pillow-like forms within a woven polyethylene mesh.

Substrate—A term that describes the organic matter that microorganisms involved in biological treatment use as a source of organic matter and organic carbon.

Suburban—A term that is used to refer to a residential area or mixed use area that is geographically separated from a city or highly urbanized area but within commuting distance of it. Peri-urban is another term that is used to refer to residential or mixed-use development between suburban areas and the countryside.

Surface area—(1) A term that refers to the horizontal infiltrative surface area of a biofilter that receives a dose of influent. (2) The area of the external or internal surfaces of a particle, filament or other object. (3) The area of a horizontal or vertical plane through which wastewater is infiltrated into a soil profile, for example in a soil treatment unit. (4) Refers to the horizontal surface area defined by the perimeter of a constructed wetland as measured at the land surface.

Suspended growth—Refers to an aerobic biological process where the microorganisms involved in treatment are suspended in the liquid wastewater.

Sustainable systems—In the context of decentralized wastewater treatment and water reclamation, sustainable systems are systems that are selected, designed, and implemented for a particular application that are capable of achieving long-term, reliable performance, have affordable costs for construction and operation, and have acceptably low resource requirements and environmental impacts.

Technically viable systems—Decentralized systems for a particular application that are capable of achieving a required treatment efficiency for an intended discharge or reuse plan and are also capable of satisfying high priority owner requirements.

Tertiary treatment (Advanced treatment)—A term used to encompass processes and unit operations that typically follow secondary treatment and are designed to remove specific constituents such as nutrients, trace organic compounds, heavy metals or dissolved salts. Examples of tertiary treatment operations include: denitrifying porous media biofilters, adsorptive media packed bed reactors, and ion exchange columns.

Textile media—A type of media used in package media biofilters that is comprised of textile fibers configured in sheets that are draped over rods within a module.

Toilet wastewater—Toilet wastewater consists of urine and feces plus toilet tissue.

Total BOD (tBOD)—A measure of the total biochemical demand for oxygen exerted by microorganisms during complete degradation of organic matter and conversion of ammonium to nitrate.

Total dynamic head (TDH)—The pressure against which a pump or siphon must work to discharge a given flow rate.

Total Kjeldahl nitrogen (TKN)—A laboratory method of measurement that determines the concentrations of reduced forms of nitrogen. Total Kjeldahl nitrogen (TKN) includes organic N and ammonia N.

Trace organic compounds—Refers to a group of organic compounds that can occur in wastewater and other impaired waters that are derived from biogenic substances, pharmaceuticals, consumer product chemicals, pesticides, and flame retardants. These compounds can be present at very low levels but still be constituents of concern. Trace organic compounds are sometimes referred to as organic micropollutants.

Trans-membrane pressure (TMP)—Refers to the pressure required to drive liquid flow through a fine pore membrane.

Transmittance—In the context of disinfection, transmittance refers to the ability of ultraviolet light to penetrate into water, wastewater, or other impaired waters.

Transport piping—The solid-wall pipe that delivers wastewater from a pump tank to the location of a treatment unit (e.g., a porous media biofilter).

Treatment technique—A required process (in the United States) intended to reduce the level of a contaminant in drinking water.

Treatment train—Within a decentralized system a treatment train consists of a sequence of compatible unit operations that connect the source to an intended discharge or reuse option.

Treatment wetland—See Constructed wetland.

Ultrafiltration—Filtration through a membrane that is used for removal of molecules that are in the size range of 0.003–0.1 μm (e.g., viruses, proteins, colloidal silica).

Ultraviolet light irradiation—A disinfection technology used to destroy pathogenic microorganisms. Radiation around 260 nm penetrates the cell wall and is absorbed by cellular materials (DNA, RNA) and prevents replication or causes death of the microorganism.

Underdrain—That component of a biofilter or filter that exists at the bottom and is used to collect filtrate and convey it out of the biofilter or filter in a discharge pipe.

Uniformity coefficient (UC)—A measure of the uniformity of particles sizes in a mixture of particles (e.g., a volume of sand). The uniformity coefficient is defined as the ratio of D_{60} to D_{10} where D_{60} is the diameter that 60 % by wt. of particles are smaller than and D_{10} is the diameter that 10 % by wt. of particles are smaller than.

Unit operation—A physical facility (e.g., basin, column, reactor, landscape) in which a physical, chemical, and/or biological process is made to occur for the purpose of removing or destroying constituents of potential concern in wastewater or other impaired waters.

Unsaturated—A term used to describe the water content in porous media where the porosity is not completely liquid filled. In a porous media biofilter or a soil profile, unsaturated flow is important since the porosity in the media can contain liquid (i.e., the wastewater being treated) plus air-filled porosity and this helps maintain aerobic conditions through passive aeration. It also helps ensure that the wastewater applied percolates under film flow conditions with close contact to media surfaces.

Upflow anaerobic sludge blanket (UASB)—An advanced primary treatment unit that is designed with circuitous flow through a baffled tank so liquid contacts settled sludge. The tank can be sealed to enable collection of biogas.

Upset—A term that refers to a change in conditions that causes the function and performance of a treatment unit or system to deteriorate. Upset is often used in the context of biological treatment operations when changes to influent flow or composition adversely affect function and performance.

Urban—A term that refers to a geographic location with higher density development such as occurs in a city or metropolitan area.

Urine diverting toilets (UDTs)—Refers to a type of water flush toilet that has two separate discharge compartments and drainage lines: one for feces and toilet tissue and another for urine.

Vacuum sewer—A type of wastewater collection and conveyance system that relies on vacuum forces to transport untreated wastewater from individual buildings in relatively small diameter pipelines to a centralized vacuum station.

Vadose zone—A depth interval in the subsurface characterized by unsaturated conditions where pores in a porous media are filled with some volume of air as well as water. The vadose zone is also referred to as the unsaturated zone.

Vault toilet—Refers to a waterproof tank, lined pit, or similar containment structure into which human waste is deposited alone or in combination with a small volume of water.

Vector—In the context of water and sanitation, a vector is any insect, rodent or other animal capable of transmitting infectious disease-causing agents.

Vegetated subsurface bed wetland (VSB)—A type of constructed wetland for treatment of wastewater or other impaired waters that includes a bed of porous media that is planted with emergent vegetation through which impaired water or drainage from sludges that are being treated, flows in a horizontal direction from the inlet to the outlet.

Velocity (V)—A measure used to describe the speed of motion of a liquid in a pipe, channel or basin in units of length per time (e.g., ft/s).

Vertical flow—(1) Refers to a flow path that is upward or downward from one location to another. (2) Refers to constructed wetlands in which the wastewater being treated flows in a downward direction from an inlet to an outlet.

Vertical flow subsurface bed wetland—A type of constructed wetland for treatment of wastewater or other impaired waters and sludges that includes a bed of porous media that is planted with emergent vegetation through which impaired water or drainage from sludges that are being treated flows in a vertical direction from the top to bottom or vice versa.

Volumetric loading rate—A design parameter used to size the aeration basin of an aerobic bioreactor that is equal to the mass of organic substrate (BOD or COD) per day per ft^3 of aeration zone.

Waste activated sludge (WAS)—Refers to the excess biomass that is produced during aerobic biological treatment and requires periodic removal from a bioreactor to maintain a desired solids retention time. Waste activated sludge can also include organic and mineral matter that becomes associated with the biological flocs that are separated from the liquid (either by clarification or filtration). The excess solids are removed by pumping out a portion of the aeration compartment, tank or basin or by diverting a portion of the return flow from a clarifier as a sludge with a low solids content (e.g., 1 % by wt. or less).

Waste biological solids—Refers to excess biomass that is removed from aerobic biological treatment operations including waste activated sludge from aerobic treatment units and membrane bioreactors and also excess biomass that sloughs off of media in recirculating porous media biofilters.

Wastewater—Wastewater consists of water plus materials added during water use. The types and concentrations of materials depend on the characteristics of the source (e.g., house, restaurant, school, veterinary clinic). Materials can include human excreta, foodstuffs, consumer products, pharmaceuticals and personal care products, heavy metals, silt, etc.

Wastewater collection—Process and physical facilities involved in collecting wastewaters from individual sources using a sewer system.

Wastewater conveyance—Refers to the process of transporting wastewater (untreated or treated) under gravity or pressure forces from one location to another.

Wastewater management—A set of elements and activities that can encompass wastewater generation, collection and conveyance, treatment, discharge and recovery of resources (e.g., water, organic matter, nutrients, energy).

Water reclamation (wastewater treatment and discharge or reuse)—A modern term that refers to treatment of wastewaters or other impaired waters to improve the water quality by removing inorganic and organic substances and pathogenic microorganisms to the extent needed to permit safe release of the treated wastewater (reclaimed water) to the natural or built environment by a chosen discharge or water reuse option.

Water recycling—The process of reusing reclaimed water for a function within the source responsible for the wastewater that was treated to produce the reclaimed water (e.g., graywater produced within an office building is treated and the reclaimed water is used for toilet flushing in that building).

Water reuse—Use of reclaimed water for an intended beneficial purpose. Nonpotable water reuse includes landscape irrigation, ornamental uses, and toilet flushing. Potable water reuse includes using reclaimed water to augment sources of drinking water supplies (indirect potable reuse) or direct delivery into a drinking water supply (direct potable reuse).

Water use efficiency—Water use efficiency can encompass water use conservation measures with traditional fixtures and appliances (e.g., showering less frequently and for a shorter duration) or water efficient fixtures and appliances (e.g., a toilet with a lower flush volume per use).

Web Soil Survey—Refers to an online tool developed and maintained by the National Resources Conservation Service that enables the user to obtain descriptive and assessment information for specific parcels of land. http://websoilsurvey.nrcs.usda.gov/app/HomePage.htm

Wetland volumetric efficiency—refers to the fraction of the wetland water volume that is actually involved in water movement through the wetland accounting for the presence of inactive flow zones.

Yellow water—Term that can be used to represent human urine.

Yield (Y_N)—The net production of solids during aerobic biological treatment that is typically based on the characteristics of the wastewater being treated, the solids retention time, and the temperature.

Zero-order reaction rate—The rate of a reaction that is independent on the concentration of reactants and only dependent on time as long as the reactants are above a minimum level for the process and are not limiting. Biological denitrification processes are often expressed as zero-order reactions. First-order reaction rates are dependent on the concentration of one reactant (e.g., BOD_5) and second-order reaction rates are dependent on the concentration of two reactants (e.g., O_2 and BOD_5).

Zone—(1) Refers to a portion of a treatment unit to which influent is distributed during an individual dosing event. A treatment unit (e.g., porous media biofilter or a soil treatment unit) can have a single zone or a number of zones. Use of multiple zones can help with delivery and distribution (e.g., by reducing the discharge flow rate required by a dosing pump). (2) Refers to a portion of a constructed wetland that is characterized by different physical conditions (e.g., a portion with vegetated porous media vs. a portion with an open water surface) and may have different functions occurring (e.g., physical such as flow entering and being distributed at the inlet end of a wetland vs. biological such as in the portion of a wetland where treatment processes occur).

Zone control valves—Refers to an electrically actuated valve that can be programmed to direct a dose of wastewater to one or another zone of drip dispersal area based on a dosing schedule.

Appendix C

Acronyms, Abbreviations and Symbols

Appendix C presents a list of acronyms, abbreviations and symbols as they appear in the summary section of each chapter.

A'_{IS}	Area of horizontal soil infiltrative surface provided based on the STU layout
A'_{M}	Approximate total surface area of membranes required
A_{BS}	Cross-sectional surface area of the biofilter
A_{C}	Area provided by each unit
A_{c}	Surface area of the clarifier
A_{F}	Trickling filter surface area, Cross sectional area or Landscape footprint or dispersal area
AF	Adjustment factor
A_{FM}	Minimum footprint area required for dispersal
A_{FS}	Area of the filter surface
Ag	Silver
AGB	Attached growth bioreactor
A_{IS}	Area of the soil infiltrative surface
Al	Aluminum
ALR	Areal loading rate
A_{M}	Final total surface area of membranes required
AMC	American Manufacturing Co.
AOI	Area of interest
A_{S}	PMB surface area required based on design flow and HLRD
A_{S}'	PMB surface area provided based on a chosen L and W
ASTM	American Society of Testing and Materials
ATU	Aerobic treatment unit
Avg	Average
A_{W}	Total required surface area of the wetland ($L \times W$) or Horizontal surface area of a constructed wetland
A_{W}'	Wetland surface area actually provided (LW)

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AWWA	American Water Works Association
A_{xc}	Cross-sectional area perpendicular to flow
BDL	Below detection limit
BFS	Blast furnace slag
bgs	Below ground surface
BNR	Biological nutrient removal
BOD_5	Biochemical oxygen demand exerted after 5 days
BREEAM	Building research establishment environmental assessment method
BW	Blackwater
BWEFA	Basic water efficient fixtures and appliances
BWR	Basic water requirement
C	Concentration or Concentration of a constituent at the outlet or a fractional distance from the inlet, concentration of a solute in solution in equilibrium with the mass sorbed onto the solid, Hazen-Williams coefficient, or Orifice discharge coefficient
C&I	Commercial and institutional
C*	Background concentration of constituents
Ca	Calcium
C_A	Concentration of chlorine added to the wastewater being disinfected
$Ca(OH)_2$	Calcium hydroxide
cap	Capita (or persons)
cBOD	Carbonaceous BOD
$cBOD_5$	Carbonaceous biochemical oxygen demand measured over 5 days
C_{CS}	Concentration of chlorine in the wastewater being disinfected
Cd	Cadmium
C_{DC}	Chlorine decay with time
C_{DM}	Chlorine demand of the wastewater being disinfected
CDPHE	Colorado Department of Public Health and Environment
C_E	Effluent concentration
CFR	Code of Federal Regulations (United States)
CFU	Colony forming units
CGP	Construction Grants Program in the U.S.
C_{GW}	Concentration in groundwater
C_i	Concentration in the influent, or Concentration of a constituent in a particular waste stream
CI	Confidence interval
CIDWT	Consortium of Institutions for Decentralized Wastewater Treatment
CIP	Clean in place
Cl_2	Chlorine gas
C_O	Concentration of ozone in the wastewater being disinfected

CO_2	Carbon dioxide
COC	Constituent of concern
COD	Chemical oxygen demand
Coli.	Coliform bacteria
C_{PW}	Concentration in pore water
CR	Chlorine residual required in the wastewater being disinfected
C_{RAW}	Average concentration in raw wastewater
CSA	Canadian Standards Association
CSM	Colorado School of Mines
CSO	Combined sewer overflows
C_{STE}	Average concentration in septic tank effluent
CSTR	Continuously stirred tank reactor
CT	Concentration multiplied by time of contact
C_{T*}	Concentration of a particular constituent in the average daily flow after a source separated stream is removed
CW	Clothes washer, or Constructed wetland
D	Diameter or Dose of a disinfectant agent
d	Days or Depth of unsaturated media
D&I	Design and implementation
d.w.	Dry weight
D_{10}	Diameter that 10 % by wt. of particles in a mixture is smaller than
D_{60}	Diameter that 60 % by wt. of particles in a mixture is smaller than
D_B	Depth of the biofilter bed
DBP	Disinfection byproducts
d_F	Diameter of the filter vessel
D_F	Depth of the filter medium
DFU	Drainage fixture unit
dh/dz	Hydraulic gradient
D_L	True inside diameter of the laterals
D_M	True inside diameter of the manifold
DOC	Dissolved organic carbon
D_{PD}	Design doses per day or Dispersal events per day per zone
DU	Dwelling unit (e.g., homes, apartments, condominiums)
d_u	Depth of unsaturated soil
d_W	Water depth
DW	Dishwasher
DWRC	Decentralized Water Resources Collaborative
dX/dt	Rate of change of cells in the bioreactor
E	Estimated concentration
E. coli	Escherichia coli
EAF	Electric arc furnace slag
EBNR	Enhanced biological nutrient removal

EDU	Equivalent dwelling unit
EF	Efficiency factor
EGL	Energy grade line
EPA	Environmental Protection Agency
EPAct	United States Energy Policy Act
EQ	Exceptional quality
ERC	Engineering Research Center (NSF)
ES	Effective size (D10)
ET	Evapotranspiration
eV	Wetland volumetric efficiency accounting for inactive flow zones
f	Ratio of BOD5 to ultimate BOD
F	Faucet or Conversion factor
F/M	Food to microorganism ratio
F'_M	Membrane flux producing effluent from the MBR
FAQ	Frequently asked questions
f_b	Fraction of TSS and FOG separated as scum and sludge that are biodegraded
F_B	Fraction of TSS + FOG removed in the tank that remain as septage solids
F_C	Membrane flux used for cleaning
Fe	Iron
Fecal coli.	Fecal coliform bacteria
F_H	Fraction of indoor water use that is hot water use
F_M	Design membrane flux limit
FOG	Fats, oils and greases
FOS	Factor of safety
F_R	Fractional reduction in use from water efficient fixtures and appliances
F_{R-H}	Fractional reduction in hot water use due to water efficient fixtures and appliances
F_S	Fraction of total tank volume occupied by solids
FSM	Fecal sludge management
F_U	Frequency of use (e.g., 2 urinal flushes per person per day)
FWS	Horizontal flow free water surface wetland
g	Gram or Acceleration due to gravity
GAC	Granular activated carbon
gal	Gallon
GIS	Geographic information system
GW	Graywater or Groundwater
H	Height of ponding above a soil infiltrative surface
HDI	Human Development Index
h_{dv}	Headloss through a distributor valve

h_e	Change in elevation between the water level in the septic tank and the service lateral connection point
h_f	Headloss caused by friction during flow in a length of pipe
h_{fl}	Headloss due to flow in a lateral between the inlet-end and the far-end orifices
h_{fm}	Headloss due to flow in the manifold piping
h_{ftp}	Headloss due to flow in the transport piping and fittings
HGL	Hydraulic grade line
h_{hv}	Friction losses in the discharge assembly of a STEP system pumping unit
h_l	Friction losses in the service lateral
HLR	Hydraulic loading rate
HLR_A	Hydraulic loading rate that is actually applied
HLR_D	Hydraulic loading rate used for design
h_{md}	Headloss in the pump discharge assembly
HOCl	Hypochlorous acid
h_p	Pressure head needed to transport QDP flow in the main sewer line of a STEP system
h_r	Residual head at the distal orifice
HRT	Hydraulic retention time
HRT_E	Effective hydraulic retention time
HRT_N	Nominal hydraulic retention time
h_s	System static or elevation head
i	Infiltrability or Different contributions (e.g., meals, guest toilet use, employee uses) within a particular source (e.g., motel), or Sources contributing to the flow being estimated in one building (e.g., restrooms, locker room, laundry services)
I	Supplemental irrigation, or Intensity of ultraviolet light irradiation
I&I	Infiltration and inflow
IAPMO	International Association of Plumbing and Mechanical Officials
ID	Inside diameter or True inside diameter of a pipe
I_E	Intensity of UV radiation accounting for lamp aging and transmittance
IR	Infiltration rate
IR_o	Infiltration rate at startup
IR_t	Infiltration rate at time, t
IS	Infiltrative surface
ISA	Infiltrative surface architecture
ISU	Infiltrative surface area utilization
j	Different buildings that are present in a development (e.g., office building, restaurant) or Different sources contributing to the development flow being estimated, such as a motel gas station, cafeteria, etc.

k	Reaction rate constant, First-order reaction rate constant, or μ_m/Y
K	Constant or Potassium or Distribution coefficient for sorption
k_{20}	Reaction rate constant at 20 C
k_A	Area-based reaction rate constant
K_C	Saturated hydraulic conductivity of a crust
k_d	Endogenous decay coefficient
K_D	Linear distribution coefficient for sorption
K_E	Effective saturated hydraulic conductivity
kg	Kilogram
kL	Kiloliter
K_S	Substrate concentration at 50 % of μ_m or Saturated hydraulic conductivity
KU	Unsaturated hydraulic conductivity
kWh	Kilowatt-hour
L	Length or Liter
L:W	Length to width ratio
LCA	Life-cycle assessment
LDU	Landscape drip dispersal unit
LECA	Lightweight expanded clay aggregate
LEED	Leadership in energy and environmental design
LF	Lineal feet
L_{FPer}	Length of the PMB perpendicular to the lateral orientation
LF_Z	Lineal feet of tubing in a zone
L_I	Length of the trench or narrow bed or other infiltration unit
L_L	Length of laterals in the PMB or zone of it
LLR	Linear loading rate
L_M	Length of the manifold
L_N	Number of laterals in a zone
LTAR	Long-term acceptance rate
L_U	Length of the undisturbed land between adjacent zones
LWA	Lightweight expanded clay aggregate
M	Minutes of usage per day (e.g., 8 min of faucet use per person per day)
MASSTC	Massachusetts Alternative Septic System Test Center
MBR	Membrane bioreactor
MCRT	Mean cell residence time
M_d	Mass discharge rate
MFFA	Minimum flow fixtures and appliances
MGD	Million gallons per day
Mi	Mass in a particular stream
min	Minute
MLSS	Mixed liquor suspended solids
MLVSS	Mixed liquor volatile suspended solids

MPN	Most probable number
M_S	Separation of the first orifice from the manifold
M_{SM}	Mass of filter media
M_{SS}	Mass of a particular constituent in the average daily flow of the source separated waste stream
M_T	Mass of a particular constituent in the average daily flow of a combined wastewater stream
M_{VLR}	Mass volumetric loading rate
N	Number of activities or events (e.g., 4 toilet flushes per person per day), Number of microorganisms present, or Nitrogen
N'_C	Estimated number of units required
Na	Sodium
NaOCl	Sodium hypochlorite
nBOD	BOD caused by biological nitrification of ammonia
N_{BR}	Number of bedrooms (bedrooms in a dwelling unit)
NCl_3	Trichloride
ND	None detected
N_D	Nitrogen removal by denitrification
N_{DE}	Efficiency of nitrogen removal by denitrification
N_{DU}	Number of dwelling units (e.g., homes, apartments, condominiums)
NDWRCDP	National Decentralized Water Resources Capacity Development Project
N_E	Effluent TKN concentration
n_e	Effective porosity contributing to flow
NEDU	Number of EDUs
NH_2Cl	Monochloroamine
NH_4^+	Ammonium nitrogen
$NHCl_2$	Dichloroamine
N_I	Influent TKN concentration
N_L	Number of laterals or Nitrogen loading via deep percolation
N_{LS}	Nitrogen loading to the edge of a surface water
N_O	Total number of orifices in the PMB or zone of it
N_o	Number of microorganisms present initially
NO_3^-	Nitrate nitrogen
NOAA	National Oceanic and Atmospheric Administration
NOLat	Number of orifices in a lateral
NOx	Nitrous oxides (NO, NO_2)
NO_X	Sum of $NO_2^- + NO_3^-$
N_P	Household size (number of persons)
NPCA	National Precast Concrete Association
NPV	Net present value
NR	Not reported
NR + F	Nitrogen added by rainfall and fixation

NS	Not specified
N_S	Number of a given unit of expression in a source (e.g., number 237 of motel rooms,. . .) or Nutrient load from a source (e.g., house)
NSF	National Sanitation Foundation (U.S.) or National Science Foundation (U.S.)
NTU	Normal turbidity units or Nephelometric turbidity units
N_U	Number of users (e.g., 6 males using a urinal), Number of units (e.g., persons) causing a water-using event or activity during a given period (e.g., guests per motel room), or Net plant uptake and storage of nitrogen (same as UP)
N_{WW}	Nitrogen applied in effluent dispersed
N_Z	Number of zones
O&M	Operation and maintenance
O&P	Operational and performance
O_2R	Total oxygen requirement
O_3	Ozone
OCl^-	Hypochlorite ion
OLR	Organic loading rate
OM	Organic matter
OR	Overflow rate
OR_D	Overflow rate for design
P	Apparent no. of tanks in series for modeling that varies by constituent to account for weathering based on field data or Phosphorus or The number of tanks in series to represent the departure from plug flow or precipitation or Person (or capita)
PAA	Peracetic acid
PAO	Phosphorus accumulating organisms
Pb	Lead
PBF	Packaged media biofilter
P_{BR}	Persons per bedroom
Perc	Deep percolation
PET	Potential evapotranspiration
PF	Peaking factor
PFU	Plaque forming units
P-kA-C*	Refers to an approach to mathematical modeling of constituent removal in a constructed wetland
P_M	Mean monthly precipitation
PMB	Porous media biofilter
PO_4^{-3}	Phosphate
P_{off}	Time a pump is off between dosing events or Time a pump is off between dispersal events under average daily flow conditions

P_{on}	Time a pump is running during dispersal to a zone or Time a pump is running during a dosing event
ppm	Parts per million
Prec	Precipitation
PRFP	Process to further reduce pathogens
PSC	Phosphorus sorption coefficient
psi	Pounds per square inch
PTIS	Acronym for “P-kA-C* tanks in series”
PW	Soil pore water
P_X	Waste activated sludge solids that needs to be wasted
Q	Flow rate by fixture or appliance use (e.g., 2.5 gal/min for a showerhead) or Daily flow rate (design or actual)
q	Rate of water movement through a cross-sectional area or Area-based design hydraulic loading rate
Q_A	Average daily flow rate or Average daily indoor use in a DU with traditional plumbing or Average indoor water use per household per day
Q_A/DU	Average DU flow when total flow is normalized to DUs contributing
Q_A/P	Per capita average daily flow rate (gal/day per capita)
QA/QC	Quality assurance/quality control
Q_{A-FA}	Average daily indoor use contribution of fixtures and appliances
Q_{A-Hot}	Average daily indoor hot water use in a DU with traditional fixtures and appliances
Q_{A-R}	Average indoor use in a DU with efficient fixtures and appliances
Q_C	Hydraulic capacity or Hydraulic capacity for flow through a constructed wetland or Daily permeate flow used for cleaning a unit or Rate of Chlorine addition
q_C	Rate of water movement through a cross-sectional area of a crust-topped soil
Q_{CAP}	Flow capacity of a segment of a sewer system
Q_D	Design daily flow rate
Q_{Dis}	Flow rate during dispersal in a zone
Q_{DP}	Design flow rate for sizing a segment of a sewer system
Q_E	Effluent flow rate or Discharge rate from an emitter
Q_{EDU}	Discharge rate for each EDU contributing to a sewer system
Q_F	Forward flow rate
Q_{Flu}	Flow rate during flushing in a zone
Q_i	Influent flow rate or Flow rate of a particular waste stream
Q_L	Average daily water use contribution due to leakage
Q_{Lat}	Flow rate into a lateral
Q_M	Methane produced per time or Flow rate into the manifold

Q_{MIN}	Discharge rate from a single connection to a sewer system
Q_O	Average daily water use contribution by other activities and events or Orifice discharge rate
Q_{OF}	Flow rate out the orifice furthest away from the manifold
Q_{OI}	Flow rate out the orifice closest to the manifold
Q_P	Flow rate a pump needs to be able to deliver against the TDH, Permeate produced, or Peak flow rate
Q_R	Recycle flow rate or Filtrate recirculation flow rate
Q_S	Flow rate required for scouring
Q_{SS}	Average daily flow of the source separated waste stream
Q_T	Average daily flow in a combined wastewater stream
q_U	Rate of water movement through a cross-sectional area of an unsaturated soil profile
Q_U	Lumped flow rate per unit of expression (e.g., gal/day per guest), Flow rate during an activity (e.g., 2.5 gal/min during showering) or Flow rate during a water use (e.g., 2.5 gal/min during showering)
Q_{VLR}	Flow volumetric loading rate
Q_W	Daily flow rate of waste solids or Average daily flow of waste solids from the MBR
R	Hydraulic radius or Reaction or Recycle ratio or Ratio of recirculated flow to the influent flow
R_A	Ratio of average concentration in raw wastewater to septic tank effluent or Aspect ratio
RAS	Return activated sludge
RBC	Rotating biological contactor
R_C	Hydraulic resistance to flow through a crust
r_d	Rate of endogenous decay
R_E	Reduction efficiency (%) or Removal efficiency
ReNUWIt	Reinventing the Nation's Urban Water Infrastructure
REUWS1	Residential end uses of water study 1
REUWS2	Residential end uses of water study 2
R_F	Septage removal frequency
R_{FV}	Ratio of network piping to dose volume
r_g	Rate of bacterial growth
r_g'	Net rate of bacterial growth
RL	Reporting limit
RME	Responsible management entity
R_{NR}	Rate of NO_3^- removal
R_Q	Ratio of discharge out of the distal orifice vs. the closest orifice
RSF	Recirculating sand biofilter
R_{STU}	Fractional removal of N or P in a soil-based treatment unit (e.g., infiltration trenches)
r_{su}	Rate of substrate utilization

R_{TU1}	Fractional removal of Nor P in a 1st treatment unit (e.g., septic tank)
R_{TU2}	Fractional removal of Nor P in a 2nd treatment unit (e.g., sand filter)
S	Shower, Substrate, Hydraulic gradient of water surface from inlet to outlet, Mass of solute sorbed per mass of media (mg/kg), or concentration of limiting substrate, Sulfur or Slope of a sewer line
S&GA	Fractional removal of N or P in subsurface soil and groundwater by attenuation during water movement from the STU boundary to the edge of stream
S1, S2...	Source contributing to the development wastewater generation
SBR	Sequencing batch reactor
SC	Solids concentration in septage
SCADA	Supervisory control and data acquisition
SD	Standard deviation
S_{DM}	Standard deviation of monthly precipitation
S_E	Effluent concentration of substrate (BOD or COD) or Spacing between emitters along the tubing
S_F	Safety factor for nonideal conditions in clarifiers
SFE	Sand filter effluent
SGB	Suspended growth bioreactor
S_I	Influent substrate (BOD5 or COD) concentration
S_{in}	Influent concentration of TSS and FOG solids
S_L	Separation distance between adjacent laterals
SLR	Solids loading rate
S_M	Mass of septage generated
SMP	Soluble microbial products
S_o	Distance between orifices in a lateral
SO_2	Sulfur dioxide
SO_4^{-2}	Sulfate
S_{out}	Effluent concentration of TSS and FOG solids
SPSF	Single pass sand biofilter
SRT	Solids retention time
SSO	Sanitary sewer overflows
SSWMP	Small Scale Waste Management Project
ST	Septic tank or system type
S_T	Spacing between parallel tubing lines
STA	Soil treatment area
STE	Septic tank effluent
STEG	Septic tank effluent gravity sewer
STEP	Septic tank effluent pressure sewer
STU	Soil treatment unit

STUMOD	Soil treatment unit model
S_V	Volume of septage generated
SVI	Sludge volume index
S_W	Separation distance of a lateral from a sidewall
T	Temperature or Contact time
t	Time
T90	Time to 90 % inactivation of pathogens
tBOD	Total BOD includes long-term carbonaceous BOD plus nitrogenous BOD
T_D	Portion of a day during which dispersal occurs or Portion of a day during which dosing occurs
TDH	Total dynamic head
TF	Toilet flush
TFU	Textile filter unit
TIN	Total inorganic nitrogen
TIS	Tanks in series
TKN	Total Kjeldahl nitrogen
TMDLs	Total maximum daily loads to a water body
TMP	Trans-membrane pressure
TN	Total nitrogen
TOC	Total organic carbon
TP	Total phosphorus
TS	Total solids
TSS	Total suspended solids
T_U	Time used during an activity (e.g., 8 min per shower)
TVS	Total volatile solids
TVSS	Total volatile suspended solids
U.S.	United States of America
UASB	Upflow anaerobic sludge blanket reactor
UC	Uniformity coefficient (D60/D10)
UDT	Urine diverting toilet
UN	United Nations
UPC	Uniform Plumbing Code
USEPA	United States Environmental Protection Agency
U_U	Uses per NU per time period (e.g., one toilet flush per person per day)
UV	Ultraviolet light
UVT	Ultraviolet light transmittance
V	Volume or Volume of each activity or event (e.g., 5 gal per toilet flush) or Volume of effluent processed at media saturation, or Velocity of flow in a sewer line
v/v	Volume per volume

V_A	Volume of the aeration zone (or aeration tank or basin) or Volume of wetland containing water active in flow or Volume of the aerobic tank
VAR	Vector attraction reduction
V_B	Bulk volume of the wetland = $L \times W \times d_W$ or Total volume of the biofilter
V_B'	Empty bed volume for flow in the biofilter
V_C	Volume of contact basin
V_d	Volume of dilutive water
V_{DE}	Volume of a dispersal event to a zone or Volume of the dose event
V_{DT}	Volume of the dosing tank
V_F	Volume of the trickling filter
V_i	Estimated settling velocity of the solids interface
VIP	Ventilated improved pit latrines
V_M	Volume of the tank with the membrane module in it or Volume of the membrane zone (or tank or basin)
V_{max}	Maximum settling velocity of the interface
V_N	Nominal volume
V_{RT}	Volume of a combined dosing/recirculation tank
VSB	Horizontal flow vegetated subsurface bed wetland
V_{SM}	Volume of filter media required
VSS	Volatile suspended solids
V_{ST}	Total tank volume
V_U	Water volume used per water use (e.g., 1 gal per urinal flush) or Volume used per event (e.g., 3 gal per toilet flush)
VVSB	Vertical flow vegetated subsurface bed wetland
V_W	Water volume equals the open porosity in the wetland media
V_{WE}	Effective water volume accounting for porosity and inactive zones
W	Width
WARMF	Watershed analysis risk management framework model
WAS	Waste activated sludge
WERF	Water Environment Research Foundation
WHO	World Health Organization
WI	Wisconsin
W_I	Width of an individual trench or narrow bed
WRA	Water Reuse Association
WRF	Water Reuse Foundation
WSS	Web Soil Survey
W_U	Width of the undisturbed land between each trench or bed or chamber
X	Concentration of VSS (or TSS)
X_A	Concentration of VSS (or TSS) in the aeration tank volume

X_E	Concentration of VSS (or TSS) (or cells) in the effluent or Effluent concentration of VSS
X_I	Influent concentration of VSS (or TSS)
X_M	VSS concentration in the tank with the membrane in it
X_R	Concentration of VSS (or TSS) in the recycle line
X_W	Concentration of VSS (or TSS) in the waste activated sludge
Y	Maximum yield coefficient
y	Fractional distance from the inlet to outlet
YN	Net solids production
Y_{Net}	Net solids production
Y_{obs}	Observed yield coefficient
α	Empirical parameter
β	Empirical parameter
Δ_{QA}	Savings due to water efficient fixtures and appliances
Δ_{QA-Hot}	Hot water savings (i.e., avoided use) due to MFFA
ε	Porosity of clean gravel in a VSB or plant-based void ratio in a FWS
θ	Temperature activity coefficient
μ	Specific growth rate
μ_m	Maximum specific growth rate
Ψ_u	Suction force due to capillary action of the soil pores

Appendix D

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