
DESIGN STANDARDS FOR

INTERMEDIATE - SIZED

WASTEWATER TREATMENT SYSTEMS

2012

NEW YORK STATE

DEPARTMENT OF ENVIRONMENTAL CONSERVATION

DIVISION OF WATER

BUREAU OF WATER PERMITS

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DRAFT

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Foreword

The Clean Water Act (CWA) was enacted to restore and maintain the chemical, physical, and biological integrity of the Nation's surface waters by reducing direct pollutant discharges into waterways, financing municipal wastewater treatment facilities, developing technology necessary to eliminate the discharge of pollutants, and managing polluted runoff.

In New York State, Article 17 of the Environmental Conservation Law (ECL) entitled "Water Pollution Control" was enacted to protect and maintain both surface and groundwater resources. Article 17 of the ECL authorized the creation of the State Pollutant Discharge Elimination System (SPDES) program to protect New York's water resources. Onsite Wastewater Treatment Systems (OWTS) may discharge to either surface water or groundwater.

Pursuant to Article 17 Title 7 of the Environmental Conservation Law all OWTS's, without the admixture of industrial or other waste, and having a groundwater discharge greater than 1,000 gallons per day or surface water discharge of any size must be covered by a State Pollutant Discharge Elimination System (SPDES) permit issued by the NYSDEC.

Design standards for large Publicly-Owned Treatment Works (POTWs) have been updated independently by The Great Lakes-Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers (GLUMRB) and the New England Interstate Water Pollution Control Commission (NEIWPCC). Design standards for small individual residential systems are updated by the NYSDOH, local county health departments, or watershed organizations. These Design Standards for Intermediate Sized Wastewater Treatment Systems (Design Standards) are being updated by NYSDEC to meet the needs not addressed by the large and small systems design standards (refer to Appendix A for the applicable technical standard). Over the last twenty years OWTS technologies have increased in the level of treatment provided and in complexity of design and operation. It is timely that the Design Standards are updated now to keep up with current needs by the wastewater treatment system design professionals.

Numerous references to the GLUMRB's "Recommended Standards for Wastewater Facilities," 2004 (Ten State Standards) and NEIWPCC's "Guides for the Design of Wastewater Treatment Works" - Technical Report-16, 2011 (TR-16), appear in this manual. The Ten State Standards has been adopted by DEC as

New York State's official standards for municipal wastewater treatment and collection facilities according to 6 NYCRR Part 750-2.10.g.1. The Ten State Standards may be viewed at: <http://10statesstandards.com>, and is available in print from Health Education Service, Inc., P.O. Box 7126, Albany, New York 12224, (518) 439-7286. Health Education Services (HES) may be accessed online at <http://www.hes.org>. TR-16 is also an acceptable design standard for municipal systems of any size, or any non-municipal, intermediate-sized systems that treat only sanitary wastewater. TR-16 addresses standards for alternative collection systems such as vacuum sewers, low pressure/grinder pump sewers, septic tank effluent (pump) and septic tank effluent (gravity) sewers (STEP/STEG).

New York State Department of Health (NYSDOH) regulations for residential wastewater treatment systems discharging less than 1,000 gpd, are entitled "Appendix 75-A, Wastewater Treatment Standards – Residential Onsite Systems." Design guidance for residential onsite systems are published under the title Residential Onsite Wastewater Treatment Systems Design Handbook. The NYSDOH Design Handbook is available from HES, Inc. (www.hes.org). Both documents are on the NYSDOH website.

Disclaimer

The use of this manual does not guarantee the proper function or performance of a treatment system. This manual also does not claim nor imply to address safety related issues associated with the design, construction/installation or testing of the components of the system, or abandonment of the system. The availability of a public sewer may result in the abandonment of some onsite wastewater treatment systems.

Introduction

With the increased development of rural and suburban areas, the need for wastewater treatment systems has grown. In many cases, the cost of a centralized municipal system may be prohibitive. Onsite Wastewater Treatment Systems (OWTSs) have been gaining recognition as viable wastewater management alternatives that can provide reliable service at a reasonable cost while still preserving environmental quality. In addition, acknowledgment of the impacts of OWTS discharges on both surface water and groundwater quality has increased interest in optimizing the design, construction, monitoring, operation and maintenance of the systems.

Purposes of these Design Standards:

This manual has been prepared to provide licensed professional engineers, owners, operators, and others with guidance on the design, operation and maintenance of intermediate-sized wastewater treatment facilities. The purposes of this manual are to protect the waters of the state, encourage system designers to include water conservation, energy-efficiency, and life-cycle cost evaluation in the design of facilities, and reduce project review and processing time by the appropriate regulatory agencies.

It is anticipated that the information in this manual will aid designers in the preparation of complete project submissions (maps, plans and reports) which will result in wastewater treatment systems that meet these purposes. These Design Standards offer:

1. Prescriptive and performance-based design standards for systems serving private, commercial and institutional (P/C/I) facilities with discharges to Soil Treatment Systems (STS).
2. Prescriptive and performance-based design standards for municipal systems using alternative collection systems or discharging to Soil Treatment Systems (STS).
3. Prescriptive and performance-based design standards for surface water discharges.
4. Select, preferred references for additional design guidance.

How to use these Design Standards:

This manual provides engineering design standards for a variety of wastewater treatment systems that may be used to treat sanitary sewage generated at private, institutional, commercial and multi-home facilities (without the admixture of industrial or other waste [6 NYCRR Part 750 definitions]). These systems may discharge to surface water, or to groundwater through a subsurface soil absorption system. A septic tank followed by soil absorption, in many cases, provides the most cost-effective, long-term solution for wastewater treatment. Proper design, operation, and maintenance of these systems are essential to their viability. Secondary and tertiary treatment options are also included because the physical characteristics of the site or chemical composition of the wastewater may be such that a more complex system is necessary to meet public health and environmental criteria, and discharge limitations. The manual is organized in 9 sections as follows:

Section A. Facility Planning and Permitting

This section provides a short description of the process for planning, locating and designing a wastewater treatment system according to the state's SPDES and SEQR processes. Other related permits that may be required are also addressed.

Section B. Project Evaluation and Design Criteria

This section discusses site evaluation criteria and soil evaluation requirements, flood protection requirements, wastewater characterization, and design flows based on the type of establishments served (residential, commercial, institutional, recreational, food service or other private business). Non-contaminated flow diversion and other treatment considerations are briefly discussed and typical effluent limits for surface water discharges are given.

Section C. Sewer Systems and Sewage Pump (Lift) Stations

This section describes requirements for building sewers referencing the state building and plumbing code. Both conventional sewers and alternative collection systems are covered. Additional info is given for the construction of conventional gravity sewers, manholes, pipe material, and dry wells and wet wells comprising pump or lift stations.

Effluent, vacuum and pressure sewer details and criteria are provided from Water Environment

Federation “Alternative Sewer Systems,” Manual of Practice FD-12, including the use of clean-outs and infrequent use of manholes.

Section D. Preliminary and Primary Treatment, Flow Measurement and Appurtenances

This section provides information on components that precede secondary treatment units like septic tanks, effluent filters, grease interceptors, distribution boxes and flow-splitters, discharge to a Soil-based Treatment System (STS), or serve as alternatives to conventional onsite treatment. Flow equalization recommendations are given as an alternative means of providing additional storage volume. Flow measurement requirements are given based on three ranges of facility design flow.

Section E. Subsurface Treatment and Discharge

This section addresses the application of pretreated wastewater to the soil. Application rates are given in Table E.1. Options for distribution networks are discussed, and design methods and calculations are given for pressure distribution and dosing systems.

Section F. Secondary Treatment

This section is divided into fixed film (attached growth) and suspended growth sections. For *fixed growth* systems, media filtration includes sand filters, as well as fabric, gravel, peat and other materials. The design guidance for Rotating Biological Contactors (RBCs) has also been updated.

For *suspended growth* systems, the design standards provide activated sludge design criteria for extended aeration and contact stabilization modes and Sequencing Batch Reactor designs. Oxidation ditch, and lagoon or pond systems design guidance is also given.

Section G. Tertiary Treatment

This section is divided into four subsections: granular media filtration, physical-chemical treatment systems, biological nutrient removal systems, and constructed wetland systems. The Ten States Standards

tertiary treatment technology selection is limited to phosphorus removal by chemical treatment and high-rate effluent filtration, so TR-16 Chapter 7 on advanced treatment has been cited for the majority of this section. Many of the tertiary treatment processes can provide sufficient pollutant removal on their own; others may require a certain level of pretreatment or post-treatment (filtration, clarification, or disinfection) to meet the effluent limitation. Constructed wetlands are included as treatment units that can provide additional pollutant removals when receiving secondary effluent.

Section H. Innovative Systems and Variances from these Design Standards

This section is the Department's procedure for new technology review. It provides a way for a design engineer to propose a proprietary technology to a Regional reviewing engineer by demonstrating how it meets or exceeds the design criteria of a similar design in these Design Standards. New or innovative designs, not included in these Design Standards, may also follow this procedure to gain site-specific approval.

Section I. Surface Water Discharges: Disinfection and Reoxygenation

This section is divided into two subsections:

The disinfection section has design guidance for chlorination and dechlorination. Updated information on ultraviolet disinfection and references for more additional UV design criteria are provided.

The effluent reoxygenation section was renamed to accommodate the terminology of the Water Environment Federation Manual of Practice (MOP) Number 8, "Design of Municipal Wastewater Treatment Plants," and includes diffused/mechanical and cascade aeration, outfall specifications, and controlled release requirements.

Section J. Operation, Maintenance, and Control

This section has information on systems requiring a certified operator based on 6 NYCRR Part 650, and a Residuals subsection based on 6 NYCRR Part 360. The Operation and Effluent Control, and the Emergency Repair and Rehabilitation subsections refer to 6 NYCRR Part 750. The Remote Telemetry,

Instrumentation and Alarms section includes integral components of systems with technologies that need to be remotely monitored by the manufacturer, trained service providers, or an operator. The System Additives subsection includes precautionary information from EPA on several categories of additives.

Appendix A Wastewater Treatment System Regulatory Framework in New York State

This appendix is a table that provides the distribution of responsibility between DEC and DOH based on the 1984 MOU, and according to system size and type (residential, private, commercial and institutional).

Appendix B Conversion Factors

This appendix is a table of conversion factors useful for the design of systems in English or S.I. units.

Appendix C Sewer and Manhole Leakage Tests

This appendix provides testing standards and criteria from the Ten State Standards, ASTM and AWWA.

Appendix D Septic Tank Watertightness Testing

This appendix is based on the Consortium of Institutes for Decentralized Wastewater Treatment (CIDWT) Installation of Wastewater Treatment Systems, Appendix G (the National Review Version 1, 2008).

Appendix E Recommended Guidance for the Design of Wastewater Drip Dispersal Systems

This appendix provides installation, operation, monitoring, and maintenance recommendations, and a list of nine references for Drip Dispersal systems.

A. Facility Planning And Permitting

A. 1 Introduction

This section describes the planning and permitting process components including which agency has jurisdiction, permit or management requirements depending on the discharge type (surface or subsurface), permit type (individual or general), proximity to environmentally sensitive areas or public water supply watersheds, and possible additional requirements or permits. Wastewater collection systems, wastewater treatment facilities and soil-based treatment systems should also be designed in a context of asset management. In-time, replacement, modification or expansion of POTW facilities occur as communities grow and the facility ages. Asset management is detailed below in Section A.5.

A. 2. Planning & Permit Application Process

Construction of a new or modified wastewater treatment system will require regulatory authorization. (See Appendix A “Wastewater Treatment System Regulatory Framework Table”.) The first step in the process should be to contact the Regional Permit Administrator to arrange a pre-application conference. Typically, the initial consultation with DEC should be with the Division of Environmental Permits’ Regional Office serving the county where the project is proposed. Applicants find this meeting to be a helpful forum to explain a proposed project to the DEC and other interested agencies. Preliminary answers to questions about project plans, and permit application procedures are typically provided to the applicant at this conference.

A complete submission for a project should include SEQR documentation (see Section A.4), an engineering or wastewater facilities report including detailed engineering plans that demonstrate compliance with applicable design standards, a SPDES permit application (see A.4 below), a location map (1:24,000 scale topographic), and a site plan showing existing structures, roads, water courses, and other pertinent features. While developing the site plan, among other references, the applicant should consult the DEC’s Interactive Online Maps, especially the Environmental Resource Mapper, to avoid sensitive or protected resources. These interactive online maps can be found at: <http://www.dec.ny.gov/pubs/42937.html>. Also, garbage grinder use should be evaluated in the design phase of the project and accounted for in tank sizing per Section D.6.

Transportation Corporations and Sewage Disposal Corporations

Where there is a potential for a facility to be used in the future as part of a sewage works corporation (formed pursuant to Article 10 of the Transportation Corporations Law) or a municipal system (Publicly Owned Treatment Works, POTW), New York State's standards for municipal wastewater treatment and collection facilities (Ten State Standards or TR-16) should be consulted. Note that the terms *Sewage Disposal Corporation* (referenced in 750-1.6(f)) and *Sewage Works Corporation* (referenced in Article 10 of Transportation Corporation Law) are considered equivalent in meaning. DEC Regions may review any wastewater treatment systems proposed to be operated by a Transportation Corporation (also see Section C.10).

Discharge Limits

In certain instances for proposed discharges to surface waters, the project information is analyzed by DEC for waste-assimilation capacity, for allocating water-quality-based effluent limits, and/or the establishment of final effluent limitations for the facility. Before submitting an application proposing surface discharge the applicant should request a pre-application conference as discussed earlier in this subsection. Appropriate (draft) discharge limits (subject to public comment) will be discussed with DEC staff at the conference.

Underground Injection Control (UIC)

In addition to existing state and local oversight, decentralized wastewater treatment systems that discharge subsurface and serve more than 20 people may be subject to regulation under the USEPA Underground Injection Control (UIC) Program. The Safe Drinking Water Act (SDWA) authorizes the USEPA to establish minimum federal requirements for state and tribal UIC Programs to protect underground sources of drinking water from contamination caused by injection activities. Protection includes the oversight of construction, operation, and closure of injection wells. Groundwater discharges that may require a SPDES or UIC permit as a Class V well include stormwater infiltration basins (a.k.a. stormwater drainage wells), large-capacity septic systems, laundromats without dry cleaning facilities, food processing disposal, aquaculture, and others.

In most cases, if a SPDES permit is issued by NYSDEC for a Class V well, EPA will not require a UIC

permit to also be issued (the discharge is considered to be covered “by rule”).

Public Water Supply Watersheds

Where a proposed wastewater treatment system is to be located in the watersheds of public water supplies, the rules and regulations enacted by the New York State Department of Health for the protection of those water supplies must be followed. **Information on source water protection, including local municipal and Watershed Protection Agency rules and regulations, can be accessed at the NYSDOH website.** Where such wastewater treatment systems are to be located in the watershed of any stream or body of water from which the City of New York obtains its water supply, the approval of New York City’s Department of Environmental Protection (NYC DEP) must be obtained. **Information on the City’s water supply watershed protection program can be accessed at the NYC DEP website.**

A. 3. Obtaining a SPDES Permit for a Wastewater Discharge

DEC may issue an individual (site specific) SPDES permit or an applicant may apply for permit coverage under the SPDES Private, Commercial, and Institutional (PCI) General Permit (SPDES General Permit) for existing and proposed discharges. The SPDES PCI General Permit authorizes discharges to groundwater between 1,000 and 10,000 gallons per day (gpd) of treated sanitary wastes only, without the admixture of industrial waste, from onsite wastewater treatment systems (OWTS) serving private multi-family dwellings, or other private, commercial, or institutional facilities. See Appendix A for subsurface discharges under 1,000 gpd, over 10,000 gpd, and for surface discharges, or non-sanitary discharges of any quantity.

Title 6 of the New York Code of Rules and Regulations (6 NYCRR)

The regulations requiring SPDES permits is detailed in Title 6 of the Official Compilation of Codes, Rules and Regulations of the State of New York, Part 750 (6 NYCRR Part 750), and can be accessed at the DEC public website: <http://www.dec.ny.gov>.

Subsurface discharges less than 1,000 GPD of sanitary waste, without the admixture of industrial wastes, do not require a SPDES permit from NYSDEC.

Applications and requirements for an individual SPDES permit (Form “D”) are discussed in 6 NYCRR,

Part 750 1.6. A form “D” application can be accessed at <http://www.dec.ny.gov/permits/6222.html>.

Modification of SPDES Permits is discussed in 6 NYCRR, Part 750-1.18 and can be accessed at <http://www.dec.ny.gov/regs/4585.html#16194>.

Applications and requirements for a SPDES PCI General Permit are discussed in 6 NYCRR, Part 750-1.21. An application form for a SPDES PCI General Permit can be accessed at <http://www.dec.ny.gov/permits/6222.html>.

Operating in accordance with a SPDES Permit is detailed in 6 NYCRR Part 750-2, and can be accessed at <http://www.dec.ny.gov/regs/4584.html>.

6NYCRR Part 750-2.10 states in part that “...prior to construction...the permittee shall submit to the Regional Water Engineer an approvable engineering report, plans, and specifications that have been prepared by a person or firm licensed to practice professional engineering in the State of New York in accordance with [design] standards accepted by the department.”

6NYCRR Part 750-2.10 (g) states that “The following standards are accepted by the department:

- (1) Ten States Standards (see section 750-1.24) for use in designing POTWs and POTW collection systems;
- (2) Intermediate Design Standards (see section 750-1.24) for use in designing facilities that are not POTWs, which treat only sanitary sewage; and
- (3) Other standards that are acceptable by the department.”

The Ten States Standards does not include subsurface discharge systems. DEC policy and practice is to use the Intermediate Design Standards for the soil-based treatment systems (STS) used by POTWs that have subsurface discharges. Other standards acceptable to DEC include TR-16, Water Environment Federation Manuals of Engineering Practice (MOP), or other standards identified by the reviewing engineer.

Construction Inspection and Certification

Construction inspection and certification is also part of the SPDES rules and regulation under 6 NYCRR §750-2.10. For individually permitted systems, requirements are given in subdivision (c), and for facilities

eligible for coverage under the SPDES General Permit the requirements are in subdivision (f).

Construction inspection under either permit must be conducted by a licensed professional engineer who supervised the construction and shall include inspections during the course of construction at critical installation points to insure that the treatment system is installed as specified in the design plans.

For designs submitted under an individual SPDES permit, the design engineer must certify to DEC that the treatment system is constructed in accordance with the plans and specifications that had previously been approved by DEC, and provide with that certification as-built drawings reflecting changes to the approved plans. Any modifications from the approved design require approval by DEC prior to construction.

For designs submitted under the SPDES General Permit where there is no County Health Department, the design engineer must certify to DEC that the treatment system is constructed in accordance with these Design Standards. The permittee should retain a set of as-built plans provided by the design engineer for the treatment system as constructed for future access by DEC.

For designs submitted under the SPDES General Permit where there is a County Health Department, the permittee must certify to DEC that the Health Department has approved the treatment system plans and specifications submitted by the design engineer as having been constructed in accordance with these Design Standards. The permittee should retain a set of as-built plans provided by the design engineer for the treatment system as constructed for future access by DEC.

Upon receipt of the certification by the design engineer or permittee, DEC will issue a letter of acceptance acknowledging their receipt of the certification. It is at this point that the permittee may begin discharging.

Closing a Facility

Closure requirements for wastewater treatment facilities are specified in 6 NYCRR, Part 750-2.11, and can be accessed at <http://www.dec.ny.gov/regs/4584.html#16221>.

The SPDES General Permit application form is available for download on the DEC Public Website. Plans submitted with a SPDES General Permit must be certified by a licensed professional engineer or the permittee/health department (plans and specifications are not approved by DEC).

The SPDES General Permit is not applicable for facilities located in the counties of Kings, Nassau, Suffolk and Queens that were not previously authorized in accordance with GP 95-01, nor in the following environmentally sensitive areas:

- special (100-year) flood hazard area as defined in 42 United States Code 4001;
- freshwater wetlands or adjacent area as defined in Environmental Conservation Law (ECL) Article 24;
- tidal wetlands and adjacent area as defined in ECL Article 25;
- coastal erosion hazard area as defined in ECL Article 34
- wild, scenic, and recreational river corridors as defined in ECL Title 27, Article 15.

Facilities located in these areas require project-specific review and must apply for an individual SPDES permit. Pursuant to 6 NYCRR Part 750-2.10, engineering plans and specs ***shall*** be approved by DEC before construction can begin. If you have questions regarding your facility location in these environmentally sensitive areas, please contact your DEC Regional Permit Administrator.

Facilities applying for subsurface discharges under the SPDES General Permit also need approval from the appropriate city health department, county health department, or district office of the New York State Department of Health before a system -cesspool, septic system, etc. - can be built. Other DEC permits or approvals from other agencies may also be required.

A. 4 State Environmental Quality Review (SEQR)

All SEQR issues must be satisfactorily addressed prior to issuance of a SPDES permit. ***Generally, approval of the plans and specifications for the proposed OWTS cannot be granted prior to issuance of the SPDES permit.*** Construction of the proposed facilities will be governed by the conditions of the permit and the approval letter for the project.

The provisions of the Uniform Procedures Act (UPA) require that applications for DEC permits be considered incomplete unless certain requirements of SEQR have been met. This initially involves the applicant filing a completed Environmental Assessment Form (EAF) and other DEC permits in accordance with the portions of the law. Please see <http://www.dec.ny.gov/permits/363.html>

Other permits issued by federal or local agencies may also be required. Contact the DEC Division of Environmental Permits, Regional Office serving the county where the project is proposed for the best approach to meeting these permit requirements.

A step-by-step discussion of the SEQR process is presented in “The SEQR Cookbook”. The Cookbook and the SEQR Handbook (and Q & A about SEQR) can be found at <http://www.dec.ny.gov/permits/57228.html>. For further information on SEQR, UPA timetables and permits, contact the Division of Environmental Permits’ Regional Office serving the county where the project is proposed.

A. 5. Asset Management

An "asset" is a component of a facility with an independent physical and functional identity and age (e.g., pump, motor, sedimentation tank, main buildings and staff). Asset management is a process for maintaining the assets of wastewater collection and treatment facilities at the desired level of service to meet user needs, public safety, environmental protection, and permit requirements at the lowest life cycle cost. Lowest life cycle cost refers to the optimum cost for rehabilitating, repairing or replacing an asset.

Energy efficient system components contribute to a lower life-cycle cost. The New York State Energy Research and Development Authority (NYSERDA) may be contacted for energy-efficiency recommendations and technical assistance at <http://nyserda.ny.gov>. Their Water and Wastewater Energy Management Best Practices Handbook contains a section with twenty-one management practices for wastewater treatment plants.

Asset management is implemented through an asset management program and typically includes a written asset management plan. Asset management is applicable to centralized, decentralized and onsite wastewater collection and treatment systems.

These Design Standards determine how the system or component could or should be designed. Asset management needs are determined by the type of system, or system component to be designed. Optimally, design choices consider the technical and managerial capacity of the owner/operator to operate and maintain the facility. Asset management is a process to care for the systems, and to keep them operating at or exceeding the intended level of service.

An asset management plan is a collection of best management practices that can guide an agency's or municipality's investments throughout each stage of an asset's life cycle: planning, acquisition, operation, maintenance, renewal, and ultimately, decommissioning. Although not required by DEC under 6 NYCRR Part 750-2, developing an asset management plan provides the foundation for maintaining the performance, and a strategy for the long-term health, of a facility.

Typical components of a basic asset management plan include:

1. Taking inventory to identify the current performance of assets. This first step involves identifying, locating, and evaluating the system's assets. It includes reporting each asset's condition and remaining useful life.
2. Prioritizing assets is done by determining assets that are critical to current operational performance. The asset's 'criticality' to the rest of the system is an important factor in maintaining the required level of service that the wastewater collection and treatment system must provide. Level of service includes the protection of public health, and meeting SPDES permit limits. Assets critical to providing these should be given a higher priority.
3. Developing an Asset Management Plan. After ranking priorities, permittees and managers need to plan for future rehabilitation and/or replacement of assets. This phase formulates a capital improvement plan. A fiscal strategy should be developed to set money aside each year to fund a capital improvement reserve. Facility operations, maintenance, and capital investment strategies must be integrated and coordinated to sustain performance at the lowest total cost of ownership (life-cycle cost).
4. Implementing the plan. This step requires a detailed budget and financial forecasting of revenues typically on a five (5) year basis/cycle through the design life of the system components. It may involve finding additional funding sources and/or increasing customer rates.
5. Reviewing and revising the plan. The plan should be reviewed each year and updated.

B. Project Evaluation

B.1. Introduction

To design an effective, environmentally acceptable on-site wastewater treatment system, it is necessary to evaluate the physical characteristics of a site to determine if adequate conditions exist to treat and discharge wastewater on a long-term basis. A comprehensive site evaluation provides the necessary information to select an appropriately sized, cost-effective treatment system from a wide range of design options.

B.2. Site Evaluation

Points to be considered when evaluating a site for the location of an on-site wastewater treatment system include the identification of flood prone areas; the proximity of dwellings (both permanently and periodically inhabited) and other structures; the location of nearby utilities including water supplies and water lines; proximity to surface waters, wetlands and other environmentally sensitive areas; terrain and other surface characteristics; subsurface conditions; prevailing winds and odor control; area for system replacement and/or expansion; and the assimilative capacity of any potential receiving stream. Existing easements and rights-of-way must also be considered. Site characteristics may preclude the use of one type of wastewater treatment system in favor of another. Local temperature extremes may make it necessary to enclose the facilities to maintain treatment efficiency.

For all systems having greater than 1,000 gpd design flow, a separate area equal in size (100%) to the proposed subsurface soil absorption system must be reserved for replacement purposes due to failure. For all systems having design flows less than 1,000 gpd, a 50% reserve area is required. The site evaluation should also include consideration of future expansion, in addition to the replacement area.

B.3. Separation Distances

Airborne Separation

Wastewater treatment systems with open-air, odor-producing units should be located as far as possible from human habitation, or public use or gathering areas. Table B.1 contains a list of minimum separation distances that must be maintained between treatment facilities and dwellings or property lines to provide some attenuation of airborne nuisances such as aerosols, pathogens, odors and noise. Special designs or

considerations may warrant a reduced distance. Buffer areas should be permanent, with the intent to preserve the future use and enjoyment of adjacent properties, and not subject to later development that can create nuisance conditions.

The operation of various unit processes is known to be adversely affected by wind and temperature extremes. Possible effects should be analyzed and prevented or controlled by appropriate design considerations (e.g. wind-breaks, high freeboard, equipment or process enclosure, etc). If enclosures are used to preserve treatment efficiency in the colder portions of the state, care should be taken to protect units from condensation and subsequent corrosion or operational problems. Eliminating the insulating snow-pack using an unheated pole-barn with no walls in northern New York has resulted in the freezing of sand filters.

If there is concern that the separation provided is not sufficient to avoid odorous nuisance conditions, steps should be taken to minimize odors. Such steps include chemical addition, consideration of prevailing winds, odor control systems, and covering or enclosing the facilities. Sludge processing, storage, and disposal units are not included in Table B.1; for intermediate-sized facilities it is assumed solids are hauled off-site. If these facilities are proposed to be on-site, the Ten State Standards (Section 11.28.c.) should be referred to for odor control and location criteria/requirements; and NYSDOH codes for separation distances (see footnotes of Table B.2). Approval of a wastewater treatment system site location and design should not be interpreted as relieving the owner from responsibility for remedial action should objectionable conditions arise in the future.

Table B.1 Minimum Aerial Separation Distance (in feet) from Treatment Facility

Treatment Type	Radial Distance to Existing Downwind Dwellings (On or Off the Property)	Distance to Downwind Property Line from Treatment Unit
Wastewater Treatment Processes Open to the Atmosphere e.g. Open Sand Filter, and Oxidation Ditches.	400	350
Wastewater Treatment Processes Enclosed ¹ in a		

Building, and Buried or Covered Sand Filters	200 ²	150
Facultative and Aerated Lagoons	1000	800
Effluent Recharge Bed	750	550

¹ Enclosed building design requires consideration of ventilation, safety, operation and maintenance access, and odor and noise control devices. See Ten State Standards and TR-16. Mechanical forced air ventilation may increase the required separation distance for odors.

² Non-residential structures located on the same parcel may qualify for lesser distances.

Groundwater Separation

Table B.2 provides guidance for the minimum separation distances that should be met for subsurface soil-based treatment and dispersal systems to protect water supply facilities, and to avoid sewage contamination and nuisance conditions. Factors such as system elevation, ground slope, and direction of groundwater flow, well-pumping rates, and existence of impervious barriers will affect the necessary separation distances. Increased separation distances may be required if warranted by local conditions. Decreased separation distances shall not be allowed unless sufficient proof is provided by the design engineer to show local conditions will prevent contamination at a closer distance. Such a justification will be judged on a case-by-case basis by the regulatory agency responsible for plan review. Separation distances to wells are also given in the Public Health Law in Section 206(18), Appendices 5-B Table 1 and 5-D Table 1. Separation distance between water supply and sewer lines is given in footnote b of Table B.2 and discussed further in Section C.8.

Table B.2 Minimum Horizontal Separation Distance (in feet) between:

Existing Feature Below and	Watertight Septic Tank	Sewer Line	Absorption Field or Unlined Sand Filter (Including Replacement Area)
Drilled Well – Public (e)	100	50 (a)	200
Drilled Well – Private (e)	50	50 (a)	100/150 *
Dug Well / Spring (f)	75	50 (a)	150
Water Line (Pressure)	10	10 (b)	10
Water Line (Suction)	50	50 (a)	100/150 *
Foundation	10	--	20
Surface Water (h)	50	25	100
Open Drainage (h)	25	25	50 (c)
Culvert (Tight Pipe)	25	10	35
Culvert Opening	25	25	50
Catch Basin	25	--	50
Interceptor Drain	25	25	50 (c)
Swimming Pool – In Ground	20	10	35/50 *
Reservoir (water supply)	50(d)	50(a)	100 (d)
Property Line	10	10	10
Top of Embankment	25	25	50
Wetland (DEC) (g)	100	100	100
Stormwater Management Practice (h)	50	25	100

*These three greater separation distances are for seepage pits.

Note: For certain graywater systems discharging to shallow soil, generally use 50% of the separation distance required for absorption fields – refer to details and local exceptions in Section D.12.

These systems are not allowed for residential facilities.

- a) If the separation distance of at least 50' between a drinking water well and a sewer line cannot be met, the sewer line must be constructed of ductile iron pipe (DIP) or double-sleeved piping.

Either piping option must be installed below frost lines and be constructed of watertight joints. If DIP is used, the pipe can be no closer than 25 feet to the well and should be connected from structure to structure (e.g., manhole to manhole, manhole to pumping station, etc.). If double-sleeved piping is used, the pipe can be no closer than 25' to the well and the double sleeve must extend to points at least 100 feet from the well.

- b) Water (pressure) and sewer lines may be in the same trench if water line is placed on an undisturbed bench or shelf so that the bottom of the water main is at least 18 inches higher than the top of the sewer and the sewer is not subject to settling, vibration, superimposed loads, or frost action.
- c) May be reduced to 35' if the bottom of the drain is above the finished grade of the subsurface soil treatment system, keeping the drain water and wastewater separate.
- d) For a public water supply reservoir, 100 feet to septic tank and 200 feet to absorption field or seepage pit is required (See local watershed rules and regulations for possible superseding specifications, e.g. NYCDEP has 250' separation between reservoirs or reservoir stems and soil-based wastewater treatment systems).
- e) When wastewater treatment systems are located upgradient and in the direct path of surface water drainage to a well, the closest part of the treatment system shall be at least 200 feet away from the well. The listed water well separation distances from contaminant sources shall be increased by 50% whenever aquifer water enters the water well at less than 50-feet below grade. If a 50% increase cannot be achieved, then the greatest possible increase in separation distance shall be provided with such additional measures as needed to prevent contamination.
- f) See NYS DOH Individual Water Supply Wells - Fact Sheet #5: Susceptible Water Sources (Well Points, Dug Wells, Springs, and Shore Wells), August 2008.
- g) A reduced separation distance, if any, will be determined through the permit review process.
- h) Use Surface Water separation distances for stormwater ponds and wetlands. Use Open Drainage separation distances for infiltration systems, filtering systems (open), or open channels.

B.4. Soil Evaluation for Subsurface Discharge

Prior to performing any subsurface soil investigation, New York State law requires that the design engineer or contractor contact DIG SAFELY NEW YORK or other underground locating service provider.

Soil profile observations must be made on all sites proposed for soil absorption systems. A preliminary soil evaluation, and site investigation shall be done prior to design and installation of a treatment system on any property.

B.4. a. Deep Soil Pit Testing

A soil evaluation must be performed for subsurface absorption systems to a ***depth of at least five feet below*** the bottom of the proposed system.

A primary objective of the deep soil pit test is to identify mottling, or other means of determining the seasonally high groundwater table, to ensure adequate vertical separation (depth) to bedrock, impervious layer and hence the type of system to use. Therefore, soil pits must be dug to allow accurate description of soil types and horizons, while soil borings may be used to determine soil variability over a large area. Multiple pits may be necessary if the evaluation finds several different soil conditions. Soil pits should generally be as deep as the backhoe can excavate, wide enough to allow natural sunlight to shine directly on the exposed face of the pit wall, and, if at all possible, should be dug at the perimeter of the expected soil absorption area.

The seasonally high groundwater level shall be at least two feet below the trench bottom of a conventional absorption field, and at least three feet below the bottom of a seepage pit. The reviewing engineer may require greater depths (vertical separation) in rapidly permeable soils to ensure that the necessary treatment is provided. If these distances cannot be met, the use of a fill or mound system should be considered before a surface discharge is allowed.

Deep Soil Pit Entry

Before entering the pit, make sure that it is safe to enter and that all safety regulations are being met. The pit should be constructed properly with a step-type configuration to allow safe entry and exit. The pit should have no sidewall slumps or any other indications for a potential cave-in. Be sure that no heavy equipment or large objects (such as rocks or boulders) are resting on the surface immediately adjacent to the pit sidewalls. Excavations should be fenced or backfilled to avoid falls or unauthorized entry.

Variation in required depth or manner of observation may be allowed when a deep seepage pit is proposed to avoid unsafe conditions that might violate OSHA safety standards.

Some factors to be evaluated for all soil absorption systems are:

- a) Thickness of layers or horizons.
- b) Texture (USDA), consistence, and structure of soil layers.
- c) General color and colored mottling (i.e. an indication of seasonally high groundwater table). This should be done using Munsell color charts in natural light only
- d) Depth to water (if observed) and depth to estimated or observed seasonally high groundwater level.
- e) Depth to and type of bedrock (pervious or impervious) if observed.
- f) Other prominent features such as visible pores, stoniness, roots, or animal traces.

Soil evaluation should be based on finished elevations of the site, and should consider proposed site modifications that could affect subsurface conditions such as cutting and filling, storm water infiltration practices installed, pre-construction soil stock-piling, post-construction topsoil placement/depth. Proposed cut or fill work that is to be done must be accounted for when determining the suitability of the site for soil absorption systems. This does not include fill work necessary to construct a fill or mound system.

The first preference for *conventional absorption trench or bed systems* is to install trenches/beds in native soil. Protecting the site from compaction should be a high priority: flag, or otherwise designate, the absorption field so it is not used as a staging area, parking lot, or pathway for heavy equipment.

Groundwater mounding may occur under an absorption system in the presence of an impervious layer or over a zone of saturation. The potential for groundwater mounding should be investigated during the site evaluation. Operational problems and groundwater contamination may result if the groundwater approaches the base of the system. Special consideration should be given to sites that are incorporating storm water infiltration practices.

A mathematical analysis should be performed to predict the extent of groundwater mounding that would occur when the absorption system is in operation. Depending on system size and site conditions, the reviewing engineer may require a site/system-specific hydro-geologic evaluation by a hydro-geologist.

In general, greater recharge from the absorption system, wider application widths, and slower horizontal saturated conductivities will result in the formation of higher groundwater mounds. An aerated zone of two feet for an absorption field and three feet for seepage pits is desirable between the base of the system and the top of the groundwater mound.

B.4. b. Percolation Testing

Hydraulic conductivity can be estimated using soil percolation tests. Tests should be run during spring months, as system failure is more likely during wet months. If not done in spring, the following additional soil evaluation information shall be submitted to the appropriate jurisdiction (see Appendix A):

- Description of percolation test procedure
- Depth of percolation hole(s) - (must be based on the finish grade elevation of the area)
- Record of thickness of soil horizons, soil types, texture (USDA), consistence, and color
- Elevation/depth of the seasonally high groundwater level, or record of colored mottling
- Elevation/depth of soils to bedrock or impervious strata, and other prominent features such as visible pores, stoniness, roots, or animal traces.
- Number of percolation test holes dug
- Percolation Rate (minutes/inch) – stabilized
- Sewage application rate (gal/day/sq.ft.)
- Deep soil test pit and percolation hole locations shown on site plan

For a *conventional trench or bed system*, the percolation test should be performed at the depth of the proposed system based on the proposed finished grade elevation of the site. Percolation tests should be run in an area immediately adjacent to, or in between, the areas proposed for absorption trenches.

At least two percolation tests for every 1000 sq. ft. of absorption area shall be performed in holes spaced uniformly throughout the site. If the soil conditions are highly variable, more tests may be required. For larger systems, i.e. greater than 5000 gpd, where soil maps and confirming observational data indicate uniform soils over a large area, fewer percolation tests may be approved by the DEC Regional Water Engineer, or by the regulatory agency responsible for plan review.

If a seepage pit is under consideration, percolation tests should be done at least at one-half depth and at full depth of the seepage pit. If different soil layers are encountered when digging the test hole for a seepage pit, a percolation test should be performed in each layer with the overall percolation rate being the weighted average of the test results based upon the depth of each layer. Test pit soil layers (for seepage pits) with percolation rates slower than 30 minutes per inch (mpi) must be excluded from these calculations.

For mound systems the percolation tests must be run just within the estimated boundary of the basal area of the mound. Percolation tests should be performed at a depth of 20 inches in slowly permeable soil, 12 inches in shallow soil over pervious bedrock, and 16 inches if the high water table is within 20 inches of the ground surface.

The procedure noted below should be followed when performing a percolation test:

- 1) Dig a hole with vertical sides having a diameter of approximately 12 inches, or 12 inches square. Scrape or scarify sides and remove loose soil from the bottom of the hole. For seepage pits, a larger excavation should be made for the upper portion of the hole with the actual test hole in the bottom (see Figure B.1).
- 2) Install measuring stick. Place two inches of $\frac{1}{2}$ - to $\frac{3}{4}$ -inch washed gravel in the hole to protect the bottom from scouring action when water is added.
- 3) Presoak: Fill the test hole with water and allow it to completely seep away. This pre-soaking should be done continuously for **at least 4 hours before the test**. Soils with high clay content should be **pre-soaked overnight**. In sandy soils, soaking is not necessary. Instead, after filling the hole twice with 12 inches of water, if the water seeps away completely in less than ten minutes, the test can proceed immediately. After the water has seeped away, remove any loose soil that has fallen from the sides of the hole.

- 4) Pour clean water into the hole, with as little splashing as possible, to a depth of six inches.
- 5) Observe the time in minutes required for the water to drop one inch (from the six-inch to the five-inch mark) and record the results on a Percolation Test Data sheet.
- 6) Repeat the test a minimum of three times until the time for the water to drop one inch for two successive tests is approximately equal. The last seepage rate measured (minutes per inch) will then be taken as the stabilized rate for percolation. If different perc test results are obtained from separate soil pits in the same general area, the slowest percolation rate is used in design.

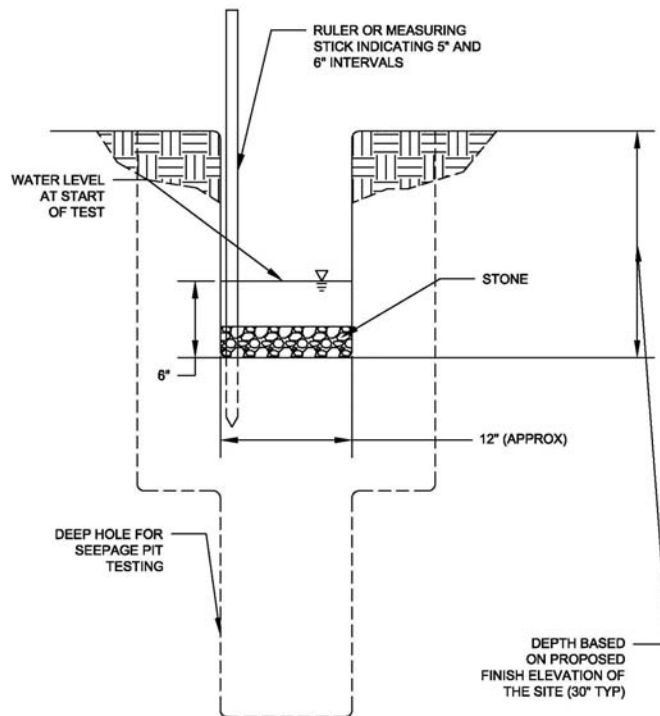


Figure B.1. Soil Percolation Test Arrangement

NOTE: A percolation test whose results are inconsistent with the deep test pit soil evaluation shall be disregarded, and the percolation test(s) shall be performed again.

B.5. Locating Facility relative to Flood Plains

Most communities in New York participate in the National Flood Insurance Program. As such, they have local laws that are at least as restrictive as federal regulations governing floodplain development found in 44 CFR 60.3. Any project that encroaches into a floodplain, as shown on federal Flood Insurance Rate Maps, requires a permit from the local community.

Pertinent sections of 44 CFR 60.3:

60.3(a)(3) “new construction and substantial improvements shall...be constructed with electrical, heating, ventilation, plumbing and air conditioning equipment and other service facilities that are designed and/or located so as to prevent water from entering or accumulating within the components during conditions of flooding.”

60.3(a)(4) “if a subdivision proposal or other proposed new development is in a flood-prone area (i.e., within the 100-year flood delineation), any such proposals shall be reviewed to assure that ... (ii) all public utilities and facilities, such as sewer, gas, electrical and water systems are located and constructed to minimize or eliminate flood damage...”

60.3(a)(6) (the local administrator shall) ...“Require within flood-prone areas (i) new and replacement sanitary sewage systems to be designed to minimize or eliminate infiltration of flood-waters into the systems and discharges from the systems into flood waters and (ii) onsite waste disposal systems to be located to avoid impairment to them or contamination from them during flooding.”

The following precautions, found in most community flood damage prevention local laws, are recommended for any installation located within a 100-year floodplain (identified as Zones A1-A30, AE, AH, A and also Zones V1-V30, VE and Zone V on the federal Flood Insurance Rate Map) to meet the requirements of 44 CFR 60.3.

If base flood¹ (100-year flood) elevation data are available, new construction and substantial improvements of any non-residential structure, together with attendant utility and sanitary facilities, shall:

- 1) Be anchored to prevent flotation, collapse, or lateral movement during the base flood. This

¹ 44CFR 59.1, "Base flood means the flood having a one percent chance of being equaled or exceeded in any given year." i.e. commonly called “the 100-year flood”.

requirement is in addition to applicable State and local anchoring requirements for resisting wind forces.

- 2) Have the lowest floor, including basement or cellar, elevated to or above two feet above the base flood elevation.
- 3) Be flood-proofed so that the structure is watertight below two feet above the base flood elevation, including all manholes, access ports, lift stations and pump-houses, with walls substantially impermeable to the passage of water. All structural components located below the base flood level must be capable of resisting hydrostatic and hydrodynamic loads and the effects of buoyancy. In Zones V1-V30, VE and V flood proofing of structures is not an allowable alternative to elevating the lowest floor to two feet above the base flood elevation.
- 4) If the structure is to be flood proofed, a licensed professional engineer or architect shall develop and/or review structural design specifications and plans for construction and shall certify that the design and methods of construction are in accordance with accepted standards of practice and shall provide a copy of that certification to the local floodplain administrator.
- 5) New and replacement electrical equipment, heating, ventilating, air conditioning, plumbing connections, and other service equipment shall be located at or above the base flood elevation or be designed to prevent water from entering and accumulating within the components during a flood and to resist hydrostatic and hydrodynamic loads and stresses. Electrical wiring and outlets, switches, junction boxes and panels shall be elevated to or above the base flood elevation unless they conform to the appropriate provisions of the electrical part of the Building Code of New York State or the Residential Code of New York State for location of such items in wet locations;
- 6) New and replacement water supply systems (e.g., supplying water to a WWTP) shall be designed to minimize or eliminate infiltration of flood waters into the system;
- 7) New and replacement sanitary sewage systems shall be designed to minimize or eliminate infiltration of flood waters. Sanitary sewer and storm drainage systems for buildings that have openings below the base flood elevation shall be provided with automatic back-flow valves or other automatic back-flow devices that are installed in each discharge line passing through a building's exterior wall; and,

- 8) On-site waste disposal systems shall be located to avoid impairment to them or contamination from them during flooding.

Furthermore, if wastewater treatment projects are being federally funded, Executive Order 11988 requires that Federal agencies funding and/or permitting critical facilities avoid the 0.2% (500-year) floodplain or protect the facilities to the 0.2% chance flood level; wastewater treatment facilities are considered critical facilities. New York also has regulations governing state agency projects and state-funded projects in flood hazard areas. “6 NYCRR Part 502: Floodplain Management Criteria for State Projects” pertains to state projects, real estate development, or the financing of such development, by any state agency or on state land. The term “project” is defined in Section 502.2.u. (Statutory authority: Environmental Conservation Law, §§1-0101, 3-0301, Article 36).

B. 6. Design Criteria

Typically an OWTS is designed based on an expected or known hydraulic loading rate and a determined soil acceptance rate or application rate for a filter media. However, a hydraulic loading rate does not take into consideration the effect of elevated organic and solids loading on the soil or filter media. Therefore, to design an effective OWTS the raw wastewater must be accurately characterized, and the daily wastewater flow volume, and flow rate over the Significant Delivery Period(s) must be reliably estimated.

B. 6. a. Wastewater Characterization

Raw wastewater from commercial and institutional facilities can generally be divided into two types; residential (average-strength) and non-residential (high-strength).

Residential wastewater is typically generated by water-using activities such as personal hygiene, food preparation and cleaning. Wastewater is discharged from various plumbing fixtures and appliances such as toilets, sinks, bathtubs and clothes washers from the following intermediate-sized facilities:

- Cluster housing and multi-home developments
- Apartment buildings
- Mobile home parks
- Other facilities that generate wastewater similar in characteristics to residential wastewater

Traditionally, the most important wastewater characteristics to consider when designing residential

OWTS are biochemical oxygen demand or BOD₅ (organic loading), total suspended solids or TSS (solids loading), and fats, oils and grease (FOG) levels. In specific cases, total phosphorus (TP) or ammonia (NH₄) discharges may need to be limited.

Typical influent concentrations of these parameters in residential wastewater may range as follows:

Parameter	Concentration, mg/l ¹
BOD ₅	155 – 286
TSS	155 – 330
FOG	70 – 105
TP	6 – 12
NH ₄	4 – 13

¹ EPA 625/R-00/008-Chapter 3, Table 3-7, 2002.

Commercial and institution facilities may generate non-residential (high-strength) wastewater from garbage disposal use, food preparation, food service, on-site linen service or sanitary dump stations. Non-residential wastewater will typically have higher concentrations of BOD₅, TSS and FOG than those listed above. High-strength wastewater is generated in the following facilities:

- Hospitals, nursing homes, other medical institutions
- Hotels, motels, schools, and prisons
- Kennels, veterinary clinics, animal rescue shelters
- Sanitary dump stations serving roadside rest areas, campgrounds, or other recreational facilities
- Food service establishments
- Laundromats or facilities with on-site linen laundry
- Supermarkets, butcher shops, bakeries

These facilities may generate wastes deserving special consideration due to elevated concentrations of wastewater constituents. Some typical activities include:

- Floor stripping (1)
- Cleaning and disinfecting (1)

- Disposing of waste pharmaceuticals (2)
- Disposing of sanitary wastes from people receiving medical care (e.g.: chemotherapy) (2)
- Disposing of food wastes with FOG concentrations two-to-five times that of residential (various cuisine types) (3)

Notes:

- 1) Frequency of floor stripping and type and frequency for disinfection of surfaces in institutional type facilities is often mandated by law.
- 2) NYSDEC does not encourage disposal of any unused prescriptions into any wastewater treatment system. Information on proper disposal of household prescriptions and over-the-counter drugs can be accessed at www.dec.ny.gov/chemical/45083.html.
- 3) Studies (R.L. Siegrist, CO; B. Lesikar, TX; B. Stuth, WA; and J. Converse, WI) show that wastewater from a restaurant is typically 2.7 and 2.8 times higher for BOD₅ and TSS, respectively, than residential wastewater. The influence of other factors such as a self-serve salad bar, types of oil used, dishwashing procedures, and restroom use should be considered.

Any of these components can interfere with the normal biological processes most on-site treatment systems use. These characteristics can vary day to day or hour by hour and can have a major impact on system performance. It may be appropriate to provide pretreatment or advanced treatment of the wastewater or reduce the hydraulic loading rate of the treatment system components to compensate for toxic effects, elevated organic and/or solids loading.

Commercial and institutional facilities often generate a majority of their daily wastewater over Significant Delivery Periods depending on the nature of the business, and also may not have a continuous base flow. For example, schools have high flows during only a few hours of the day, and no flow is present in the late afternoon and night. Peak hourly flows generated during these times of the day must be accounted for in the design of the system components. Solids build-up and anaerobic conditions due to low flows must also be taken into account. If subsurface disposal is to be used, the treatment process should remove nearly all settleable solids and floatable grease and scum to allow efficient operation of the soil-based treatment and dispersal system. See Section D Preliminary and Primary Treatment, Flow Measurement and Appurtenances.

Industrial wastewater must be segregated and treated under a SPDES permit, as a general rule, but inclusion with sanitary wastes may be considered, if justified. Institutions may also generate industrial wastes from sinks or drains installed in science labs, or auto-repair, art, or other vocational classrooms. These wastes should be directed to an approved SPDES-permitted treatment system, industrial waste holding tank, or an approved hazardous waste collection receptacle.

B. 6. b. Design Flow

A determination of the expected wastewater flow rate is necessary for the design of effective wastewater collection and treatment systems. At a minimum, the design average day_(i.e. annual average day flow), design maximum day_(i.e. the highest annual average day flow), and design peak hourly flow rates shall be identified and used as a basis for design. Although numerous factors must be considered, in general, for subsurface soil-based treatment and dispersal systems, the expected wastewater flow rate used for design should be the design_maximum day flow. Other treatment units shall consider all design flows in conjunction with any proposed equalization.

Peaking factors are commonly used to correlate the average day flow, maximum day flow, and peak hourly flow. In general, the maximum day flow is approximately 1.5 to 2.0 times the average day flow and the peak hourly flow is approximately 4.0 times the average day flow. For some establishments variations in wastewater flow rate may be more extreme. In these cases it is necessary to examine the Significant Delivery Period of the wastewater and base the design flow upon this information to prevent an excessive rate of flow through the collection and treatment systems. Equalization prior to treatment units shall be considered for all designs.

The determination of the expected wastewater flow rate is typically based on one of the following three methods: 1) by measuring the actual wastewater flow rate of existing facilities or similar facilities; 2) by using water-usage data to estimate the expected wastewater flow rate if sufficient data is available for an establishment; or 3) by using Table 3 as a basis for the design of proposed facilities or existing establishments when the actual wastewater flow rate cannot be measured.

Method 1: Measured Wastewater Flow Rate Data

A minimum of one year of data collected during similar operational conditions. Table 3 values and/or water-usage data should be considered for comparison purposes. If sufficient measured wastewater flow rate data is not available, Method 1 should not be

used. The average of daily (24-hour) measured wastewater flow rates over the analysis period is an acceptable estimation for the average daily flow for design purposes. The analysis period must account for operational variations (i.e. peak seasonal, week-ends, etc.). The single highest daily (24-hour) measured wastewater flow rate during the analysis period, excluding extraneous data, is an acceptable estimation for the maximum average day flow. However, there should be a reasonable explanation for omitting “extraneous” measured values.

Method 2: Water Usage Data

If measured wastewater flow data is not feasible to gather, the reviewing engineer may require a minimum of one year of water use data collected during similar operational conditions. Table 3 values and/or measured wastewater flow rate data should be considered for comparison purposes and must be utilized if sufficient water-usage data is not available. The average of daily (24-hour) water-usage rates over the analysis period, after adjustments for infiltration and inflow as well as for water that will not reach the sewer (i.e. boiler water), is an acceptable estimation for the average day flow. The analysis period must account for operational variations (i.e. seasonal, weekends, etc.). The single highest daily (24-hour) water-usage rate during the analysis period is an acceptable estimation for the maximum average day flow, excluding extraneous data, and after adjustments for infiltration, inflow, and water that will not reach the sewer. There should be a reasonable explanation for omitting “extraneous” measured values.

Method 3: Typical Per-Unit Hydraulic Loading Rates (Table B.3)

Typical average daily flows in gallons per dwelling unit, per customer/client, or per sq. ft. for a variety of commercial, institutional and recreational establishments are presented in Table B.3. When an establishment includes several different types of uses from the table, each use must be computed separately and the design flow must be based on the sum of all uses. The hydraulic loading rates provided in Table B.3, used in conjunction with maximum expected operational conditions (i.e. maximum occupancy), is an acceptable estimation for the maximum average day flow. Design flows for facilities not listed in Table B-3 shall be based on actual water meter readings of established flows from other existing or similar installations (per Method 2).

Except for the 110/130/150 gpd residential and lodging values, the per-unit hydraulic loading rates in Table B.3 may be decreased by 20 percent for establishments equipped with water saving plumbing fixtures (sinks, toilets and showers). The residential unit flow of 110 gpd per bedroom already accounts

for the 20% flow reduction. A combination of high- and low-flow fixtures can also be considered on a pro rata basis. Fixtures that use even less water (e.g. toilets using 0.5 gallons of water per flush) are available and the reduction of wastewater flow attributable to these and other new technologies shall be considered on a case-by-case basis. The reduction allowance shall depend in part upon the ability of the builder or owner to ensure adequate maintenance and/or replacement in-kind when necessary.

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Table B.3. Typical Per-Unit Hydraulic Loading Rates*Residential*

<i>Type of Use</i>	<i>Unit</i>	<i>Gallons per Day</i>
Apartment	Per Bedroom	110/130/150 *
Mobile Home Park	“Single-Wide” Home	220
	“Double-Wide” Home	330
Single Family Residence	Per Bedroom	110 / 130/ 150 *

* 110 gpd for post-1994 plumbing code fixtures; 130 gpd for pre-1994 fixtures; and 150 gpd for pre-1980 fixtures. Homes or lodging establishments with high-flow fixtures need to account for any higher peak flow periods.

* For Individual Household Systems under 1000 gpd, use design flows in the NYSDOH Wastewater Treatment Standards – Residential Onsite Systems - Appendix 75-A.

Campgrounds

<i>Type of Use</i>	<i>Unit</i>	<i>Gallons per Day</i>
Day Camp	Per Person	15
	Add for Shower	10
	Add for Lunch	5
Campground	Per Unsewered Site *	55 (includes showers)
	Per Sewered Site	100
Campground Day Use	Per Person	5
Dumping Station **	Per Unsewered Site	10
	Per Sewered Site	5

* Additional capacity for food service, and laundry shall be provided. Structures available for overnight occupancy other than those meeting the definition of a camping unit shall be based on 150 gpd / unit for design flow purposes. (see NYSDOH – Chapter 1 State Sanitary Code Subpart 7-3 Campground).

** The addition of flow for dump-station sewage may be pro-rated by using an estimated percentage of

sites suited for RV use based on historical data.

Institutional

<i>Type of Use</i>	<i>Unit</i>	<i>Gallons per Day</i>
Assisted Living Facility/ Complex	Per Bed – (1); (2) add 10 gpd for in-room kitchen;	110/130/150
Group Home (residential style building)	Per Bed - (1) add 150 gpd per house for garbage grinder;	110/130/150
Nursing Home (hospital care)	Per Bed - (1); (2)	175
Hospital	Per Bed - (1); (2)	175
	Per Out-Patient	30
Church	Per Seat; (1)	3
Church Hall / Fire Hall	Per Seat; (2)	10
Library/ Museum	Per Patron; (1); (2)	5
Public Park	Per Person (toilet only)	5
Prison / Jail	Per Inmate; (1); (2)	150
School – Day	Per Student	10
-or-	Elem./ Jr. High / Sr. High	7 / 9 / 12
	Add for meals / showers	5 / 5
School Boarding	Per Student; (1); (2)	75

(1) add 15 gpd per employee

(2) add for Food Service Operations, e.g. 24-hour restaurant)

Commercial

Type of Use	Unit	Gallons per Day
Airport/Bus/Rail Terminal	Per Passenger (1) Per Toilet	5 400
Barber Shop / Beauty Salon	Per Station without and with hair-care sink	50 / 200
Bowling Alley	Per Lane (1);(2)	75
Bed & Breakfast	Per Room (see note under Residential)	110/130/150
Casino	Per Employee/shift plus Per Sq. Ft. for non-lodging customer use	15 0.3
Country Clubs & Golf Courses	Per Round of Golf (1);(2) (add for bar, banquet, shower or pool facilities and golf tournaments)	20
Concert Hall / Arena / Assembly Hall / Theater / Stadium / Skating Rink	Per Seat (1);(2)	5
Day Care	Per Child (1)	20
Doctors Office	Per Doctor	250
Dog / Pet Grooming	Per Station	500
Also see Kennel and Veterinary Office below.		
Dentist	Per Chair (3)	250
Drive-In Theater	Per Car Space (2)	5
Factory / Distribution Warehouse	Per Employee/shift; add for showers	15 10
Fairgrounds	Per Visitor (2)	5
Health Club	Per Patron	20

Highway Rest Area	Per Traveler (2)	5
	Per Dump Station Vehicle	7
Hotel	Per Sleeping Unit (2) add for banquet hall, night club, pool/spa, theatre, etc.	110/130/150
Kennel	Per Kennel/Run/Cage	50
Laundromat	Per Machine	580
Marina	Per Slip (2) with shore-side restroom facilities including shower;	20
	add per slip for dump station	7
Migrant Worker Housing	Per Person	50
Motel	Per Sleeping Unit;	110/130/150
	add for in-room kitchen;	10
	add for in-room jacuzzi/spa	20
Office Building	Per Employee; (2)	15
	add for showers	5
Service station / Convenience store	Per Toilet (2)	400
Shopping Center / Grocery Store / Department Store	Per Sq. Ft.; (1);(2) add for deli, bakery, butcher	0.1
Swimming Pool / Bath House	Per Swimmer	10
Veterinary Office	Per Veterinarian	200

(1) add 15 gpd per employee/shift

(2) add for Food Service Operations, e.g. 24-hour restaurant)

(3) Dental offices must recycle mercury amalgam instead of washing it down the drain.

The NYSDEC website has guidance that references the 2002 law:
www.dec.ny.gov/chemical/8845.html

[illegible]

B. 6. c. Non-Contaminated Flow

Similarly, leaking sewer joints, pipe-tank seals, tank-riser seals, cracks in treatment tanks and manhole covers that are not watertight can be significant sources of clean water entry into the system. These extraneous flows can cause periodic hydraulic overloads and affect treatment performance which can lead to system failure. Exfiltration from these items can have a negative impact on ground water quality.

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Discharges within 250 feet of a stream, pond, lake or wetland may be prohibited or require a SPDES permit.

Water Softener Discharge

Studies by soil scientists have found that the volume or chemical composition of wastewater from the regeneration cycle (backwash, recharge, rinse) of a properly operated and maintained household-sized water softener is not harmful to an OWTS that is properly designed, operated and maintained. It was noted, however, that the volume of wastewater can be reduced by:

- Activating the regeneration cycle based on need (not on a timer)
- Utilizing a water softener with a large mineral tank.
- Employing water conservation measures to reduce flow to be treated by the softener system.

However, some proprietary enhanced treatment system manufacturers (e.g., aerobic treatment units) have warranties that are voided by the discharge of water-softener wastewater due to problems cited in septic tanks/aerobic tanks with elevated chloride content (up to 100 times the normal non-softened concentration of 50 mg/l). The change in density caused by the brine has caused discharge of solids from primary treatment tanks into secondary treatment units disrupting flocculation and settling.

While it is true that discharges high in sodium content can seal pores in some clay soils, it is not clear whether the discharge of softened water or the discharge of the backwash water is problematic. The 2002 EPA Special Issues Fact Sheet on Water Softeners states that: "... some people have the misconception that the salt brine that enters the ion exchange tank also exits the tank as wastewater. In fact, the influent with its high concentration of sodium ions is very different than the effluent, which has a high concentration of calcium and magnesium ions. Consequently, the potential for chemical clogging of clayey soil by sodium ions is reduced. The calcium and magnesium input may even help improve soil percolation."

If clay soil "sealing" is not a problem, but concern for treatment process upset is, softener discharges can by-pass the primary and secondary treatment units and discharge directly to the soil treatment/absorption system. Alternatively, a dedicated soil absorption system may be designed by the engineer if it is deemed necessary or appropriate.

The discharge of water softener recharge / regeneration wastewater to septic tanks, or aerobic or enhanced treatment units is not recommended for wastewater treatment systems treating more than 1,000 gpd. If softener backwash is to be added to any wastewater system with a design flow over 1,000 gpd, then the design engineer will need to provide data showing that the effluent will meet applicable water quality standards.

B. 6. d. Treatment Considerations

Acceptability of any discharge to a surface water body will be contingent upon the ability of the proposed OWTS to meet applicable water quality standards and criteria. Upstream and downstream discharges and uses must be considered in the water quality review process. DEC's Division of Water TOGS 1.3.1 provides guidance used to determine if a discharge will meet water quality standards. Most typically, these determinations are made on the basis of critical low flow conditions or, minimum average seven consecutive day flow expected to recur once in 10 years (MA7CD/10yr. Flow).

The minimum degree of treatment required for the discharge of sanitary wastewater into non-intermittent surface waters is effective secondary treatment. Typical effluent limits are shown in Table B.4-A below:

Table B.4-A Typical effluent limits for non-intermittent streams

Parameter	Type	Limitation	Units
BOD ₅	30-Day Arithmetic Mean	30 (1)	mg/l
BOD ₅	7-Day Arithmetic Mean	45 (1)	mg/l
TSS	30-Day Arithmetic Mean	30 (1)	mg/l
TSS	7-Day Arithmetic Mean	45 (1)	mg/l
Settleable Solids	Daily Maximum	0.3/0.1 (2)	ml/l
pH	Range	6.0 – 9.0	SU
Fecal Coliform	30-Day Geometric Mean	200 (3)	Number of colonies per

			100 ml
Fecal Coliform	7-Consecutive Day Geometric Mean	400 (3)	Number of colonies per 100 ml
Total Residual Chlorine	Daily Maximum	(3) (4)	mg/l
Ammonia	30-Day Arithmetic Mean	(4)	mg/l as NH ₃
Total Phosphorus	Site-specific	(4)	mg/l as P
Temperature	Site-specific	70° for trout waters as an effluent limit; 90° for non-trout waters as an in-stream, mixed limit	Degrees F

- (1) effluent values shall not exceed 15% of influent values for both BOD₅ and TSS.
- (2) No Sand Filtration = 0.3 mg/l, Sand Filtration = 0.1 mg/l.
- (3) monitoring of these parameters only required during the period when disinfection is required.
- (4) limitation may be required depending on site-specific conditions.

In certain instances, the project information is forwarded to the Central Office of the DEC for a waste assimilative capacity analysis and allocation for setting water quality based effluent limits. A water quality review of the project is conducted utilizing the guidance of TOGS 1.3.1. The waste assimilative capacity analysis and allocation process provide the basis for setting water quality based effluent limits as necessary. Such limits represent additional treatment, beyond secondary, to ensure that all applicable water quality standards and criteria are met.

In New York State, the most stringent water-quality-based effluent limits for a sanitary discharge are these for a discharge to an intermittent stream. An intermittent stream is defined as:

- 1) Any stream that periodically goes dry at any point downstream of the proposed point of discharge, or
- 2) Any stream segment below the proposed point of discharge in which the MA7CD10 stream flow is less than 0.1 cfs as estimated by methods other than continuous daily flow measurements.

Discharge to an intermittent stream will be allowed only when all other methods of disposal have been considered and judged unacceptable. Data should be supplied to show that the discharge from any wastewater treatment facility would not contravene stream standards. Typically, discharges to intermittent streams must meet the limits shown in Table B.4-B.

Table B.4-B Typical effluent limits for intermittent streams

Parameter	Type	Limitation	Units
BOD ₅	Daily Maximum	5	mg/l
TSS	Daily Maximum	10	mg/l
Settleable Solids	Daily Maximum	0.1	ml/l
Total Residual Chlorine	Daily Maximum	0.02	mg/l
Ammonia	Daily Maximum. or Average	2.2 in winter 1.5 in summer	mg/l as NH ₃
Dissolved Oxygen	Daily Minimum	≥ 7.0	mg/l
pH	Range	6.0 – 9.0	SU
Total Phosphorus	Site-specific	(1)	mg/l as P
Temperature	Site-specific	70° for trout waters as an effluent limit; 90° for non-trout waters as an in-stream, mixed limit	Degrees F
Coliform, fecal, when disinfecting	30-day geometric mean	200	Number of colonies per 100 ml
Coliform, fecal, when disinfecting	7 consecutive day geometric mean	400	Number of colonies per 100 ml

(1) Limitation, to be set by review agency, may be required depending on site-specific conditions.

Notes: 1. Efficient nitrification cannot be expected from buried sand filter treatment systems that operate on a seasonal basis.

2. Operational experience indicates that a single-pass intermittent sand filter alone may not provide sufficient treatment to meet intermittent stream effluent limits.

As noted previously in the introduction, a septic tank followed by a soil absorption system is the preferred method of treatment. However, based on the site evaluation and subsurface soil evaluation, discharge to a septic tank/soil absorption system may not provide effective treatment, and so a surface discharge may be selected.

Direct discharge of effluent from an OWTS to a receiving stream may be allowed contingent upon the design of the treatment system to meet applicable criteria and standards for the stream at the point of discharge and any point downstream and the applicant applying for and being issued a SPDES permit from DEC. When deriving a water quality based SPDES permit effluent limitation from a surface water standard or guidance value, contributing factors like analytical detectability, treatability, natural background levels and the waste-assimilative capacity of the receiving stream must be taken into account.

Effluent limitations should be obtained by contacting the Department prior to initiating design. An engineering report that addresses the capability of the treatment system to meet the proposed effluent limitations must be included with the submission of design plans. In certain instances, the project information is forwarded to the Central Office of the DEC for a waste assimilative capacity analysis and allocation for setting water quality based effluent limits (WQBELs). Such limits represent additional treatment, beyond secondary, to ensure that all applicable water quality standards and criteria are met.

B. 7. Groundwater Monitoring and Monitoring Well Requirements

Subsurface disposal is frequently the choice for smaller systems (30,000 gpd or less). Groundwater monitoring is required for systems discharging over 30,000 gpd to soil absorption systems. Surface discharges are more often selected for larger systems (see Section I Disinfection and Reoxygenation of Surface Water Discharges).

Depending on the results of the site evaluation and what was agreed upon at the pre-application meeting with the Regional DEC staff, groundwater monitoring and monitoring well requirements may be required for the project. The following specifications and monitoring frequency should be discussed

with the reviewing engineer. There may also be local requirements in some areas of the state, e.g. Suffolk County Department of Health, Nassau County Department of Health.

Monitoring Well Installation

Monitoring wells are to be designed to meet site-specific conditions of geology and hydrology with special considerations given to:

- characteristics of soil and rock formation at the site;
- depth to watertable, bedrock, and any impervious layer;
- aquifer thickness
- rate and direction of groundwater flow;
- seasonal variations in flow, depth and direction of flow;
- potential for mounding of groundwater and its effects;
- presence of and distance to nearby surface water;
- presence of and distance to nearby water supply wells; and
- other relevant site-specific considerations.

In general, monitoring well construction should follow the requirements for solid waste facilities as per 6 NYCRR 360-2.11(a)(8), and the guidelines listed below:

- not less than 2-inch inside diameter;
- screen length to be at least 5 feet, but not more than 20 feet in length;
- the depth and length of the screen must be such that annual fluctuations of the water table do not result in a dry well.

Deviations from these requirements must receive DEC approval prior to construction. If there are any questions regarding these guidelines, or they are in conflict in some manner, please consult the Regional Water Engineer.

Driller's Log

A well driller's log must be submitted to the Department for each permanent monitoring well constructed. Well completion logs must contain a diagram of the completed well, all pertinent details on well construction, a description of the materials used, and elevations of all well features and include:

- monitoring well identification number and location;
- type of drilling equipment, driller, and drilling company;
- method of drilling, including size of borehole;
- type, size and placement of casing, including amount of casing above grade;
- type, size and placement of well screen;
- type, size and placement of filter pack;
- type, size and placement of annular seal;
- description of materials penetrated;
- depth to water table, and date and time measured;
- monitoring well development details

A site map must be provided at appropriate scale showing monitoring well locations in relation to the subsurface disposal system as designed. The number and location of wells should be determined in conjunction with the reviewing engineer.

Water Level Measurement

Water level measurements for each monitoring well must be related to a permanent reference point on the well casing. The locations and elevations of all monitoring wells must be surveyed to obtain their precise location and plotted on a map. The vertical location of the ground surface and the water-level measurement reference mark of each monitoring well must be accurately measured to the nearest 100th foot using a common datum, preferably the NAD83 datum. Static water-levels should be measured to the nearest 100th foot.

Water Quality Sampling

- Proper sampling procedures should be followed to prevent contamination of the wells or the collected samples.
- Analysis should be by state certified labs and approved analytical methods;
- Sampling should be conducted by properly trained individuals;
- Appropriate QA/QC procedures should be followed;
- Appropriate sampling chain of custody record keeping should be followed;
- Obtain static water level for each well and sampling event;

- Thorough decontamination rinsing of pumping/bailing equipment is necessary prior to use in each well.
- Up-gradient monitoring wells should be sampled first.
- An amount of water equal to at least three times the volume of water standing in the well must be removed prior to sampling.
- The well should be allowed to recover to at least 75 percent of the static water level prior to sampling.
- The water taken from each well should be held or discharged on the ground far enough away from the well such that it will not affect the sampling results. Soil erosion must also be avoided.

Sampling Parameters

Baseline (first round) Sampling.

The following lists contain parameters for which analytical procedures are provided in 40 CFR Part 136. Sampling and analysis for these parameters shall be conducted using the most sensitive method currently promulgated under 40CFR Part 136. The parameters listed here are the “BASELINE (FIRST ROUND) SAMPLING PARAMETERS”: Monitoring well static water levels, Specific Conductance, pH, Chloride, Alkalinity, Total hardness as CaCO₃, Field Observations (see note 4 below), Turbidity, Ammonia, Total Kjeldahl Nitrogen (TKN), Nitrate, Nitrite, Sulfate, Total Phosphorus (TP), Chemical Oxygen Demand (COD), Total Dissolved Solids (TDS), Total Organic Carbon (TOC), Total Dissolved Solids (TDS), MBAS (methylene blue active substances - an indicator of foaming agents), and Fecal Coliform.

Subsequent (Routine) Monitoring

All parameters listed here are “SUBSEQUENT (ROUTINE) SAMPLING PARAMETERS”: Monitoring well static water levels, Specific Conductance, pH, Field Observations (see note 4), Chloride, Nitrate, Total Phosphorus, Total Dissolved Solids, MBAS (methylene blue active substances – an indicator of foaming agents), Fecal Coliform. Monitoring frequency will be determined by the reviewing engineer.

Notes:

1. The department may modify either list as necessary.
2. Common names of the given parameters are those widely used in government regulations, scientific publications, and commerce; synonyms exist for many chemicals.
3. Where "Total" is used, all species in the groundwater that contain this element are included.
4. Any unusual conditions (colors, odors, surface sheens, etc.) noticed during well development, purging, or sampling must also be reported.

C. Sewers and Sewage Pumping Stations

C.1. Introduction

Most municipal collection systems built under the federally-funded Construction Grant Program, or since 1990 funded via the Clean Water State Revolving Fund, used conventional gravity sewers to convey wastewater. Factors associated with municipal and intermediate-sized systems such as low density of residences, adverse grades or soil conditions, high groundwater, or high rock elevations may make gravity sewers impractical and thus encourage the use of alternative collection systems. These factors increase as wastewater systems are proposed for areas beyond the large and already developed population centers. Alternative systems such as septic tank effluent, grinder pump, and vacuum sewers can be proposed in these instances. Septic tank effluent sewer systems may be a combination of pump and gravity systems. Cost analysis results (capital plus operation and maintenance over the life of the system) have shown that alternative collection systems are at times more cost-effective than conventional gravity systems. Alternative systems can also be used in combination with conventional gravity and pump station systems.

Sewer systems should be designed for the estimated future tributary population, which includes maximum anticipated capacity of institutions, industrial parks, etc. In some cases projected populations have not been attained, or retro-fitted low-flow fixture use has resulted in wastewater velocities staying below scour velocity long enough to generate odors and destructive atmospheres in pipes and pump stations. Use of site-specific water use rates, existing capacity, and flow-meter data should be considered for sewer extensions to make use of existing capacity and discontinue unnecessary over-designing. Conversely, systems built for a fixed geographical area, e.g. a small lake community, should be designed with full build-out in mind and flexibility to allow for undesired or unanticipated growth (using modular treatment units with sufficient land available).

C.2. Building Sewers

Building drain

The building drain is a pipe extending from the interior plumbing to a point at least three feet outside the foundation wall, as recommended by DEC. The New York State Uniform Fire Prevention and Building Code defines a building drain as extending 30" from the outside building wall, and the piping thereafter to the point of disposal is defined as the building sewer. The New York State Uniform Fire Prevention and Building Code specifies requirements for both building drains and building sewers.

Venting

Where required locally, the New York State Uniform Fire Prevention and Building Code allows traps to be installed inside or outside the building, and requires a relief vent or fresh air intake on the inlet side of the trap. The design of any sewage collection system should account for the location of the house/building trap and vent, and their compatibility with the venting requirements of the sewage collection system.

Building sewer

The building sewer connects the building drain to on-site treatment facilities, or the municipal sewer. A second building sewer is required to convey non-petroleum grease-laden wastewater to a grease interceptor (see Section D.5). Building sewers shall have a minimum diameter of four inches and shall be laid on a firm foundation with a minimum grade of 1/4 inch per foot and straight alignment. Minimum grades assure the shallowest depth for the soil-based treatment and dispersal system.

At least one cleanout should be provided for all building sewers. If bends of 45 degrees or more are necessary, an accessible cleanout fitting shall be provided for each bend.

C.3. Conventional (solids-handling) Gravity Sewers, Manholes and Pump Stations

C.3.a. Conventional Gravity Sewers

Piping Material Specification

Generally accepted material for sewers will be given consideration, but the material selected should be adapted to local conditions, such as character of wastes, possibility of septicity, soil characteristics, exceptionally heavy external loadings, and abrasion. Maximum benefit can usually be achieved with non-metallic materials such as polyethylene, fiberglass reinforced plastic, and PVC. Other acceptable pipe materials are ABS, vitrified clay and fiberglass/polyester composite.

Concrete, asbestos cement, steel, iron and other metals are not desirable. Bituminous-fiber pipe, commonly known as Orangeburg pipe, is unacceptable for collection systems to be approved by the department.

All sewers shall be designed to prevent damage from superimposed loads. Structural reinforcing may be necessary for gravity sewers installed at depths of less than four feet. Proper allowance for loads on the sewer due to the width and depth of the trench shall be made. All flexible pipes (PVC, FRP, etc.) shall undergo alignment, deflection and leakage testing after installation according to Sections 33.5, 33.8, and 33.9 of the Ten State Standards, or Sections 2.3.6, 2.3.10.3, 2.3.10.1 and 2.3.10.2, respectively, of TR-16 (also see Appendix C “Sewer and Manhole Leakage Tests”).

Relevant ASTM standards include:

ASTM D-1785 for Schedule 40, 80 and 120 pipe

ASTM D-2729 for PVC sewer pipe & fittings

ASTM D-2852 for Styrene Rubber plastic drain pipe

ASTM D-3034 for PSM PVC sewer pipe & fittings (SDR 35 gravity sewer)

Design flow

For gravity sewers conveying raw sewage, the expected wastewater flow rate used for design should be the design peak hourly flow.

Pipe Sizing

The policy of DEC in regard to minimum diameter and slope of solids-carrying, gravity sewers for most municipal collection systems has been to comply with the guidelines prescribed by Section 33 of the Ten State Standards, or Section 2.3 of TR-16.

For these Design Standards for Intermediate-sized Wastewater Treatment Systems, the following determinations have been made:

- For small municipalities or cluster developments, no public gravity sewer conveying raw sewage shall be less than 8-inches (20 cm) in diameter. Such sewer systems using 8-inch (20 cm) or larger diameter pipe shall be governed by the Ten State Standards, or TR-16.
- For institutional, commercial and other private facilities, sewer systems conveying raw sewage may use a 6-inch (15 cm) minimum diameter collector sewer with a minimum slope

of 1/8-inch per foot (1.0%). Trunk sewers shall be a minimum of 8-inch diameter with a minimum 0.4% slope. In very small installations, 4-inch diameter sewers may be used for raw sewage if a minimum slope of 1/4-inch per foot (2%) is maintained and a velocity of at least 2 feet/second is achieved when the sewer is flowing full. The use of smooth interior pipe is recommended.

Sewer Installation

Sewers shall be sufficiently deep to prevent freezing. Where freezing conditions could occur, and sufficient depth cannot be provided, insulation or a raised earthen berm shall be provided. Excavations shall meet OSHA requirements and conform to standard accepted practices.

All pipes shall be properly bedded in accordance with standard accepted practice for the type of pipe being installed. Backfilling shall be performed in steps. Backfill shall be placed in such a manner as not to disturb the alignment of the pipe, and shall be slightly mounded to allow for settling. Where velocities greater than 15 fps are expected, special provisions shall be made to protect against displacement.

Conventional gravity sewer lines two feet in diameter or less must be laid on straight alignment and uniform slope between manholes using the minimum slopes given in Section 33.4 of the Recommended Standards for Wastewater Facilities.

Sewer joints shall be designed to minimize infiltration and to prevent the entrance of roots throughout the life of the system. All sanitary sewers, manholes and cleanouts shall be tested for leakage (see Appendix C “Sewer and Manhole Leakage Tests”).

C.3.b. Conventional Sewer Manholes

Manholes shall be placed on large pipe raw sewage gravity sewers (8” minimum) at the junction of two or more sewer lines; all points of change of grade, size or alignment; at the end of all lines; and at distances not to exceed 400-feet.

If the topography is very uneven and frequent changes in alignment and slope are necessary, a limited number of inspection pipes may be substituted for manholes. Not more than one inspection pipe should be placed between two successive manholes and in no case shall manhole separation exceed 400-feet.

For gravity sewers conveying raw sewage, cleanouts may be substituted for "end of line" manholes only where the length of run to a manhole does not exceed 150-feet.

Drop manholes (90-degree) shall be used for all conventional gravity sewers entering at an elevation of 24 inches or more above the manhole invert. Where the difference in elevation between the incoming sewer and manhole invert is less than 24 inches (61 cm), the invert should be filleted to prevent solids deposition.

Sizing for manholes and other sewer appurtenances is given in **Table C-1**. Non-standard manholes may be used on small diameter sewers (4- and 6-inch) if the system cannot be dedicated as a portion of a municipal system. There must be a smooth channel formed on the bottom and the pipe entrances shall be properly grouted.

Manholes shall be of pre-cast or poured in place concrete, and shall be waterproofed on the exterior. Exterior waterproofing may be omitted for pre-cast manholes when information proving water-tightness is provided. Inlet and outlet pipes shall be joined to the manhole with any watertight connection that allows for differential settlement to take place. Covers must be above grade or made watertight and of sufficient weight or design to prevent unauthorized entry.

Table C.1. Minimum Size of Sewer Appurtenances

Fixture	Diameter
Drop Manholes	60 inch
Standard Manholes	42 inch
Non-standard Manholes	24 inch
Inspection Pipes	24 inch
Cleanouts	8 inch ^a

a.) Cleanouts may be less than 8 inches if the pipe diameter is less than 8 inches.

C.3.c. Sewage Pumping Stations

1. Design Considerations for Pump Stations

Sewage pumping stations shall not be subject to flooding as described in Section B of this document. A suitable super-structure is desirable to allow convenient access under all weather conditions. Below-grade dry-pit pumping stations should have a float switch mounted at floor level, which is connected to the "HIGH LEVEL" alarm.

For gravity sewers and pump stations the expected wastewater flow rate used for design should be the design peak hourly flow. In general the pump installation shall be designed to handle a design flow of four times the average daily flow. Certain applications may require a higher peaking factor.

Where applicable, with due consideration given to the particular wastewater characteristics or pump station design, the pump should be preceded by readily accessible bar racks with clear openings not exceeding 1-1/2 inches. Exceptions may be allowed if pneumatic ejectors are used or if special devices are installed to protect pumps from clogging or damage. Consideration should be given to duplicate racks or a suitable overflow bypass for use in emergencies. Where racks are located below ground, convenient facilities should be provided for handling screenings.

Except where grinder or cutter pumps are used, 3-inch pumps handling **raw sewage** shall be capable of passing spheres of at least two inches (2") in diameter. Larger pumps should comply with New York's official standards for municipal wastewater treatment and collection facilities. Pumps shall be so placed that under normal operating conditions they will operate under a positive suction head except for self- or vacuum-priming pump systems.

Electrical equipment in wet wells or in enclosed spaces where explosive gases may accumulate, or where there is risk of being submerged, shall be designed and installed in strict conformance with the latest edition of the National Electric Code, NEC.

At least two (2) pumps or pneumatic ejector compressor/tank assemblies shall be provided except for unusual circumstances, which shall be reviewed individually. In the case where only two units are provided, each shall be capable of handling in excess of the expected maximum design flow and the two pumping units shall be identical. If three or more pumps are used, they should be designed such that with any one unit out of service the remaining units will have the capacity to handle the maximum design flow.

Suitable shutoff valves shall be placed on the suction line of each pump. A shut-off and a check-valve shall be placed on the discharge line of each pump. In the case of submersible pumps, valves shall be located outside of the pump station unless they are accessible from grade without the need to enter the wet well. Where the wet well volume is less than the volume contained in the force main, consideration should be given to placing a shut-off valve on the force main to permit servicing of the valves. Where the wet well non-working or reserve volume is inadequate for emergency periods, consideration should be given to an easy bypass connection in the piping to connect an emergency pump.

There shall be no physical interconnection between any potable water supply and a sewage pumping station or any of its components, which under any conditions might cause contamination of the potable water supply. Potable water supply piping to a sewage pumping station shall be equipped with an acceptable backflow prevention device that is selected and installed according to Section 42.9 of the Ten State Standards, or Section 3.5 of TR-16.

2. Types of Pumps

Categories of sewage pumping units include submersible pumps, dry-pit pumps, pneumatic ejectors, vertical wet-pit pumps, suction-lift pumps and systems, and airlifts.

Submersible pumps shall be readily removable and replaceable without the need for personnel to enter the wet well and without interrupting the normal operation of the other pump(s). Vortex-type, open impeller, and cutter/grinder pumps are acceptable. Submersible pump stations shall meet the requirements outlined in Section 44 of the Recommended Standards for Wastewater Facilities, 2004 Edition, or Sections 3.2.3.3 and 3.6.2 of TR-16, except where alternative collection systems are used, see Section C.4 Effluent Sewers.

Underground pump station structures constructed of steel shall be coated with an acceptable corrosion resistant material. The structure should be supplied with two (2) properly sized anodes for cathodic protection to be buried on opposite sides of the structure and electrically connected to the structure by heavy copper wire. To prevent corrosion, connectors should be compatible with the type of wire used. Pneumatic receiver pressure vessels shall be rated for 150% of the maximum pressure achievable in the station.

Suction-lift sewage pumping stations shall meet the requirements outlined in Section 43 of the Ten State Standards, or Section 3.2.3.3 of TR-16. The design should be confirmed with the pump manufacturer.

3. Alarm Systems

Alarm systems should be provided for all pumping stations. The alarm shall be activated in cases of power failure, pump failure, unauthorized entry, high level, pump failure or other cause of pump station malfunction. Stations which are not visited daily and are not equipped with running-time meters shall signal alarm upon operation of the lag pump or spare pump. If the alarm system is connected via phone-line to a remote location (remote telemetry), a separate alarm shall be provided to signal failure of the communication link.

4. Emergency Operation Provisions

Provision for emergency operation shall be made whenever there may be a possibility of discharge other than through the force main. Emergency procedures include a second source electrical supply, a portable pump or generator and adequate overflow tankage. Where the wet well non-working or reserve volume is inadequate for emergency periods, consideration should be given to an easy bypass connection in the piping to connect an emergency pump. Time delay and operator requirements must be considered when planning emergency options.

5. Overflows

The provision of a wet well overflow should be evaluated and consideration should be given to an adequately sized overflow/detention tank, which shall empty to the wet well when pumping operations resume. No discharge onto the ground shall be permitted. A wash down system must be provided for the detention tank.

6. Dry Wells

For dry-pit sewage pumping stations, consideration should be given to providing pumps with a watertight motor drive.

Suitable and safe means of access shall be provided to dry wells (and to wet wells) containing either bar screens or mechanical equipment. Access hatches shall be provided with handgrips. They shall be located

directly over ladders or manhole steps and shall be equipped with two hold-open devices.

Personnel shall be provided adequate positive or forced-air ventilation in dry pits or spaces, which require entry, and shall meet the minimum ventilation requirements in Section 42.7 of the Ten State Standards, or Section 3.3 of TR-16. The method of meeting ventilation and air flow requirements will be dependent upon the type of pumping station, configuration, location, and other pertinent parameters. All intermittently operated ventilation equipment shall be interconnected with the lighting system for the space, as a safety feature.

7. Wet Wells

The wet well size, configuration, and control setting shall be such that heat build-up in the pump motor due to frequent starting, and septic conditions due to excessive detention time, are avoided. Generally, a holding period of between 10 and 30 minutes for the maximum design flow is recommended.

If it is judged that grit will be a problem, pumps for raw sewage should be preceded by grit removal equipment. Where it may be necessary to pump sewage prior to grit removal, the design of the wet well should receive special attention and the discharge piping shall be designed to prevent grit settling in pump discharge lines of pumps not operating.

All electrical splices, junction boxes, or connections for sewage wet wells shall be designed and installed in strict conformance with the latest edition of the National Electric Code, NEC. Structures shall be classified as to the hazard of explosion or ignition. If a hazard classification cannot be determined, such structures shall be considered Class 1 Group D. All electrical wiring, connections, devices, equipment and enclosures within or connected to classified locations shall comply with the appropriate sections of Chapter 5 of the National Electrical Code. All submersible pump motors should be compliant with TR-16 Section 3.6.2 Submersible Pump Motors. All control panels should be NEMA-4 rated for weather, water and dust resistance, or NEMA 4X-rated for corrosive environments (in addition to the above conditions). All control panels should have a thermostatically-controlled heater to prevent condensation.

On smaller installations where an operator is not onsite at all hours, remote telemetry is recommended for pump chambers not equipped with submersible electrical connections.

Level controls (float bulbs, bubbler tubes, wires, transducers, etc.) shall be located so as not to be unduly

affected by turbulence from incoming flows and pump suction. In stations with duplicate units, provisions shall be made to automatically alternate the pumps in use, and consideration should be given to the use of running-time meters.

Covered wet wells shall have provisions for ventilation or air displacement to the atmosphere such as an inverted J-tube or similar device. If personnel are required to enter the wet-well for maintenance purposes, the wet well shall meet the minimum requirements in Sections 42.6 and 42.7 of the Ten State Standards, or Sections 3.2.4 and 3.3.2 of TR-16.

C.4. Effluent Sewers

Septic Tank Effluent Pump/Gravity (STEP/STEG) sewers will be conveying anaerobic septic tank effluent which will be corrosive and odorous. Pipe, pumping and treatment hardware should be manufactured of appropriate materials to withstand or control these adverse conditions.

ASTM designated materials include ABS (ASTM F-628), PVC (ASTM D-2729), high density polyethylene (HDPE), vitrified clay, or fiberglass/polyester composite (ASTM D-3754). Piping of other ASTM designations may be allowed provided that it can be shown to be adequate both structurally and chemically for the proposed conditions.

Table C-2 compares several small diameter sewer systems with conventional gravity sewers. For further information, see the WEF *Alternative Sewer Systems*, Manual of Practice No. FD-12 (2008); or other states' guidance.

C.4.a. Septic Tank Effluent Gravity (STEG) Sewers

Septic tank effluent gravity (STEG) sewer systems are small diameter gravity (SDG) sewers conveying partially treated wastewater. STEG/SDG systems *generally* maintain a continuous positive grade to convey partially treated wastewater to a centralized location for further treatment. For further information, see the WEF *Alternative Sewer Systems*, Manual of Practice No. FD-12 (2008), or other states' guidance.

Small Diameter Gravity Sewer Pipe Sizing

While insufficient empirical data has been generated to demonstrate the absolute, unqualified acceptability of slopes less than those below, a system of small diameter gravity sewers conveying settled sewage may be approved if designed and constructed to give mean velocities, when flowing full, of not less than 1.0 foot per second based on Manning's formula using an "n" value of 0.013. The purpose of

this is to ensure that the saving to be gained by installing the sewers at these flatter slopes is sufficient to justify the potential risk involved. It is imperative that adequate septic tank maintenance procedures be established to ensure the retention of solids in the septic tanks. While important for flow at 1.5 feet per second, it is especially important below that velocity.

Table C-2. Comparison of Collection Alternatives

(TABLE 1.1 of WEF MOP FD-12, 2008).

Issue	Effluent sewer		Grinder-pump pressure	Vacuum sewer	Conventional sewer
	STEG	STEP			
Annual inspections—suggested preventative maintenance	Yes	Yes	Yes	Yes	No
Septage pumping from on-lot septic tank, as required	Yes	Yes	No	No	No
On-lot electrical connection Required	No	Yes	Yes	No	No
Discharge wastewater characteristics					
Strength (due to ...)	Low STE	Low STE	High Undiluted	High Undiluted	Medium Diluted
Flow (due to ...)	Low Low infiltration and inflow	Low Low infiltration and inflow	Low Low infiltration and inflow	Low Low infiltration and inflow	High High infiltration and inflow
Corrosion/odor potential (due to ...)	High Sulfides from septic tank	High Sulfides from septic tank	Low to high f (line length)	Low Aeration in volatile solids	Low to high f (force mains)
FOG (mg/l)	Low	Low	High	High	Medium
Terrain effects:					
Discharge above source	No	Yes	Yes	Yes	Yes, with pump stations
Discharge below source	Yes	No	No	Yes	Yes

Undulating terrain	Yes, use a mix of both	Yes with CS	Yes	Yes, with GP
Discharge to:				
Conventional sewer	Yes	Yes	Yes	Yes
(Note)	Extra design concerns			
Biological treatment	Yes	Yes	Yes*	Yes*
Constructed wetlands	Yes	Yes	Yes*	Yes*

* Pretreatment needed.

Small Diameter Gravity Sewer Minimum Pipe Slopes

The following are the minimum slopes which should be provided for systems to be considered under these criteria:

Sewer Size	Minimum Slope in Feet Per
	<u>100 Feet (m/100 m)</u>
4 inches (10 cm.)	0.21
6 inches (15 cm.)	0.12
8 inches (20 cm.)	0.08

The smallest recommended lateral, collector, or main pipes is 4 inches for STEG systems.

STEG systems can be designed with variable or inflective vertical and horizontal gradients, which can minimize property damage resulting from construction excavation. With the use of inflective grades, both positive and negative, there may be some pipe sections that are permanently full of septic tank effluent. It may be necessary for some facilities to have pumps and check-valve or backflow-prevention device to provide access to, and prevent backflow from, the collection system. Air release valves or ventilated clean-outs may be necessary at the high points of the main to prevent air-binding.

Septic tank effluent is considered partially treated wastewater, or “settled sewage.” The cost for the design and construction of treatment works which collect and convey settled sewage for additional treatment (either centralized or distributed) should be compared to those of conventional gravity sewerage. The costs should be determined within the confines of the design criteria given below. Septic tank effluent – pump (STEP) and/or septic tank effluent – gravity (STEG) system proposal costs are often less when associated with a small community, or urban areas with shallow bedrock or seasonally high groundwater.

Neither the Ten State Standards, nor TR-16 stipulates minimum slopes for pipes diameters less than eight (8) inches since 8" is the minimum diameter sewer pipe allowed. The minimum slopes given are only for pipes conveying *raw sewage*, not *settled sewage*.

Design Factors

- All wastewater and gray water sources shall be served, including sources located within basements
- Watertight septic tanks shall be sized, constructed and maintained per Section D.6
- Septic tank effluent filters shall be used per Section D.7
- Check valve on each building service connection if back flooding is possible
- Net-positive gradient mains are required (variable grade segments allowed)
- Minimum velocity of 1 ft/s
- Design flow should be carried at a depth of no more than 0.50 of pipe diameter;
- Flushing connections are required;
- The use of 90-degree drop manholes should be avoided in sewer lines carrying septic tank effluent. A 45-degree sloping drop to a pump station should be used to avoid odor release by turbulence.

C.4.b. Septic Tank Effluent Pump (STEP) Sewers

In Septic Tank Effluent Pump (STEP) systems, effluent from a user's septic tank is pumped through a small diameter pressure pipe system that either transfers the effluent directly to treatment components or to a central collection point for transfer to treatment components. The effluent pump is located in a pump chamber either inside the tank or next to the tank in a dedicated pump-chamber. The effluent flows into the chamber and activates a sensor to raise it to a determined level. The effluent is then pumped out until the liquid level is reduced and a lower level sensor shuts the pump off. Typically, there is an additional sensor that activates an alarm for pump failure or high liquid level in the chamber.

Effort should be made to establish septic tank maintenance districts for cleaning procedures in conjunction with the facility planning and design phases, recognizing that the key to successful operation of these systems is retention of solids in the septic tank. Sewer use ordinances and/or local health codes should, therefore, include adequate provisions to ensure proper operation, use, connection to, and construction of such small diameter sewers.

Table C-3. Comparison of Commonly Used Pipe Materials for STEP/STEG Systems

(Table 4.3 of WEF *Alternative Sewer Systems*, MOP FD-12, 2008, and *Guidance Manual for the Evaluation of Effluent Sewer Systems* by Electric Power Research Institute (EPRI), et al., 2004).

	PVC			Polyethylene (HDPE)	
	Solvent weld	Elastomeric	Drain, waste, vent		
Commonly used types	Iron pipe size Schedule 40 10-cm (4-in.) diameter SDR 21 5-cm (2-in.) diameter	Iron pipe size SDR 21	Sewer dimensions SDR 35		SDR 11 (1100 kPa [160 psi]) SDR 9 (1380 kPa [200 psi])
Typical uses	STEG or STEP; Service lines; Small mains	STEG or STEP; Mains	STEG or STEP; Building sewers only (recommended)		Water crossing; Borings; Cold zone mains
Advantages	Readily available	Easy to join; Allows for thermal movement	Readily available		Seamless; Allows continuous trenching
Disadvantages	More difficult to join in cold temperatures, wet conditions, dust, and mud	Joining rings may not be available for small sizes	Thin wall (Not recommended for pressurized mains)		Cost of heat-fusion joining equipment
Relative costs	Medium	Medium	Low		Medium

STEP System Design Factors

STEP sewer systems should be designed in accordance with the standards given below or in a design guide such as the Water Environment Federation's *Alternative Sewer Systems*, MOP FD-12, 2008, or other states' guidance. Piping in STEP systems can be laid to follow grade, which make the systems suitable for areas with undulating ground, locations where rock is close to the surface or where there are

high groundwater levels. STEP sewer systems should be designed in accordance with the following:

- All components of the effluent pump system must be resistant to corrosion
- Because the effluent is relatively free of solids, the building service lateral can be as small as 1.25" in diameter
- All wastewater and gray water sources shall be served, including sources located within basements
- 24-hour storage volume, duplex pumps, or emergency generator compatible;
- Watertight septic tanks shall be sized, constructed and maintained per Section D.6
- Septic tank effluent filters shall be used per Section D.7
- Isolation valve and redundant check valve shall be used at each building service connection
- Typical 2" pipe diameter for force mains
- Tracer wire shall be installed with all force mains
- Isolation valves placed at intersections
- Minimum velocity of 1 ft/s
- Minimum Hazen-Williams $C = 120$
- All pumps shall be non-overloading over their entire curve
- Flushing connections spaced every 500 – 1000 feet along mains
- Air release valves shall be installed at high points; odors may have to be controlled
- Careful placement of air release valves shall be considered when incorporating centrifugal pumps

C.4.c. Pump Selection

Effluent Sewer Systems

Wastewater flow for single-family dwellings typically ranges from 40 to 60 gallons per capita per day (gpcd); 50 gpcd is a commonly used design parameter and is the value used in calculations here.

Design flows for effluent sewers should be taken from **Table B-3**. The flow rate, when pumping from a tank, should be slow enough to allow maximum settlement of solids, adequate fat, grease and oil retention, and an efficient power rating, but not so slow that the pump's run-time is excessive.

The ideal discharge rate for a single-family residential effluent sewer pump is 5 gpm. Where system pressures are low, it is common to use flow controllers to keep the discharge rate below 9 or 10 gpm.

High-quality submersible turbine type pumps are common in STEP systems because of their extreme resistance to corrosion, high cycle life (250,000), light weight (30 lbs.) and their ability to pump 5 to 20 gpm at discharge heads greater than 200 feet. Another advantage is their ability to operate for extended periods in the “no discharge” condition, or at heads greater than the maximum “shut-off” head.

Headloss

For general purposes, the total head of a pump system is the sum of the static head plus friction losses in the system. The static head is the difference in elevation from the surface of the pumped liquid to the highest point in the system. The friction losses are from the pump to the final discharge point.

In designing small diameter septic tank effluent sewer systems, additional consideration should be given to protect against freeze-ups due to construction at shallow depths susceptible to frost penetration. The piping from septic tank outlets is normally shallower than that of conventional gravity building sewers, which is often at or below the basement floor level.

Where velocities are greater than 15-feet per second are expected, special provision shall be made to protect against displacement by erosion or shock.

C.4.d. Septic Tank Effluent Manholes and Cleanouts

Hydraulic cleanouts are normally considered superior to manholes for pipes conveying settled sewage; however, manholes may be necessary at the junctions of two or more small diameter gravity sewer lines.

Specification of watertight manhole covers at such locations is desirable to prevent grit from entering the system. Manholes should be located where they will be least susceptible to damage from snowplowing.

Gases generated in the septic tank under anaerobic conditions may be toxic; consequently, the creation of turbulence, which enhances the release of gases from the septic tank effluent, should be avoided through proper design practices. *Accordingly, the use of 90° drop inlet manholes is discouraged, and the use of a 45° sloping drop to a pump station should be used to avoid odor release by turbulence.*

C.5. Pressure and Vacuum Sewers

Construction Factors for Pressurized Sewers

A sand bed, or other suitable bedding material, shall be prepared at least 4-inches deep but not less than one (1) pipe diameter. The bedding shall be smooth and compacted prior to pipe installation.

The excavation shall be backfilled to a depth of 18" above the pipe with sand, or other suitable bedding material. The bedding material shall contain no rock greater than 1" in diameter. Native material may be used for the remainder of the backfill.

Materials

Many types of pipe may be used for pressure sewers. Maximum benefit can usually be achieved with non-metallic materials such as polyethylene, fiberglass reinforced plastic, and PVC. ASTM D-2241 is the standard for PVC pressure pipe (SDR pipe).

C.6. Conventional Force Mains

Design Factors

Headloss determinations should be made using the Hazen-Williams Coefficients given in **Table C-4** below.

At maximum design flow, a sewage velocity of at least 2 feet per second shall be maintained in the force main. Consideration should be given to achieving a solids pick-up velocity of 4 feet per second. Velocity calculations must be based on the actual inside pipe diameter.

Table C-4. Recommended Hazen-Williams Coefficients for Sewer Pipe

Pipe Description	Hazen-Williams Coefficient, C
Plastic	150
Concrete/Cement	120
Cast Iron	100
Welded Steel	100
Riveted Steel	90

Force mains in systems that are located above the frost line and operate on a seasonal basis shall be provided with draining capability to avoid freezing problems. The Hazen-Williams Coefficient should be significantly reduced for these systems.

In general, 3-inch diameter pipe shall be the smallest used for **raw sewage force mains**. However, use of grinder pumps or similar equipment may allow use of smaller pipe. These instances will be reviewed on an individual basis. See Section C.7.a. Grinder Pump (Low Pressure) Sewers.

It is recommended that automatic air-relief valves be installed in manholes at high points in the force main to prevent air locking. Long sewage-type valves with hose connections for regular flushing and freeze prevention should be used. Consideration will be given to alternate proposals with proper substantiation.

Where the force main is relatively short, consideration should be given to installing two force mains and eliminating valves.

Pressure tests shall be made only after the completion of backfilling operations and at least 36 hours after the concrete thrust blocks have been cast. All tests shall be conducted under the supervision of the design engineer.

The duration of pressure tests shall be one hour, unless otherwise directed by the engineer. The test pressure shall be no less than 50psi, with a recommended pressure of 2-1/2 times the maximum system operating pressure.

The pipeline shall be slowly filled with water. Before applying the specified pressure, all air shall be expelled from the pipeline by making taps at the point of highest elevation. The specified pressure, measured at the lowest point of elevation, shall be applied by means of a pump connected to the pipe in a manner satisfactory to the design engineer. After completion of the test, the taps shall be tightly plugged.

Termination of Force Mains

Force mains should enter a gravity sewer at a point no more than 2 feet above the flow line of the receiving manhole. Force mains and pressure sewer trunks shall terminate in manholes using the following construction procedures:

1. The discharge shall be to the bottom of the manhole, in line with the flow if possible,
2. Where piping must be installed to bring the discharge to the bottom of the manhole, the pipe shall be adequately braced to prevent movement, shall be vented on the top, and shall allow access to the force main for cleaning purposes.

Gauges

Installation of a pressure gauge calibrated in feet of water and equipped with a diaphragm seal, glycerin fill, snubber, and a spring-loaded shut-off valve that is located on the force-main after the valves should be considered.

C.7. Small Diameter Sewers (minimum 2" to 4" diameter mains)

C.7.a. Grinder Pump (Low Pressure) Sewers

Grinder pump systems macerate raw wastewater and pump the material, under pressure, through small diameter pipes to treatment components.

Design Factors

Grinder pump sewer systems shall be laid out in a branched or tree configuration to avoid flow-splitting at branches. The required pipe size shall be determined on the basis of three principal criteria, namely:

1. Velocities adequate to assure scouring should be achieved,
2. Size should be sufficient to handle the required flow rate, and
3. Head loss should not exceed pumping pressure capabilities.

The basis of cumulative flow within the system shall be used. Design shall be for peak sewage flow rates and negligible infiltration. A velocity of two to five feet per second must be achieved at least once and preferably several times per day based on design flows.

- All wastewater and gray water sources shall be served, including sources located within basements
- Watertight Low Pressure Sewer pump vaults are required

- 24-hour storage volume or emergency generator compatible
- Minimum 1.25" building service laterals
- Isolation valve and redundant check valve at each building connection
- Minimum 2" pipe diameter for force mains
- Tracer tape/wire shall be installed with all force mains
- Isolation valves shall be placed at intersections
- Minimum velocity of 2 ft/s
- Minimum required Hazen-Williams $C = 120$
- Flushing connections are required
- Air-release valves shall be installed at high points

Grinder Pump System Arrangement

All pressure sewer pipes shall be installed at a depth sufficient to protect against freezing and mechanical damage.

Attention must be given to the necessity for providing automatic air-release valves at major changes of slope. Air-release devices are required when the liquid flow velocity is insufficient to purge air bubbles. Pressure and/or flow-control valves shall be installed at the end of all critical surge pipe-runs in order to maintain a full-pipe system and eliminate pump station flooding or plant washout.

The standard clearances for water/sewer separation/crossings apply to pressure systems (Section C.8).

Grinder Pump System Pressures

Operating pressures in general should be in the range of 40 to 60 psi and shall not exceed 60 psi for any appreciable amount of time. Provisions shall be made in both the system and grinder pumps to protect against the creation of any long-term high-pressure situations.

Service Connections

Building service connection laterals from individual grinder pumps to the collectors shall be a minimum of 1-1/4 inch PVC or HDPE pipe, and include a full ported valve (such as a corporation stop or "u" valve) located in the service line to isolate the pump from the main. Check valves specifically suited to

wastewater service should be provided at or near the pump, and at a convenient location in the pressure service line before it enters the main.

Cleanouts and Fittings

Pressure systems shall have cleanouts at intervals of 500 to 1000 feet of straight pipe runs (over 500 feet only where known O&M procedures can effectively clear the lines), at major changes of direction, where one collector main joins another main, and at terminal ends of pressure mains. Access for cleaning shall be provided at the upstream end of each main branch.

Cleanouts shall include an isolating valve, and a capped sanitary-T or sanitary-Y fitting located on each side of the isolating valve and pointed both upstream and downstream for access during maintenance procedures.

All appurtenances and fittings shall be compatible with the piping system, and shall be full-bore with smooth interior surfaces to eliminate obstructions and keep friction loss to a minimum.

Pumping Equipment

The pumping equipment shall be designed in a manner appropriate to wastewater service, and be manufactured of corrosion resistant materials. In addition it shall meet all applicable safety, fire, and health requirements arising from its intended use in or near residential buildings.

Proper system design and installation shall assure that each grinder pump will be able to adequately discharge into the piping system during all normal flow situations including peak design flow. During peak design flows, combined head losses (static, friction, and miscellaneous) shall be maintained below the recommended operating head of any unit on the given path of flow.

The pumps shall have a head capability high enough to operate efficiently over the entire range of conditions anticipated in the system. Normally this will consist of a fixed static-head component dependent on pump elevation with respect to the discharge point. The head capacity design point should be not more than 85 percent of the maximum attainable pressure.

The units must be capable of operating under temporary loads above the normal recommended system design operating pressure without a serious reduction of flow or damage to the motor. The pump should be of flooded-suction design to assure that it will be positively primed. The pressure-sewer system shall contain integral protection against back siphonage.

Outside installations are preferable, and should be located at least ten feet from the building in an area readily accessible to service personnel outside installations shall be provided with an access from the surface which is suitably graded to prevent the entrance of surface water, and equipped with a vandal-proof cover for safety. Inside installations must be examined for freedom from noise, odors, and electrical hazards. Both free-standing and below-the-floor type installations are acceptable.

The electrical portions of non-submersible grinder/macerator pumps must be protected against entrance of surface water. This may require that a motor "breather" be run from the interior of the motor and control compartment to a protected location higher than the maximum anticipated water or snow level. Waterproof factory-installed wiring and tamper-proof access covers on wiring compartments are required.

The grinder pumps shall operate at a noise level sufficiently low to be acceptable for installation inside a residential building. Generally, this should be no louder than other motor-operated devices normally found in homes (furnace blowers, sump pumps, etc.).

The grinder pump equipment shall comply with National Electrical Code and applicable local electrical inspection bureau requirements.

Grinder Pump Types

Both stable-curve centrifugal and progressing cavity semi-positive displacement pumps may be used in pressure sewer systems.

The stable-curve centrifugal, a pump having maximum head at no flow, may be considered for its ability to compensate with reduced or zero delivery against excessive high pressures, and the ability to deliver at a high rate during low flow situations in the system, thus enhancing scouring during low flow periods.

The progressing cavity semi-positive displacement pump may be considered for its relatively constant rate of delivery. The semi-positive displacement pump has no significant increase in delivery against low-flow conditions.

Grinder

The grinding pumping equipment must include an integral grinder capable of handling any reasonable quantity of foreign objects which customarily find their way into building drainage systems as a result of carelessness or accident on the part of building occupants without jamming, stalling, overloading, or undue noise. The particle size produced by the grinder must be small enough to insure that the processed solids will not clog the grinder, pump, or any part of the discharging pipe system. The grinder shall be of a configuration to provide a positive flow of solids into the grinding zone.

Open shafts shall not be exposed in the raw waste passageways, since this will cause cloth, string, etc. to become wrapped around the blades or shaft.

Grinder Pump Tank

The pump tank must be made of corrosion-resistant materials, which are suitable for contact with sewage and direct burial below grade without deterioration over the projected lifetime (at least 20 years).

The pump tank shall be furnished with integral level controls which reliably turn the pump on and off at appropriate and predictable levels. The level control shall be as trouble-free as possible with little care required for proper calibration. Float-type, pressure-type, or probe-type switches are acceptable. An alarm unit with visible and audible alarms shall be provided on a separate electrical circuit or a self-contained power supply to indicate pump failure.

The tank shall be of a 50-gallon minimum capacity and be able to accommodate normal peak flows without exceeding its peak flow capacity. The volume between the on and off levels in the tank should be based on a sensible compromise between excessive unit operation and efficient removal of raw sewage into the system.

The tank shall be capable of accommodating connection to all normal building drainage piping systems. This would include four-inch sizes of PVC, cast iron, copper, vitreous clay, and asbestos-cement pipe.

The geometry of the tank bottom, and the pump suction currents generated when the grinder pump is in operation, must be adequate to scour solids from the bottom of the tank so that there is no significant long term accumulation of settleable solids on the tank bottom.

In areas in which the groundwater table is high, tanks should be securely anchored to avoid floating.

The tank shall be vented so that air space above the wastewater is always at atmospheric pressure. Separate venting may be necessary, but normally the influent piping connected to the building drain will provide adequate venting. Separate venting would be necessary, for instance, if the building has a building trap and the grinder pump tank is exterior to the building trap, and the building drain/vent system will not vent the grinder pump tank.

Power Outages

Provisions must be made for periods of power failure. Alternatives are:

1. Dependence on built-in storage of tank and associated gravity piping,
2. Provision of additional storage capacity where power outages occur frequently (24-hour storage capacity is recommended) and
3. Provision of a mobile generator to connect to each household for a short term during an extended outage.

Service

A 24-hour repair-time either by replacement or repair is strongly recommended. DEC suggests that this can only be achieved with adequate certainty if the grinder pumps are owned, operated, and maintained by the municipality. Also, grinder pump units should be installed on property to which the municipality or Sewage Works Corporation has clear title or right-of- access for maintenance purposes (see Section C.10 Management, Operation and Maintenance).

C.7.b. Vacuum Sewers

Vacuum sewers may be considered an alternative to gravity or pressure sewers where the terrain is flat and either a high groundwater table or unsuitable soils are present, or where rock excavation is necessary. Typically, wastewater from more than one property is connected to a vacuum sewer system.

Design Factors

Design shall be for an operating vacuum range of 15 to 22 inches of mercury at mean sea level. Adjustments must be made for the altitude of the site.

Solvent-welded or vacuum suitable bell and spigot joints must be used on all plastic piping. Double O-ring slip joints should be considered to provide for temperature stresses. Solvent-welded pipe shall be checked for temperature expansion allowance before covering. Bituminous-fiber pipe (Orangeburg) shall not be used.

Gravity sections of vacuum sewer systems shall have a minimum slope of 0.20 percent irrespective of pipe size. Minimum pipe diameter shall be three inches (3"). If collection system piping of less than four inches (4") is used, special attention must be given during construction to providing adequate support to prevent flexing of the pipe.

Grease interceptors may be necessary to prevent malfunctions. Also, to reduce potential for sewer clogging, it is recommended that two 45-degree bends be used in lieu of 90-degree bends, or if not possible use long sweep 90 degree bends.

Shut-off valves shall be provided at every branch connection, and at intervals no greater than 2000 feet on main lines. Gate valves and butterfly valves shall not be used. Isolation valves shall be provided between the vacuum collection tank, vacuum pump(s), influent line, and raw sewage discharge pumps. Access points equal to the line-sizes shall be provided at the end of every main or branch line, and at changes in line diameter.

Building sewers shall be constructed in the same manner as main lines. Air vents should be located adjacent to the building and be protected from freezing, snow, and flooding.

Vacuum Valves

Vacuum value pits should be designed to prevent entrance of water, although vacuum valves should be capable of operating when submerged under water or ice conditions. Electronically controlled vacuum valves should be avoided. A 24-hour repair-time either by replacement or repair is strongly recommended.

The use of exterior valve breather tubes and vents should be avoided. If exterior vents must be used, they are to extend above maximum snow level and have moisture traps. Moisture traps should be provided on all breathers.

Central Collection Station

A vacuum pump shall be designed not to cycle more than six times in one hour during average daily flow conditions. A minimum running time of one minute per cycle is required. A standby pump shall be provided to handle peak loadings, as well as an emergency backup generator in case of power outage.

C.8. Water/Sewer Separation

For new construction, sewers should be laid at least 10-feet horizontally from any existing or proposed water line, measured edge-to-edge.

For replacement, retro-fitting, or where local conditions prevent a lateral separation of 10-feet, a sewer may be laid closer than 10 feet from a water main if:

- a) It is in a separate trench with an 18-inch vertical separation (water main invert 18-inches above the crown of the sewer), or
- b) It is in the same trench with the water main located at one side on a bench of undisturbed earth (water main invert 18-inches above the crown of the sewer).

Whenever sewers must cross under water mains, and the sewer cannot be buried to meet the above vertical separation requirement, the water main shall be relocated to provide this separation, or the sewer line constructed of slip-on, or mechanical-joint cast-iron pipe, or PVC pressure pipe for a distance of 10 feet on each side of the water line and be pressure-tested to assure water-tightness. At least ten feet of separation must exist between the point of crossing and joints in the water line.

Water mains may cross under sewers if the sewer is adequately supported to maintain line and grade, and the 18-inch vertical separation is maintained between crown and invert as described above, in Section 38 of the Ten State Standards, and in Section 2.8.3 of TR-16.

There must be no physical connection between a public or private potable water supply system and a

sewer, or appurtenance thereto, which would render possible the passage of any sewage or polluted water into the potable water supply. No water pipe shall pass through or come in contact with any part of a sewer manhole.

C.9. Creek Crossings

Permits are required for crossing or working adjacent to certain streams, as outlined in Section A. In the event that no stream-crossing permit is required, the crossing shall be made in such a manner as to minimize disturbance of the streambed and the impact on stream water quality as described in Section C.8 above, in Sections 35, 36 and 37 in the Ten State Standards, and Sections 2.5, 2.6 and 2.7 of TR-16.

C.10. Management, Operation and Maintenance

The requirement to form a Sewage Disposal Corporation (SDC) applies to the permitting of all privately owned sewage treatment plants, unless a variance is granted. The terms *Sewage Disposal Corporation* (referenced in Part 750-1.6(f)) and *Sewage Works Corporation* (referenced in Article 10 of Transportation Corporation Law) are considered equivalent in meaning. The formation of a SDC may be required in accordance with the following regulation:

- ❖ 750-1.6(f) - *A permit for a sewage disposal system or approval of a sewer extension serving or intended to serve more than one separately owned property shall be issued only to either a governmental agency, municipality, or sewage disposal corporation formed and regulated pursuant to Article 10 of the Transportation Corporations Law. The commissioner may, on written application, grant a variance from this provision in a particular case, subject to appropriate conditions, including bonding requirements, where such variance is in harmony with the general purposes and intent of this Chapter.*

The New York State Transportation Corporations Law (TCL), Article defines *Sewage Works Corporation* (SWC):

- ❖ TCL, Article 10, Section 115(1) – *a corporation heretofore or hereafter organized to provide a sewer system as hereinafter defined for the disposal of sewage, through an established system of pipe lines, treatment plants and other means of disposal, and which erects, operates, maintains and performs other necessary acts incidental thereto, disposal systems for sewer areas formed within towns or villages and other municipal areas of the state.*

Information on which wastewater treatment systems require what level certified operator and WWTP

operator requirements are in Section J.

C.11. Instruction Manuals

The equipment for new conventional pump stations, STEG and STEP pump chambers, grinder pump or vacuum sewer system pump stations, etc. must be furnished with complete detailed wiring diagrams, suggested piping installations, and detailed instructions, for use by the contractor at the time of installation, and later by the owner of the system. A complete instructional manual must be provided to the owner of the system. Manufacturer's requirements and any conflicts with these design standards should be discussed at the pre-application conference (Section A.2).

D. Flow Measurement, Pretreatment and Appurtenances

D.1. Introduction

Wastewater pretreatment varies with site conditions and the proposed type of final treatment and discharge system. Pretreatment components remove contaminants from the wastewater to provide an effluent that can be accepted and treated by downstream components and discharged to groundwater or a surface water body. Greater levels of treatment may be needed to meet stream requirements and permitted discharge limits, or to assure the viability of a soil-based treatment and dispersal system. Combinations or multiple units of the listed pretreatment methods may need to precede the treatment methods listed in Sections E, F and G. Pretreatment units should remove most settleable solids and floatable fats, oils and greases (FOG).

When designed in accordance with the criteria presented, pretreatment tanks (i.e. septic tanks and grease interceptors), and flow measurement, flow control and septic tank effluent screening devices will allow the biological, and physical-chemical treatment processes discussed in Sections E, F and G to operate more efficiently, increase their operating life, and capability of achieving either non-intermittent, or intermittent stream limits (limits given in Tables B.4-A and B.4-B).

Substantiation of the ability of the proposed treatment system to achieve the needed level of treatment is the responsibility of the design engineer. For some commercial wastewaters alternative treatment techniques such as physical-chemical treatment (Section G) may be more appropriate than biological treatment. Requirements for septic tanks, intermittent sand filters, and physical-chemical treatment methods are included in this manual. Criteria for the other treatment methods are presented in the Ten State Standards, and TR-16. Except for requirements and allowances listed in these Standards, which may be considered to supersede them, the Ten State Standards and TR-16 are referred to and shall be followed.

Lack of description or criteria for a particular process is not intended to suggest that such a system should not be used, but only that consideration by the reviewing agency will be on the basis of information submitted with the design (See Section H). It is incumbent on the design engineer to fully demonstrate that the process or equipment is capable of achieving the treatment objectives outlined herein.

D.2. Preliminary Treatment Devices

Coarse screens, trash racks, or coarse bar racks, should be provided where sewers deliver wastewater to a treatment facility to protect pumps and other equipment. Comminutors may be used instead of screening devices to protect equipment where stringy substance accumulation on downstream equipment will not be a substantial problem. Criteria for these devices are presented in the Ten State Standards, and TR-16.

Wastewater from a building sewer or a small diameter sewer pipe flowing into a septic tank does not require coarse screens or trash racks. A *trash tank* may be used prior to an Aerobic Treatment Unit or other Enhanced Treatment Unit (ETU), or may be an integral part of the unit.

D.3. Flow Equalization

Flow equalization should be provided for all treatment modes with the exception of septic tanks, intermittent sand filters, and lagoons. The design of any other treatment unit shall consider minimum and peak design flows when determining the equalization volume needed.

Flow equalization can be provided by the storage volume in:

- A. a dedicated equalization (surge) tank,
- B. a pump, siphon or dosing tank, or
- C. a trash tank (prior to or integral to the design and manufacture of an ETU). Depending on the facility, site or receiving water, and the size of the integral trash tank, the reviewing engineer may require a septic tank prior to the ETU.

The storage volume should be equal to 24-hours times the daily design flow rate, or the flow rate in gallons per minute over the operating time of the significant delivery period of the facility.

D.4. Flow Measurement

Maximum and minimum flows and instantaneous peak flow variations are necessary factors in properly sizing and designing system components. Therefore, some means of wastewater flow measurement or estimation shall be provided for all wastewater treatment facilities as follows:

- A. All WWT systems with a design flow over 25,000 gpd are required to install a flow

measuring device (i.e. weir, flume, etc.) with the ability to record & totalize (with a meter) the daily discharged flow.

B. For smaller plants (more than 5,000 and less than 25,000 gpd), a flow measuring device (i.e. weir, flume, etc.) is required to obtain at least a manual instantaneous effluent daily flow with the ability to install a continuous recording device, if necessary. A totalizing water meter may also be acceptable for these size plants.

Time-lapse meters or dose counters may be used where the waste is pumped. Where applicable, the wastewater flow may be estimated by the use of a totalizing water meter on the water supply servicing the establishment(s) tributary to the OWTS.

C. For most septic tank systems with subsurface dispersal of the effluent, flow measurement is not necessary. Flow measurement is strongly recommended when the design flow is 1,000 gpd or greater. Systems with pressure distribution or other dosing device should employ dose counters, or time-lapse meters.

D.5. Fats, Oils and Greases (FOG) Removal and Grease Interceptors

Non-petroleum FOG treatment, intercepting and removal devices include:

- Hydro-mechanical (**Type I**) Grease Interceptors, sometimes combined with:
- Grease Removal Devices (both manual and automatic GRDs) or
- FOG Disposal Systems, and
- Gravity (**Type II**) Grease Interceptors (tanks).

Codes and Jurisdiction

The specification of non-petroleum FOG-laden plumbing fixtures, and grease interceptors (of all kinds) for use on the premises of buildings is the jurisdiction of the Department of State, Codes Division and the NYS Uniform Fire Prevention and Uniform Codes. Type I grease interceptors are required by the Plumbing Code of NYS (PCNYS), and are installed as close to the FOG source as practical. Code Enforcement Officials apply these Codes, which are based on a modification of the International Plumbing Code (IPC). A Type II grease interceptor external to the food service area may also be required due to local conditions.

Public Health Law and SPDES Regulations

Local Health Units review plans and specifications according to the Public Health Law, Part 14 “Food Service Establishments” and the 1984 MOU with NYSDEC, using the DEC Design Standards. Type II grease interceptors may be required by DEC or DOH where onsite wastewater treatment systems are used, and may be required by a sewer use ordinance. NYSDEC issues the SPDES permits for these same facilities under the Environmental Conservation Law and SPDES regulations.

Sewer Use Ordinances

For all grease interceptors discharging to sewers, the local sewer district or authority must be contacted for their requirements for Type II grease interceptors. DEC defers to the PCNYS and municipal sewer use ordinances for all FOG-control devices discharging to municipal sewers. This manual provides design standards for Type II grease interceptor tanks discharging to an OWTS.

Onsite FOG Management

Waste fats, oils and greases (FOG) are generated in various P/C/I facilities including restaurants, bakeries, catering halls, hospitals, nursing homes, schools, correctional facilities, churches and grocery stores. FOG-laden fixtures of kitchen/food service areas of the above facilities shall be served by a separate sewer line and plumbed to a grease interceptor upstream from the septic/aerobic tank (see Figure D-1).

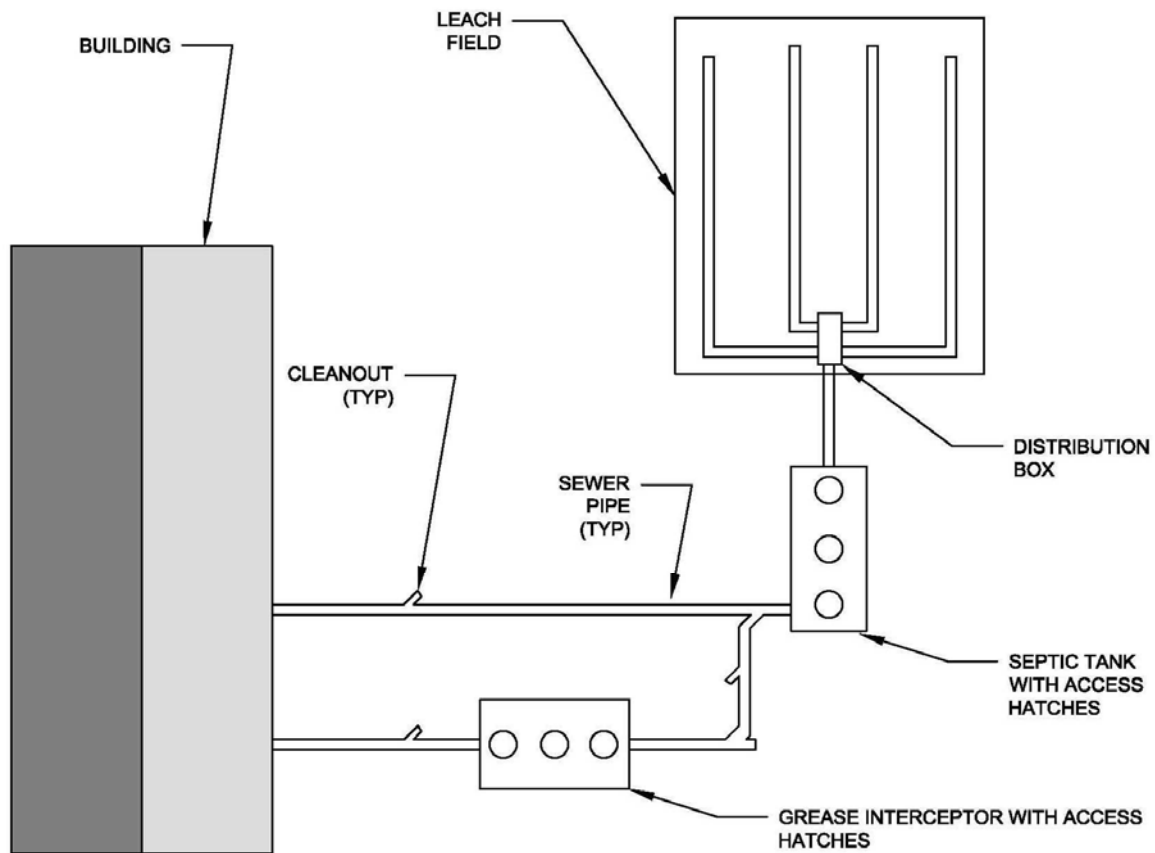


Figure D-1 Grease Interceptor and Septic Tank Building Sewers Discharging to Soil-Based Treatment Systems (STS)

The grease interceptor should be a gravity grease interceptor (Type II), and a Certified Tank, unless a Type I grease interceptor installed per the PCNYS, and its associated O&M plan, is deemed adequate locally.

Several factors can affect the performance of a gravity grease interceptor: wastewater temperature, solids concentrations, retention time, and maintenance practices. Facilities such as grocery stores may have multiple sources of FOG-laden wastewater (bakery, meat dept., deli counter, take-out counter, hot food buffet and in-house eating area). It is incumbent on the design engineer to provide effective FOG-removal systems for individual graywater sources or combined sources within the facility.

The most cost-effective way to minimize the effect of FOG on on-site wastewater treatment components is to implement a FOG Best Management Practices plan. (Refer to O&M subsection). Ground garbage, discharge from ice-machines, and wastewater from on-site linen service must NOT be directed to the grease removal/recovery system but directly to the septic tank.

Hydro-mechanical (Type I) Grease Interceptors

Where the FOG-generating fixtures upstream of a proposed Type I grease interceptor can be identified and sized, the hydro-mechanical interceptor (1) shall be sized according to the Plumbing Code of NYS, (2) shall be plumbed into the FOG-laden piping, and (3) if required due to local conditions and Operation and Maintenance concerns, effluent shall be directed to a Type II gravity grease interceptor. Annual, or more frequent, maintenance is required.

Gravity (Type II) Grease Interceptors

Type II grease interceptors are most often required where an OWTS is used to protect the soil-based treatment and dispersal system. FOG-laden wastewater should flow to the grease interceptor by gravity. The underground interceptor tank shall be placed as shallow as possible to minimize the effect of groundwater pressure on the tank system. The location must be chosen to insure easy access for maintenance, and be installed near the building so the grease does not solidify before reaching it. Maintenance will require a pumper truck.

Capacity of Gravity Greased Interceptors

For FOG-laden discharges from non-residential, single-lot Onsite Wastewater Treatment Systems, or small-scale collection sewers discharging to privately-, commercially-, or institutionally-owned treatment systems, Type II grease interceptors shall be sized using Table D.1 or Table D.2.

Table D.1. Gravity (Type II) Grease Interceptor Sizing Based on Grease-laden Building Sewer Flowing Full

Pipe Size	Max. Flow	Nominal Interceptor Volume (Gallons) (based on 30 minute settling time)
2"	20 GPM	750
3"	60 GPM	1,800
4"	125 GPM	4,000
5"	230 GPM	7,000
6"	375 GPM	11,500

The Table D.1 method assumes that the maximum flow would be a full pipe with gravity flow. Based on standard engineering calculations using *full pipe gravity flow*, with a 1/4" pitch per foot (2% slope) and a Manning's n of 0.012, the maximum flow rate and interceptor volume is given in nominal increments.

Table D.2. Gravity (Type II) Grease Interceptor Sizing Based on Known Drainage Fixture Units (DFUs) and Pipes Flowing Half-Full

DFUs ⁽¹⁾	Flow (gpm)	Pipe Dia.(inches)	% Slope	Interceptor Volume ⁽²⁾
8	10	2	2	500 gallons
21 ⁽³⁾	-	-	-	750 gallons
35	29	3	2	1,000 gallons
90 ⁽³⁾	-	-	-	1,250 gallons
172	44	4	1	1,500 gallons
216	62	4	2	2,000 gallons
307 ⁽³⁾	-	-	-	2,500 gallons
342	80	5	1	3,000 gallons
428	120	5	2	4,000 gallons
576	140	6	1	5,000 gallons
720	190	6	2	7,500 gallons
2112	290	8	4	10,000 gallons
2640	400	10	1	15,000 gallons

Table D.2 Notes:

(1) The maximum allowable Drainage Fixture Units (DFU) plumbed to the kitchen drain lines that will be connected to the grease interceptor. DFU values for specific fixtures are in UPC Table 7-3.

(2) These volumes are based on: the pipe size from the 2009 UPC code Table 7-5, pg.129; Useful Tables for Flow in Half-full Pipes (ref: *Mohinder Nayyar Piping Handbook*, 3rd Edition, 1992). *Based on 30 minute retention time* (ref. George Tchobanoglous, Franklin L. Burton, and Metcalf & Eddy, Inc., *Wastewater Engineering Treatment, Disposal and Reuse*, 3rd Edition, McGraw-Hill, Inc. New York, 1991, revised by Ron Crites and George Tchobanoglous in 1998, and subtitled *Small and Decentralized Wastewater Management Systems*). *Rounded up to nominal interceptor volumes.*

(3) These values were determined based on commercially available interceptor volumes. (ref. “Basic Principles for Sizing Grease Interceptors,” Plumbing and Drainage Institute, 2006.)

The Table D.2 method assumes that the FOG-laden drainage pipe will never be more than 50% of full capacity, and also uses a 30 minute detention time.

The use of bigger tanks is not always better. If gravity grease interceptors (Type II) are sized by both flow and expected retained solids, they will be larger, ostensibly needing less frequent cleaning. However, actual field experience has now shown that over-sizing, along with poor venting and infrequent pump-outs, can result in the generation of hydrogen sulfide gas and sulfuric acid, destroying the interceptor and drainage system. So there is no single design method for grease interceptor sizing. Sound engineering judgment should be applied to each system design, and followed by timely and adequate inspections and pump-outs.

Compartments

New gravity grease interceptor (Type II) designs should incorporate two or more compartments (**Figure D-2**), or two single tanks in series, with a minimum effective capacity of 500 gallons. Multiple grease interceptors, in series, are recommended if the required effective capacity is greater than 2,000 gallons.

The inlet compartment or tank should have a capacity of 65 to 75 percent of the total capacity. For a two-compartment tank, the dividing wall should extend from the tank bottom

to six inches above the flow line and shall not extend to the roof interior without adequate venting. The transfer port or horizontal slot between compartments shall be located in the middle 25% of the distance between the tank floor and the waterline, and be a minimum area of 50 square inches.

Inlet and Outlet

Inlet and outlet pipes shall be connected to the tank with a watertight sealed flexible joint. These connections shall conform to the American Society for Testing Materials "Standard Specification for Resilient Connectors between Reinforced Concrete On-Site Wastewater Tanks and Pipes – C 1644 (latest edition)" to accommodate tank and/or pipe movement. The inlet and outlet pipes shall be constructed of PVC pipe SDR 35 minimum or equivalent. When any portion of these pipes will be subject to vehicular traffic, they shall be constructed of Schedule 40 PVC or equivalent.

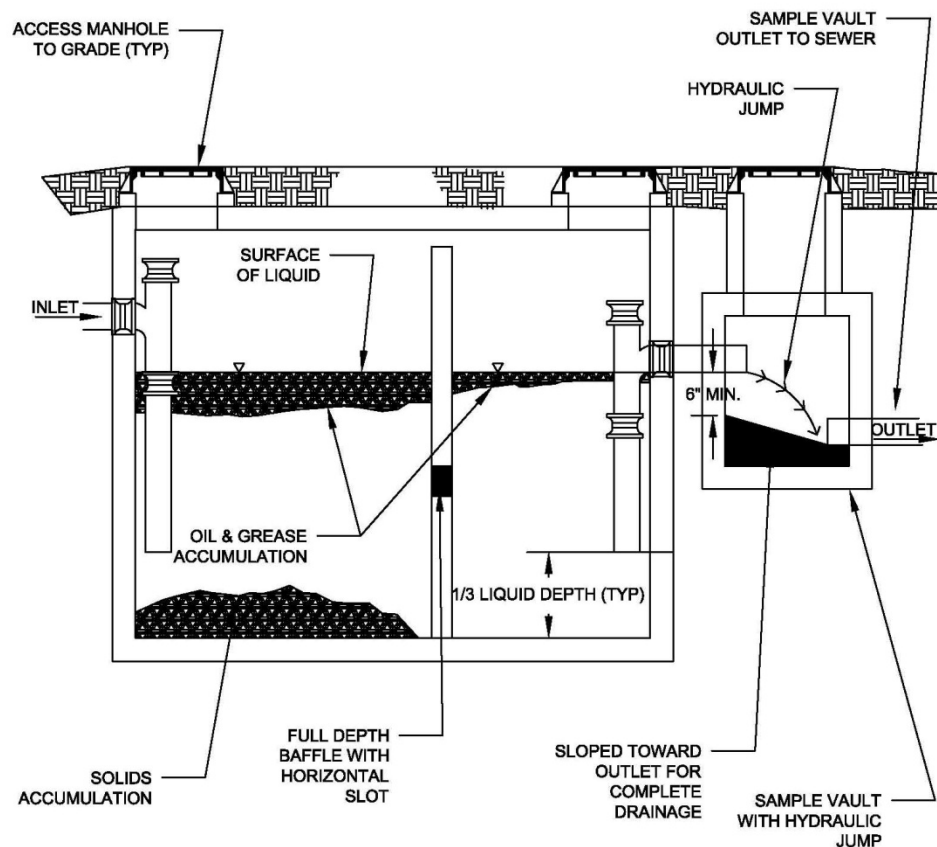


Figure D-2 Double Compartment (Type II) Grease Interceptor

Gravity grease interceptor tanks shall have an inlet and outlet sanitary tee. Sanitary tees should be installed vertically on the inlet and outlet pipes. Sanitary tees should be the same size as the inlet and outlet piping, but not less than 4" in diameter (6" is recommended for the outlet to reduce the exit velocity of the tank). The tees shall be minimum SDR 35 PVC or equivalent. A pipe nipple with an open top should be installed in the top of the tees and should terminate 6" above the waterline.

The elevation of the inlet pipe should be 2" to 4" above the elevation of the outlet pipe and extend down to the mid-depth of the tank. The same elevation drop should be used for each tank in a series. Inlet pipe designs by the Water Environment Research Foundation, WERF, minimize short-circuiting, and maximize grease retention.

The outlet tee should have a vertical pipe-rise 6" above the flow line and pipe-drop extending down to within 6" to 12" of the tank floor. An effluent filter, specifically designed for grease interceptor applications, is recommended on the outlet of a new grease interceptor in lieu of a sanitary tee. The outlet tee or filter shall be located no farther than 6 inches from the outlet end wall. Tees or filters shall be located directly under access manholes.

- Effluent filters / screens shall be sized upon peak hydraulic loading and rated capacity of the device per the manufacturer's specification. Effluent filters can be placed in a separate tank/chamber/vault. If this is done, an outlet tee shall also be installed on the grease interceptor tank.
- A high water alarm system should be provided on a tank that includes an effluent filtering system.

Type II gravity grease interceptor effluent should be discharged to a septic tank or other treatment unit; discharge should be to the pipe upstream to minimize turbulence in the receiving unit.

Effluent Sampling

A sample port/vault should be installed just downstream of the (final) grease trap to facilitate sampling of the discharge (See **Figure D-2**).

Gravity grease interceptor tanks shall have an access opening/manhole with a 20-inch minimum dimension over the influent tee and effluent tee or filter; a 24-inch opening/manhole is recommended. For tanks that have more than one (1) effluent filter, the access opening shall be of sufficient size to allow for tank pumping and removal of the filters. The access opening/manhole shall be extended to finished grade. Where risers are required, they shall be watertight. Tanks longer than 15 feet shall have an additional manhole located just upstream of the dividing wall. Access covers shall be of sufficient weight (59 pound minimum), mechanically fastened or provided with a lock system to prevent unauthorized entry. The cover and frame assembly should be gas-tight (gasketed).

Optimum dimensioning: depth (approximately) equal to width or diameter; length greater than the width. Cylindrical tanks must be positioned with the longest dimension oriented horizontally. The liquid depth of the tank should be between 36" to 72". The effective volume shall be based on a maximum 60" depth with any additional depth used for sludge storage. Tank length shall be greater than the width.

Designs for multiple interceptor tanks in series shall include the following:

- A. The tanks shall be vented for positive air/gas displacement.
- B. The outlet tee on the first tank shall be at least 6" in diameter and shall extend to at least one-third of the depth, but no further than 12", above the tank floor.
- C. The connector pipe between tanks shall be at least 6" in diameter and sloped at 1/8 inch per foot.
- D. The inlet tee on the second tank shall be at least 6" in diameter and shall extend to mid-depth of the tank.
- E. The outlet on the second tank shall be at least 6" in diameter. An effluent filter is recommended (see the Effluent Filter subsection below).

Venting

The interceptor shall be vented in accordance with requirements of the manufacturer and the Plumbing Code of NYS (typically back through the inlet plumbing and to a roof vent). Proper venting is a deterrent to build-up of gasses in the grease interceptor. Venting of hydrogen sulfide gas will preclude formation of sulfuric acid and resulting disintegration of concrete and corrosion of metal parts, e.g., re-bar.

Construction and Materials

Grease interceptors shall be watertight. Components of the system shall be constructed of durable materials not subject to corrosion, decay, frost damage, deformation or cracking. Typically, gravity grease interceptor tanks are constructed of pre-cast or poured in place concrete. Poured in place tanks must be designed and certified by a licensed professional engineer. Protection against sulfuric acid damage above the waterline may be provided in varying degrees determined by local conditions, and may include properly venting the tank, coating the interior or exterior of the concrete tank, or specifying medium- and high-strength sulfate resistant concrete mixes.

For concrete tanks, refer to the National Pre-cast Concrete Association, or the Pre-cast Concrete Association of New York (PCANY). For fiberglass-reinforced polyester (FRP) or high-density polyethylene (HDPE) tanks refer to IAPMO (International Association of Plumbing and Mechanical Officials)/ANSI Z1001 fast track certification standard for Grease Interceptors and Clarifiers, or ASTM F2649-(latest version) Standard Specification for Corrugated High Density Polyethylene (HDPE) Grease Interceptor Tanks.

All grease interceptor tanks must be *Certified Tanks*. Included in the certification requirements is the meeting of the appropriate standard for the material of construction as follows:

- A. Pre-cast concrete grease interceptor tanks shall conform to the American Society for Testing Materials “Standard Specification for Pre-cast Concrete Grease Interceptor Tanks C-1613,” and be a *Certified Tank* (see *Construction and Materials subsection in Section D.6 Septic Tanks*).

The joints for horizontal seam and vertical seam concrete tanks shall conform to the

American Society for Testing Materials “Standard Specification for Joints for Concrete Pipe, Manholes, and Pre-cast Box Sections Using Preformed Flexible Joint Sealants C-990 (latest edition)”.

When a Type II gravity grease interceptor tank is installed under a driveway, parking lot, in a heavily saturated soil or other area subject to heavy loading, the tank shall be designed to withstand an H-20 wheel load.

- B. Poured-in-place Type II grease interceptors shall meet the specifications for cast-in-place concrete given in ACI 318 Building Code Requirements for Structural Concrete (also see ASTM C-1227 [latest version]) and be designed by a licensed professional engineer.
- C. High-density polyethylene (HDPE) and fiberglass-reinforced plastic (FRP) tanks must be factory assembled, with any proposed baffles in place. Care must be taken during installation and backfilling to avoid damaging the walls. After backfilling, the tanks should be inspected and if any damage is present, the tank should be repaired or replaced. When these tanks are installed in areas where high ground water levels may be present, flotation collars should be used to prevent flotation when the tank is pumped.

Note: Resistance to the hydrostatic and hydrodynamic forces of groundwater, backfill, tank filling and tank evacuation is accounted for in the structural design of concrete tanks. High-density polyethylene and fiberglass-reinforced plastic (FRP) tank manufacturers require that their installation procedures be strictly followed to prevent tank damage at installation and to account for high groundwater conditions and tank filling and pump-outs.

- D. For Fiber-Reinforced Polyester (FRP) tanks see UL Standard 1316 titled “Glass-Fiber-Reinforced Plastic Underground Storage Tanks for Petroleum Products, Alcohols, and Alcohol-Gasoline Mixtures,” or ANSI/AWWA D120-02 Thermosetting Fiberglass-Reinforced Plastic Tanks.
- E. Metal tanks are prohibited for new installations unless prior written approval is granted by the Regional Water Engineer. They will be considered only if they are

constructed and coated in accordance with the provisions of the Underwriters Laboratory Standard UL-70.

Installation

Care must be taken during placement of bedding, installation and backfilling to prevent damage to the tank system. The grease interceptor manufacturer's installation instructions must be strictly followed. Tanks should not bear on large boulders or rock ledges. The tank should be placed on a minimum 4" level layer of sand or pea gravel. The underlying soils and bedding materials shall be adequately compacted to eliminate later settlement. When a tank is installed in an area where high groundwater levels may be present, an evaluation must be made to determine if flotation collars are necessary to prevent flotation when the tank is pumped.

Performance Testing

All Type II gravity grease interceptor tanks and related appurtenances (inlet and outlet pipe seals, risers, covers, etc) shall be (vacuum or water pressure) tested and certified to be watertight prior to backfilling (See ASTM Standards C-1227 for septic tanks and C-1613 for grease interceptors). Type I grease interceptors shall conform to the PCNYS.

Backfill

There shall be no connections such as joints, splices or fittings within the "over-dig" around the grease interceptor tank. Backfill should be placed in uniform layers less than 24" thick and should be free of large stones (> 3" in diameter) or other debris. After backfilling, the tank should be inspected and if any damage is present the tank should be repaired or replaced. Surface water shall be directed away from tank openings.

Operation and Maintenance

Grease interceptor additives should not be used as some products which claim to "clean"

grease interceptors contain compounds which provide temporary relief but may also result in permanent damage to the disposal field and cause premature clogging.

FOG is not a stable compound suitable for long-term storage, and its disposal can be difficult and costly. Storage of FOG in anoxic conditions beyond 30 days is not recommended by the Plumbing and Drainage Institute. Sulfur-reducing anaerobic bacteria produce hydrogen sulfide as a by-product of their metabolism. Proper and adequate venting should be confirmed.

Pumpouts

It is recommended that grease interceptors be pumped out on a schedule based on the results of a weekly log. Initially, the gravity grease interceptor tank should be inspected monthly. (Interior hydro-mechanical units should be inspected more frequently based on usage and manufacturer's instructions.) Scum and sludge should be measured in the first compartment of a two compartment tank, or in the first tank of a multiple tank system. Tanks must be pumped when the bottom of the *scum layer* is within 3 inches of the bottom of the outlet baffle or tee, or when the *sludge level* is within 8 inches of the outlet device. The tank should not be disinfected, washed, or scrubbed. Proper venting must be in place to minimize sulfuric acid damage of concrete tanks.

A gravity interceptor should be inspected monthly to determine FOG and solids accumulations. The tank shall be fully pumped out and cleaned at a frequency such that the combined FOG and solids accumulation does not exceed 25% of the total designed effective hydraulic capacity of the tank. This is to ensure that the minimum hydraulic retention time and required available hydraulic volume is maintained to effectively intercept and retain FOG discharged to the sewer system. The tank should be refilled with water (if possible).

Pumped-out grease interceptors often contain toxic gases. Only qualified personnel should attempt to enter or repair a grease interceptor. The average owner **SHOULD NOT ENTER THE TANK.**

FOG Management Practices for Commercial/Institutional Facilities

- Train kitchen staff on FOG-handling practices.

- Hang FOG-handling posters in the kitchen.
- Food waste should be disposed of in the trash not the sanitary sewer system.
- Identify grease recycling containers.
- Provide ample paper towel dispensers for dry wiping grease from spills, pots, frying and grilling equipment.
- Contract with FOG haulers/recyclers.
- Use strainer baskets in sinks to catch food waste.
- Direct all drains from FOG-producing sources to properly sized grease interceptors.
- Avoid food grinders. If grinders are approved, discharge them to a solids interceptor upstream of a grease interceptor.
- Schedule regular maintenance and cleaning of grease interceptors, keep a log.
- Have a copy of the recommended grease interceptor cleaning procedures on site.

Standard-setting and Certifying Organizations:

- ASPE: American Society of Plumbing Engineers
- WEF: Water Environment Federation
- AWWA: American Water Works Association
- NSF: The Public Health and Safety Company
- PDI: Plumbing & Drainage Institute
- CSA: Canadian Standards Association

D.6. Septic Tanks

Design and Sizing

Septic tanks receive sewage from the building sewer, small diameter collection sewer main, or from the grease interceptor clear zone. It must be followed either by a subsurface soil-based treatment area, or by a secondary or tertiary/enhanced treatment system prior to surface discharge. Septic tanks should be placed as shallow as possible to minimize the effect of groundwater pressure on the tank system (see Installation subsection below), and allow the subsurface system to function aerobically.

Table D.5 shows the calculations that shall be used to determine minimum effective tank capacity (or volume) for treating wastewater from commercial and institutional facilities, cluster housing and other

multi-home developments, and municipal systems treating continuous wastewater flows for 9 to 24 hours per day as well as expected peak loadings.. Tanks larger than this minimum may show enhanced performance. No tank shall have a capacity less than 1,000 gallons. Anticipated sludge and scum accumulations should be taken into account when determining the required effective capacity. to prevent an excessive rate of flow through the system

It may be necessary to increase tank size (effective volume) when a commercial or institutional facility has a Significant Delivery Period. As the delivery period gets shorter the tank volume shall increase proportionately to give a 24-hour detention time for the service delivery period based on the higher flow rate. *As an example, 2,000 gallons discharged over a 6-hour delivery period would result in an 8,000 gallon per day design flow and septic tank volume; Table D-5 is not used.*

Table D-5. Septic Tank Sizing (effective volume) for Multi-home dwellings,*

Commercial/Institutional Applications and Municipal Systems

Daily Flow, Q (gpd)	Minimum Effective Tank Capacity (gal)
Under 5,000	1.5Q
5,000 - 15,000	$3,750 + 0.75 Q$
15,000 +	Q

* For one- and two-family homes refer to NYSDOH Wastewater Treatment Standards –Appendix 75-A, 75-A.6

Garbage grinders are not recommended for facilities served by on-site wastewater treatment systems. Where garbage grinders are used or proposed, tank capacity must be increased by 1/3 to accommodate the increased volume of solids, and an effluent filter is recommended. For facilities (houses, motels, spas etc.) with high-usage plumbing fixture(s) such as in-room jacuzzis, the septic tank capacity shall be increased by 250 gallons for each high-volume fixture. Additional septic tank capacity should be considered for apartment type buildings with multiple washing machines in a common laundry area. A lint-trap or filter should be placed on the discharge line of any washing machine(s) that discharge(s) to a septic tank.

Sewage Pumped to Septic Tanks

DEC discourages the use of raw sewage pumping stations, or sewage ejector systems, when the discharge is directly to a septic tank. It is not recommended to pump any more than 25% of the design flow of the system to a septic tank. The ejector or pump station should discharge into the sewer, e.g. building sewer, prior to the septic tank.

The effective capacity of the septic tank shall be calculated based on the pumping rate (gallons per minute). This will alleviate shock loads to the septic tank. Two septic tanks in series should be used as agitation or mixing may occur in the first tank due to the pump rate. The second tank can provide a quiescent zone to promote solids separation.

Shape and Dimensions

Optimum dimensioning calls for a depth approximately equal to the width of a rectangular tank. The liquid depth of the tank should be between 36" to 72". The effective volume shall be based on a maximum 60" depth with any additional depth used for sludge storage. Shallower tanks may be used if local codes allow or require.

Tank surface area may range from 2.6 to 5.3 sq. ft. per 100 gallons of tank capacity. Tanks with greater surface area-to-depth ratio are preferred. The ratio of inside tank length to width at the liquid surface should be in the range of 2:1 to 4:1. The minimum inside tank length, from inlet to outlet, shall be 6 feet. Tanks with greater length to width ratios are more effective in solids retention. Cylindrical tanks may also be used.

Compartments

Septic tanks for new facility construction shall have two compartments (**Figure D-3**). Two single-compartment septic tanks in series (**Figure D-4**) may be used in lieu of a two-compartment tank. The inlet chamber or tank should have a capacity of approximately two-thirds of the total capacity.

For two tanks in series, two single compartment tanks should be used. An appropriate-sized effluent filter is also required. (See "Two Tanks in Series" subsection below). ***Septic tanks in parallel shall not be allowed to treat raw sewage.***

For a two compartment tank the dividing wall should extend from the tank bottom to six inches above the flow line. The wall shall have a horizontal slot at mid-depth. The slot shall be at least four inches high with a minimum area of 50 square inches (equal in area to three 4" pipe cross-sections).

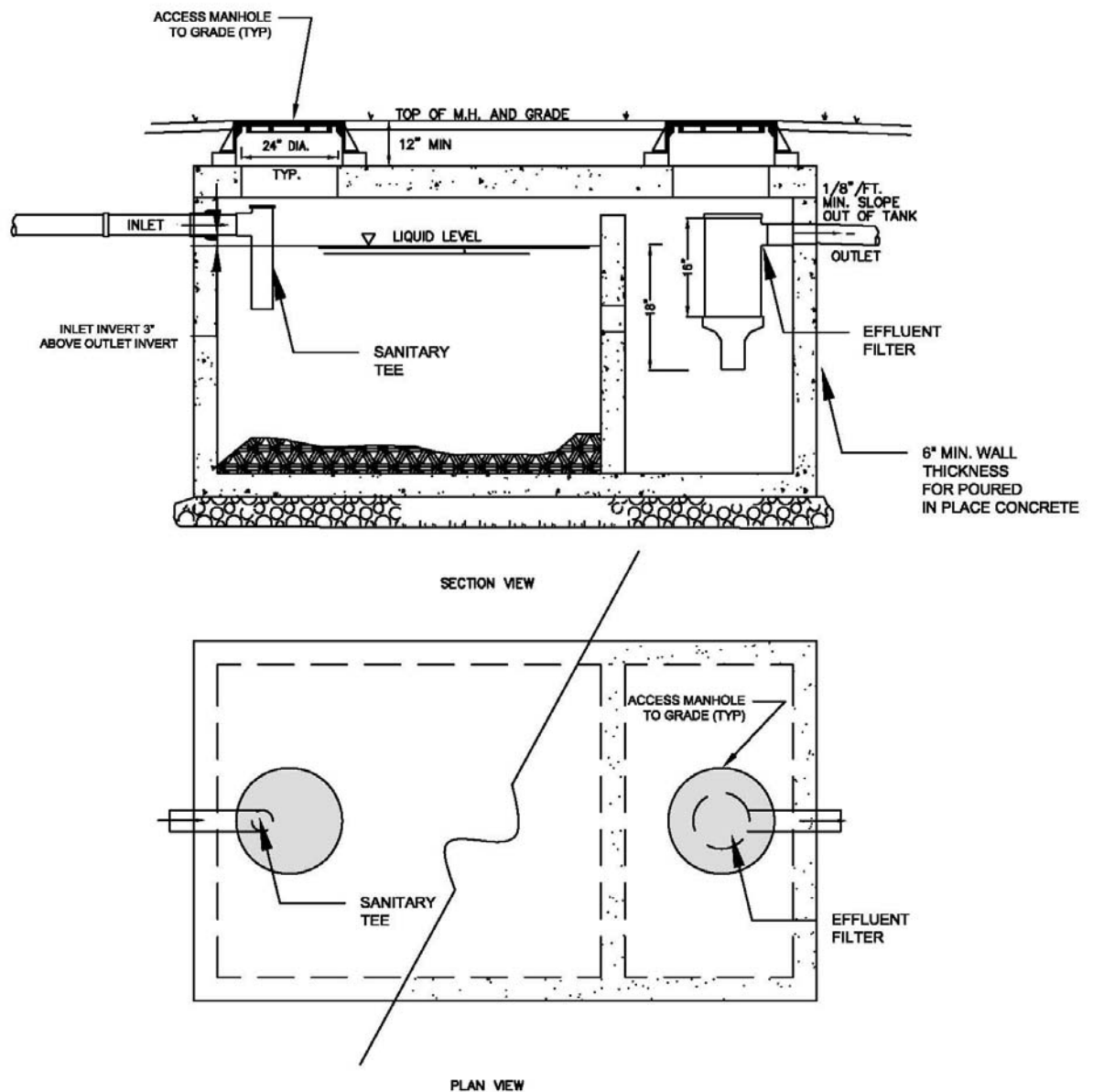


Figure D-3 Double Compartment Septic Tank

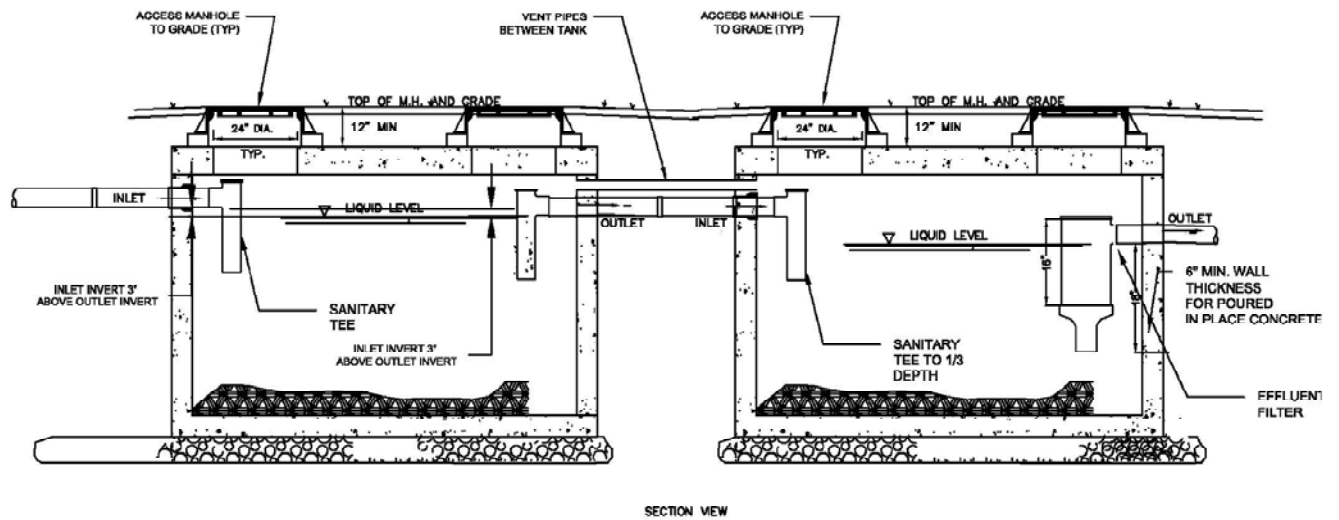


Figure D-4 Single Compartment Septic Tanks in Series

Two Tanks in Series (see Figure D-4)

Tanks plumbed in series may be used to provide the minimum effective tank capacity or as noted above

for pumped raw wastewater. Designs for multiple septic tanks shall include the following:

- A. No more than 2 septic tanks should be placed in series.
- B. The inlet tank should have a capacity of approximately two-thirds of the total capacity.
- C. The tanks shall be connected by a vent pipe at least 4" in diameter.
- D. The outlet tee on the first, or inlet, tank shall be at least 6" in diameter and shall extend down to the middle 25% of the liquid depth. .
- E. The connector pipe between tanks shall be at least 6" in diameter and sloped at 1/8 inch per foot (1% slope).
- F. The inlet tee on the second tank shall be at least 6" in diameter and shall extend down at least 1 foot below the flow line.
- G. The outlet on the second tank shall be at least 4" in diameter and provided with an effluent screen or filter.

Construction and Materials

Septic tanks shall be watertight. All tanks and components of the tank system shall be constructed of durable materials not subject to excessive corrosion, decay, frost damage, deformation (cracking or buckling) due to settlement or soil pressures. Tanks may be constructed of properly cured, pre-cast or poured in-place concrete, high density polyethylene (HDPE) or fiberglass-reinforced plastic (FRP).

All tanks must be suitable to withstand burial to three feet over the top of the tank without loss of volume or change in shape after being pumped completely empty under saturated soil conditions. Septic tanks of materials other than concrete shall still meet the minimum structural design requirements of ASTM C1227 – latest revision.

All septic tanks must be certified. Included in the certification requirements is the meeting of the appropriate standard for the material of construction as follows:

- A. The Precast Concrete Association of New York (PCANY) Precast Concrete Tank Certification Program assures water-tightness and durable construction of septic tanks, holding tanks and grease interceptors. A red tag indicates a certified tank. All manufacturers achieving certification are listed on PCANY's website. The RECOMENDED GUIDELINES FOR INSTALLATION OF CONCRETE SEPTIC TANKS is also on the website: www.pcany.org.

Concrete septic tanks may be coated with a bituminous coating to ensure water-tightness and prevent deterioration when deemed necessary by the design engineer, reviewing engineer or the manufacturer due to specific site conditions. Concrete tanks treating wastewater that is high in sulfur or sulfate must be adequately vented. Concrete tanks may also be constructed to ASTM Specification C150 Type II for moderately-sulfate-resisting cement, or by the addition of appropriate additives.

- B. Poured-in-place tanks must be designed and certified by a licensed professional engineer (i.e. signed plans, witnessed construction and leak tested), and shall conform to the ACI 318 Building Code Requirements for Reinforced Concrete.
- C. High-density polyethylene (HDPE) and Fiberglass (FRP) septic tanks shall comply with the "Material and Property Standard for Prefabricated Septic Tanks," IAPMO/ANSI Z1000-2007 (or latest version).

Both HDPE and FRP tanks must be factory assembled, with any proposed baffles in place. Inlet and outlet fittings, sometimes identified as baffles, require field installation and connection to the ends of the sewer (inlet) line and effluent (outlet) line at time of tank installation.

- E. Metal tanks are prohibited for new installations unless prior written approval is granted by the Regional Water Engineer. They will be considered only if they are constructed and coated in accordance with the provisions of the Underwriters Laboratory Standard UL-70.

Inlet and Outlet

All new septic tanks shall have an inlet sanitary tee, and an effluent screen or filter in lieu of the outlet tee (see Section D.7). Sanitary tees (or effluent filters) should be installed vertically on the inlet and outlet pipes. Tees should be the same size as the inlet and outlet piping, but not less than 4" in diameter (6" is recommended for the outlet to reduce the exit velocity of the tank). A pipe nipple with an open top should be installed in the top of the tee and should terminate 6" below the roof of the tank. The inlet tee shall rise 6 inches above the flow line and extend down to the middle 25% of the liquid level. The tees shall be minimum SDR 35 PVC or equivalent.

There shall be a 2- to 4-inch drop in elevation between inverts of the inlet and outlet of the tank (and of each tank in series). The outlet tee (with an effluent filter insert) should have a vertical pipe nipple 6 inches above the flow line and pipe-drop extending to the middle 25% of the liquid level. Tees or filters shall be located directly under access manholes. The outlet tee or filter shall be located no further than 6 inches from the outlet end wall. When effluent filters are being used, tank depth and flow rate should be taken into consideration along with manufacturer's recommendations for the model to be installed (see Section D.7).

There shall be no connections such as joints, splices or fittings within the "over-dig" around the septic tank. Inlet and outlet pipes shall be connected to the tank with a watertight, flexible joint/connector. For concrete tanks the specifications are in the American Society for Testing Materials (ASTM) "Standard Specification for Resilient Connectors between Reinforced Concrete On-Site Wastewater Tanks and Pipes - C-1644 (latest edition)" to accommodate tank and/or pipe movement. The inlet and outlet pipes shall be constructed of PVC pipe SDR 35 minimum or equivalent. When any portion of these pipes will be subject to vehicular traffic, they shall be constructed of Schedule 40 PVC or equivalent.

Access Openings

Septic tanks shall have an access opening/manhole with a 20-inch minimum dimension over each influent tee and effluent tee or filter. A 24" opening/manhole is recommended. For tanks that have more than one effluent filter, the access opening shall be of sufficient size to allow for tank pumping and removal of the filters. The opening/manhole shall be extended to finished grade. Where a riser(s) is required, it shall be watertight. Tanks longer than 15 feet shall have an additional manhole located just upstream of the dividing wall. Access covers shall be of sufficient weight (59 pound minimum), or mechanically fastened, or provided with a lock system to prevent unauthorized entry. The cover and frame assembly should be

gas-tight (gasketed).

Where more than one inlet is necessary for multiple building sewers or when the sewer enters at the side of the tank, the inlet pipe(s) shall be extended to the center manhole (for access and inspection) or the tank shall be manufactured or modified with additional manholes.

Installation

Tanks should not bear on large boulders or rock ledges. Concrete septic tanks should be placed on a minimum 4 inches of leveled #2 stone, or smaller aggregate such as sand or pea gravel to provide adequate bedding. The underlying soils and bedding materials shall be adequately compacted to eliminate later settlement.

Vertical-seam (culvert-type) pre-cast concrete tanks shall be installed in accordance with the manufacturer's instructions which may include a 4" minimum flowable-fill mud slab to aid in the assembly of the sections. When a tank is installed in an area where high groundwater levels may be present the design engineer should determine whether buoyancy forces require tank design modifications. An evaluation of HDPE, FRP and metal tanks should be made to determine if flotation collars are necessary to prevent flotation when the tank is pumped.

Note: Resistance to the hydrostatic and hydrodynamic forces of groundwater, backfill, tank filling and tank evacuation is accounted for in the structural design of concrete tanks. HDPE and FRP tank manufacturers require that their installation procedures be strictly followed to prevent tank damage at installation and to account for high groundwater conditions and tank filling and pump-outs.

When a septic tank system is installed under a driveway, parking lot, in a heavily saturated soil or other area subject to heavy loading, the tank shall be designed to withstand an H-20 wheel load.

Performance Testing

Protection from water infiltration and exfiltration is a critical element in the design, installation and/or construction of tanks used in on-site wastewater treatment systems (NPCA). All septic tanks and related appurtenances (inlet and outlet pipe seals, risers, covers, etc) shall be vacuum or water-pressure tested and

certified to be watertight after installation and prior to backfilling. (See ASTM Standards C-1227 for septic tanks and C-1613 for grease interceptors). Also see Appendix G of “Installation of Wastewater Treatment Systems by the Consortium of Institutes for Decentralized Wastewater Treatment (CIDWT), latest version.

Backfill

Backfill and bedding of all tanks of materials other than concrete shall strictly follow manufacturer’s written recommendations. Backfill should be placed in uniform layers less than 24" thick and should be free of large stones (> 3" in diameter) or other debris. After backfilling, the tank should be inspected again, and if any damage is present the tank should be repaired or replaced. Surface water shall be directed away from tank openings.

Operation and Maintenance

The use and discharge of chemicals, bacteria, enzymes or other products into a sewer served by a septic tank is prohibited unless prior written approval is granted by the Regional Water Engineer.

For SPDES-permitted systems, including the General Permit 0-05-001, Part 750-2.8 (d) “SPECIAL CONDITION - DISPOSAL SYSTEMS WITH SEPTIC TANKS” applies:

“Unless otherwise directed by the regional water engineer, if a septic tank is installed as part of the disposal system, it shall be inspected by the permittee or his agent for scum and sludge accumulation at intervals not to exceed one year's duration, and such accumulation will be removed before the depth of either exceeds one-fourth (1/4) of the liquid depth so that no settleable solids or scum will leave in the septic tank effluent. Such accumulation shall be disposed of in accordance with all applicable law and regulation.”

Environmental conditions such as temperature can affect the frequency and composition of the solids to be removed from the tank. Exceptionally cold weather can result in septic tank temperatures where little or no anaerobic digestion can take place so settled sludge and scum will accumulate faster than it would at more moderate temperatures.

Pumped-out septic tanks often contain toxic gases and are considered confined spaces by the

Occupational Safety and Health Administration (OSHA). As such, septic tanks should not be entered by anyone UNLESS they have OSHA certified training and equipment to do so.

D.7. Effluent Screens / Filters (for septic tanks and grease interceptors)

Septic tanks for new construction shall have a properly-sized effluent filter (according to the manufacturer) installed at the outlet of the tank in lieu of or contained within the sanitary outlet tee. Septic tank effluent filters or screens are recommended for all treatment systems. For buildings with laundry service, lint traps, filters or screens should be installed on the discharge line of washing machines to remove non-biodegradable solids, such as cloth fibers, sand, hair and pet fur.

Effluent filters shall be sized based upon peak hydraulic loading and rated capacity of the device per the manufacturer's specification. Effluent filters can be placed in a separate tank/chamber/vault. If this is done, an outlet tee shall also be installed on the grease interceptor tank, or the septic tank outlet. *A high - water alarm MAY be desired to keep water out of the building and where a dedicated operator or service provider can respond.*

When multiple filters are installed in a septic tank, it is imperative that the access opening be of adequate size for filter removal and maintenance. The filters should be inspected on a monthly basis to observe the characteristic solids accumulation rate of the filter.

D.8. Dosing Stations

A dosing system provides for the periodic discharge of a calculated volume of pre-treated wastewater to a soil-based treatment and dispersal field, or secondary treatment unit for either subsurface or surface discharge. Dosing uses the principle of wetting and resting. A resting period is important to maintain aerobic conditions. A dose-rest cycle is based either on when the dosing station fills (demand dosing) or on a time basis (time dosing). Dosing may be accomplished with a pump, siphon, or float-style dosing mechanism.

D.9. Distribution Boxes / Flow Splitters

See Section E.5 for use of distribution boxes and flow-splitters in distribution networks.

Distribution Boxes

A distribution box should have a removable cover, and a riser that extends to grade for inspection and access, and that also serves as a permanent location marker.

Installation

The box must be placed level, with a layer of sand or pea gravel 12 inches deep below the box, and around the sides. A slope of at least 1/8 inch per foot must be maintained from the pretreatment unit to the box.

Inlets and Outlets

The invert elevation of all outlets shall be equal and located at least two inches below the invert elevation of the inlet. If all inverts are not at the same elevation, or if uneven settlement or frost heaving has resulted in unequal flow to the lateral lines, adjustments must be made to reestablish equal division of flow. Several devices can be used including adjustable weirs, or leveling devices that can be inserted into each outlet pipe and rotated so the flow is equally distributed. Outlet distribution lines should all be laid at an equal slope of no more than 1/16" per foot until reaching the header pipe or laterals. There should be one outlet for each effluent distribution line. It is recommended that outlet inverts be placed at least two inches above the floor of the box so that water placed in the box can be used for leveling purposes.

The box shall have an internal baffle or splash plate extending to one inch above the outlet invert elevation to dissipate the velocity of the influent.

Flow Splitters

A flow-splitting device is another option that can be operated from above ground allowing fields to be alternated for longer service life. Install, operate and maintain according to manufacturer's instructions.

D.10. Wastewater Dumping Station

Dump station wastes are typically generated from users of recreational vehicles (RVs), travel trailers and recreational water craft. Dump stations can be found at campgrounds, roadside rest areas and marinas. The design of a wastewater treatment system for these facilities including the minimum effective capacity

of the septic tank shall take the following into account:

- A. RV operators are generally considered conservative with water usage which results in wastewater that has a high organic and solids loading being discharged into the dump station.
- B. The typical mode of introducing waste into the receiving station - dumping - is not conducive to the steady state flow needed for optimal biological treatment.
- C. The effect of various amounts and types of chemical additives used for odor control. These additives kill or prohibit the growth of anaerobic bacteria in the holding tank. Anaerobic bacteria are the primary treatment mechanism in a septic tank.
- D. Several peak utilization periods (Sundays, days after holidays) can be expected.
- E. Literature suggests that:

* The required effective septic tank capacity is significantly more than indicated by the typical hydraulic capacity designs used for treating domestic wastewater. Requirements of 2 to 15 times the volume required for typical domestic waste have been noted.

* RV holding tank contents should be blended / mixed with other sanitary wastewater sources to reduce the organic strength prior to treatment. Chemical analyses of RV holding tank (blackwater) have noted BOD₅ ranges of 1,600 to 12,000 mg/l (excluding wash-down water). Similar wastewater characteristics should be expected from sewage holding tanks on recreational watercraft, a.k.a. Marine Sanitation Devices (MSDs).

* A septic tank serving a dump station requires more frequent pumping due to an increase in solids loading and reduction in solids digestion/breakdown due to chemical additives.

* The use of pumps (in lieu of siphons, etc.) to dose absorption fields and sand filters is recommended due to the potential elevated strength of the septic tank effluent.

* The soil-based treatment and dispersal area and sand filter sizing should be based on organic

loading.

D.11. Holding Tanks

In that holding tanks are not designed to discharge, they are not regulated under the authority of DEC. Approvals for the use of a holding tank should be sought from NYSDOH.

D.12. Source Separation and Graywater Irrigation Systems

The following design criteria pertain to graywater *irrigation systems* discharging to subsurface soil on-site. Graywater *treatment systems* require SPDES permits. Residential graywater *treatment systems* with design flows under 1000 gpd should be designed according to Appendix 75-A, and approved by a local health unit (County or District).

Description:

Graywater, because of its low nitrogen and pollutant concentrations as compared to combined or undifferentiated wastewater, can be assimilated biologically within the topsoil (top 2 to 10 inches) and simultaneously useful for drip- or flood-irrigation. Spray irrigation of graywater is prohibited.

Graywater irrigation systems are used most frequently in areas trying to conserve fresh water. They are also used in situations where there is a desire to reduce hydraulic and pollutant loading of an existing onsite sewage treatment systems in order to increase system life.

Graywater irrigation systems should not be used at facilities discharging wastewater containing higher concentrations of pathogens from the washing of heavily soiled or potentially infectious laundry such as diapers, or similarly soiled garments unless the graywater is disinfected before irrigation.

Rinsing or discarding of any hazardous chemicals, synthetic organics or petroleum by-products from oils, paints, and solvents should be avoided by signage at the facility and proper disposal or treatment of any industrial wastes.

Irrigation of the shallow topsoil provides greater uptake of the nutrients and pollutants characteristic of graywater as compared to subsoil discharge typical of traditional onsite treatment systems, which in turn further reduces groundwater pollution. This technique also leads to reduction in the need of fresh water

use for irrigation.

For applications with source separation where graywater primarily consists of wash water (i.e. hand-washing, showers, and small kitchen sinks without garbage grinders), facilities may have graywater irrigation systems designed and installed as an alternative to traditional onsite treatment systems, or as a replacement to failing traditional onsite systems.

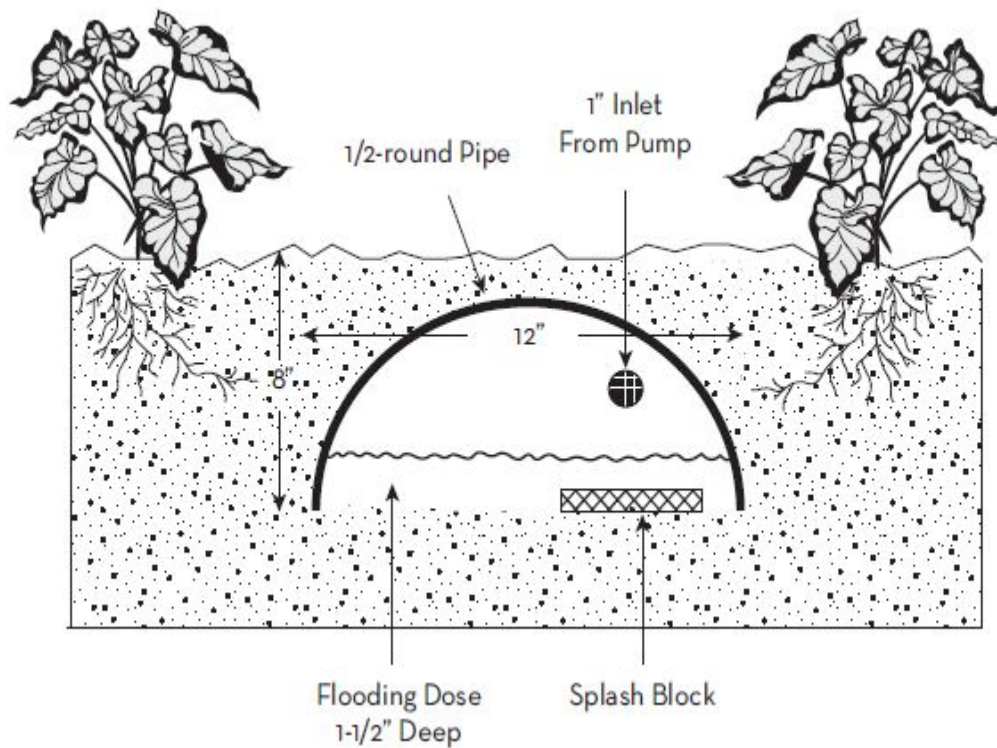


Figure D-5 Graywater Irrigation Chambers

This graywater irrigation system design may consist of:

- A) A graywater dosing basin or chamber is the collection point of all graywater. The dosing basin stores graywater until a sufficient dose has collected in the basin to flood the full length of the irrigation chambers to an approximate depth of 1.5", or stores it for up to 48 hours, maximum, for systems where water use may be intermittent. Where water-saving devices are used a maximum of 24 hour residence time shall be used. Any graywater dosage tank should be covered and secured/locked to restrict access and to eliminate habitat for mosquitoes or other vectors.

The dosing is regulated by float controls and may include an electric switch including a timer where necessary. If gravity feed to distribution beds is not sufficient to overcome slope and friction losses, a dosing/pump chamber can be included in the system. Effluent is pumped to irrigation chambers via an effluent pump or gravity siphon where sufficient slope is available. Any pressure piping (or valves or faucets) used in a graywater irrigation system shall be clearly labeled to indicate that the piping does not carry potable water.

- B) Irrigation chambers (*Figure D-5*) consist of half round pipe 8-12" in diameter, usually 30' in length, set horizontally along the contour, several inches below the surface of the soil, so the base of the half-pipe rests on the soil at 8" of depth. Effluent is sent into the chamber through 1" pipe inlets set inside the chamber, but above the base of the chamber to prevent freezing. Surface irrigation with graywater shall only be by flood- or drip-irrigation. Containment within horticultural basins or swales is encouraged for flood-irrigation. Irrigation shall be managed to minimize standing water on the surface, for example, by splitting the flow, using moderate application rates, and mulching generously.

The aerobic and biologically active top two to ten inches of soil are an extremely effective medium for rapidly stabilizing "fresh" graywater. After an excessive period of graywater storage, the available oxygen will be depleted and the stored water will become anaerobic. For this reason septic tanks are not desirable for the described system. Any type of settling tank will cause the graywater to become septic and lead to foul-smelling effluent that the topsoil will take longer to stabilize.

For facilities using an onsite wastewater treatment facility for black water treatment and disposal, the added use of a graywater irrigation system does not reduce the design, capacity, or reserve area requirements for the onsite wastewater treatment facility. The wastewater system is designed to ensure that it can handle the combined black water and graywater flow if the graywater irrigation system fails or is not used.

The following precautions shall be followed when designing an onsite graywater system:

1. Avoid human contact with graywater and soil irrigated by gray water (signage, fencing, landscaping, etc.);
2. Graywater originating from the building or dwelling units and discharged is contained within the property limits for on-site garden (non-edible plants) irrigation, composting, lawn watering, or

landscape irrigation;

3. Surface application of graywater shall not be used for irrigation of food plants (except for fruit and nut trees, which do not have an edible portion that comes in direct contact with graywater);
4. Graywater shall not be used for purposes other than irrigation (subsurface trench or drip-dispersal); and
5. Towns, cities, or counties may further limit the use of graywater described in this Section by rule or ordinance.

The following criteria shall be followed when designing a graywater system:

1. Design flows for a graywater irrigation system may be reduced by 50% versus the standard sewage system design flow in Table B.3.
2. Systems with design flows over 5,000 gpd shall provide a groundwater mounding analysis.
3. Systems with design flows over 30,000 gpd shall provide ground water monitoring.
4. The graywater irrigation system is sited outside of a floodway (i.e. not used for drainage).
5. The depth of a graywater treatment bed or trench bottom should be no more than 12" below ground level. An 8" depth is recommended. Shallow percolation tests or a soil evaluation is required to properly size treatment area.
6. Graywater irrigation systems shall maintain a minimum vertical separation distance of at least two feet from the point of graywater distribution (trench or bed bottom) to the top of the seasonally high groundwater table.
7. For replacement systems, the minimum area requirements for trenches or beds may be reduced by 50%, and minimum set-back distances may be reduced by 75% from the combined wastewater requirements. This reflects both the hydraulic load reduction of 50% (no flush toilet, no garbage grinder), and the 50% reduction for reduced pollutant load (no human body excreta or garbage grinder food waste).
8. The recommended minimum distance to nearest well: 100' for new; 50' for replacement.
9. The recommended minimum distance to nearest drainage ways, steep slopes, rock outcrops is 25' for new, and 10' for replacement systems.
10. The graywater irrigation system shall be designed and installed so that if blockage, plugging, or back-up of the system occurs, the graywater can be directed into the sewage collection system or onsite wastewater treatment and disposal system, as applicable. The system may include a means of filtration to reduce plugging and extend system lifetime with required maintenance;
11. Graywater diverter-valves should be downstream from traps and vents in plumbing that leads to septic or sewer.

E. Subsurface Treatment And Discharge

E.1. Introduction

Subsurface soil-based treatment and discharge via a conventional soil-based treatment system (STS) will not be allowed if the required depths of usable soil as measured from the bottom of the system to the seasonally high water table (2 feet), and to fractured or porous bedrock, or impervious strata (4 feet) as given in Section B of this document are not available. However, a shallow narrow drainfield infiltration system with a vertical separation of 18" to the seasonally high water table may be allowed where the wastewater is pretreated biologically and distributed using pressure distribution, or dosing, using the same hydraulic loading rate given for the percolation rate of the soil in Table E-1 (also see Section E.10 Shallow Absorption Trenches and Shallow Narrow Drainfields). Fill, raised and mound systems may be used to achieve the necessary vertical separation (Sections E-12 through E-14).

The minimum pretreatment required prior to a subsurface wastewater infiltration system (SWIS) is settling in a septic tank. A greater degree of treatment may prolong the life of the soil absorption system, and may be necessary for wastewater with elevated organic loading. All surface runoff shall be diverted away from the STA both during and after construction. Roof, foundation, cellar, and garage floor surface, storm, and groundwater drainage shall be diverted from the STA.

Planting of lawn-grass, and other shallow-rooted plants over the soil-based treatment area (STA) is required. Trees and deep-rooted shrubs shall not be planted over a STA. Grazing of livestock on a STA is not recommended. The unavoidable compaction and possible over-grazing of the area are both destructive to the function of the area. Also, no building or impervious surface (including driveways and parking lots) shall be placed upon, nor vehicles (cars, tractors, trucks) nor heavy equipment allowed upon, a soil-based treatment area, nor down-slope from the STA where system failure from soil compaction or effluent day-lighting may occur.

Staking out of the STA is required prior to other site development activities (building locations, well-drilling, material deliveries, etc.). Pre-application meetings or pre- and post-construction meetings shall be used to prevent these destructive or compacting actions from occurring. Landscaping activities following construction can also be a source of compaction or unintentional excavation or excessive fill being placed over the STA.

E.2. Application Rates

Design of soil-based treatment systems should be based on the results of the site and soils evaluation as outlined in Section B. Recommended maximum sewage application rates for various percolation rates are shown in Table E-1. The soil types listed in Table E-1 are included as a guide to likely percolation rates, although local factors such as soil structure and surface topography may cause actual percolation rates to differ from this ideal. A percolation test whose results are inconsistent with the deep test pit soil evaluation shall be disregarded, and the percolation test(s) shall be performed again.

To protect the STA from becoming anaerobic due to sealing off by high concentrations of fats, oils and greases (FOG), pretreatment using a grease interceptor in addition to a septic tank and septic tank effluent filter is required. A FOG concentration of no more than 25 mg/l applied to the soil treatment area is recommended. Dosing or pressure distribution and alternating fields are also required for these high FOG effluents.

For remediation of existing systems, secondary or tertiary treatment of the wastewater and subsequent reductions in soil-based treatment system (STS) field size (increase in application rate) may be allowed by the reviewing engineer. The linear loading rate (LLR, based on hydraulic conductivity and the horizontal flow component of each soil horizon) should be calculated by the design engineer or a hydrogeologist, and compared to the vertical wastewater application rate (Table E-1) to determine how the applied wastewater will move off site. The STS should be chosen and designed based on both vertical and horizontal hydraulic conductivities with the longest dimension parallel with the contour of the site. Dosing or pressure distribution and alternating fields are also required.

Depending on the individual circumstances (sophistication of treatment system, environmental sensitivity, presence of an operator, and available land), replacement of individual failed components may require the same or similarly-sized components (i.e. no reduction in field size). **No increase in the application rates (i.e. no reduction in field size) will be allowed for new construction.**

Conventional soil-based wastewater treatment systems (absorption trenches or beds) preceded solely by septic tanks shall not be used for rapidly permeable soils (less than 1 minute per inch) since the treatment provided may not be sufficient to protect nearby water supplies from contamination by nitrates, detergents, or other chemicals. A combination of both an acceptable percolation rate and depth of soil is needed to give the needed detention time in the soil for treatment. If amending or replacing the soil, the percolation rate and replacement procedure must be documented by the design engineer to the satisfaction of the reviewing engineer.

Conventional soil-based treatment systems should be avoided if the percolation rate is slower than 60 mpi for trenches, or 30 mpi for beds, especially if other difficult factors are present such as steep slopes, depressions, or high groundwater or bedrock. The linear loading rate should be calculated by the design engineer or a hydrogeologist, and compared to the vertical wastewater application rate (**Table E-1**), to determine how the applied wastewater will move off site. The soil-based treatment system should be chosen and designed based on both vertical and horizontal hydraulic conductivities with the longest dimension parallel with the contour of the site. Dosing or pressure distribution and alternating fields are also required.

Table E-1. Recommended Sewage Application Rates		
Percolation Rate (min/inch)	Soil Type	Application Rate (gal/day/sq. ft.)
< 1	Gravel, Coarse Sand	Not suitable ^a
1-5	Coarse-Medium Sand	1.20
1-5	Fine and Very Fine Sand	0.60
6-7	Fine Sand, Loamy Sand	1.00
8-10		0.90
11-15		0.80
16-20	Sandy Loam, Loam	0.70
21-30		0.60
31-45	Loam, Porous Silt Loam	0.50
46-60		0.45
61-120	Silty Clay Loam, Clay Loam	0.20 ^b
> 120	Clay	Not Suitable
a) May be suitable if a modified soil-based treatment system, pressure distribution, or enhanced treatment prior to discharge is utilized. b) Careful site analysis is necessary to show that these soils will transmit the flow of wastewater. Extreme caution must be used to avoid damage to the site during construction, or the system will fail. <u>Surface discharge of treated wastewater may be preferable in certain cases.</u>		

E.3. Nitrate Advisory for Soils above Aquifers

This nitrate advisory is for wastewater discharges wherever land overlies the recharge area for a confined aquifer, or an unconfined unconsolidated or bedrock aquifer used as a source of groundwater supply. Aquifers are geologic formations able to support domestic well yields or higher yields (e.g. both low-yield and high-yield unconfined aquifers). Recharge water for unconfined aquifers travels approximately vertically to reach the water-bearing zones used for well water supply sources. Recharge water for confined aquifers comes through recharge areas where water enters the subsurface and then migrates into an aquifer which is isolated from the immediately overlying landscape.

GIS for Aquifers

The design engineer may want to know the location of the Principal and Primary aquifers prior to locating a facility, or development project. TOGS 2.1.3 provides guidance to determine if an aquifer is considered a Primary Water Supply Aquifer or Principal Aquifer, and is at: <http://www.dec.ny.gov/lands/36119.html>. Where such recharge zones are not included, the design engineer may want to consult with a qualified geologist to determine possible recharge zone(s) for the identified confined aquifer.

Percolation rates faster than 10 minutes per inch

The application rates given in **Table E-1** may not be sufficient to protect groundwater in soils with percolation rates faster than / less than 10 min/inch which overlie aquifers designated by New York State as Primary Water Supply Aquifers and Principal Aquifers. In these areas, extra protection may be required to prevent degradation of groundwater quality.

To be more protective of groundwater in general where soils are unconfined and wastewater will flow through quickly, or soil is thin, a conventional septic system will not be allowed. Some additional protective action must be implemented, either with appropriate soil fill or pre-treating the wastewater in order to protect drinking water supplies, both public and private.

Groundwater Protection

The Division of Water Technical and Operational Guidance Series (1.1.1) AMBIENT WATER

QUALITY STANDARDS AND GUIDANCE VALUES AND GROUNDWATER EFFLUENT LIMITATIONS provides a compilation of ambient water quality guidance values and groundwater effluent limitations for use where there are no standards (in 6 NYCRR 703.5) or regulatory effluent limitations (in 703.6).

The standards in 703.5 and groundwater effluent limitations in 703.6 are also included in TOGS 1.1.1. The values in TOGS 1.1.1 (guidance and regulatory) are used in Department programs, including the SPDES permit program. TOGS 1.1.1 is divided into two Parts.

Part I describes ambient standards and guidance values and lists them in Table 1. Ambient water quality standards and guidance values for protection of sources of drinking water are designated Health (Water Source) and noted by H(WS).

Part II describes groundwater effluent limitations (CLASS GA) and lists them in Table 5. All fresh groundwater in New York State is classified as GA

The Division of Water (DOW) regulates point source discharges to class GA groundwater by imposing effluent limitations in the SPDES permit. These effluent limitations are set at concentrations that should prevent contaminants from exceeding ambient groundwater standards and guidance values, which are applicable in the saturated zone. Final wastewater treatment occurs in the unsaturated zone (or vadose zone) above the saturated zone.

For both nitrate (expressed as N) and the sum of nitrate and nitrite (also expressed as N) the standard is 10 mg/l for ambient groundwater. The ambient groundwater standard is 1 mg/l for nitrite (expressed as N) [from 6 NYCRR 703.5].

For both nitrate (expressed as N) and the sum of nitrate and nitrite (expressed as N) the groundwater effluent limitation is 20 mg/l. The groundwater effluent limitation for nitrite (expressed as N) is 2 mg/l. In addition, there is a groundwater effluent limitation for Total Nitrogen (expressed as N) of 10 mg/l for the counties of Nassau and Suffolk [from 6 NYCRR 703.6].

Nitrate Modeling

Nitrate modeling and field data indicate that wastewater discharges can exceed groundwater standards

when insufficient on-site, adjacent, or near-site dedicated recharge area exists to reliably dilute wastewater concentrations below drinking water standards. The object of nitrate modeling is to estimate whether groundwater at the down-gradient margin of a property or study area meets a planning target of 5 mg/l or less.

Density Threshold

When the design population density exceeds 2 to 4 dwelling units/acre (6 to 11 persons/acre) it is probable that treated wastewater from conventional subsurface disposal systems recharging the aquifer will exceed the nitrate standard for drinking water. Although NYSDEC does not make zoning regulations, it is recommended that population densities be kept below this level unless local factors are such that it can be shown that the project will not result in groundwater degradation either alone, or in combination with other discharges (including fertilizers, animal manure, and pesticides that are leached from the ground surface). In these qualifying areas, extra protection may be required to prevent degradation of groundwater quality. Protective actions may include either connection to a decentralized or centralized wastewater treatment facility discharging to a surface water body, and thereby exempting the need for this advisory, or by the addition of denitrifying on-site treatment of the wastewater prior to subsurface discharge to achieve discharge concentrations that will protect groundwater drinking water supplies.

If the population density exceeds 11 persons /acre, soil-based treatment system (STS) design shall be modified to provide enhanced treatment of the wastewater by the soil system (e.g. greater vertical separation / usable soil depth), or denitrifying treatment shall be provided prior to subsurface discharge. The addition of such wastewater pretreatment (beyond that provided by conventional septic system designs) is required if residential development density exceeds thresholds given on **Table E-2** (in *acres per person of dedicated recharge area*). **Table E-2** may be used if recharge areas for a proposed residential-only project fall entirely in one Hydrologic Soil Group. Otherwise, recharge rates from **Table E-3** must be used in calculations shown in the example below, for either residential or non-residential projects, to determine if a groundwater nitrate concentration of 5 mg/l can be met.

Hydrologic Soil Group	Inches of Annual Precipitation***							
	34	36	38	40	42	44	46	48
	Density Action Threshold, in acres per person							
A or A/D	0.62	0.58	0.55	0.52	0.50	0.48	0.45	0.44
B	0.83	0.78	0.74	0.70	0.67	0.64	0.61	0.59
C or C/D	1.57	1.49	1.40	1.34	1.27	1.20	1.15	1.10
D	2.78	2.63	2.49	2.36	2.25	2.14	2.05	1.96

Table E.2 Density Action Threshold*, in Acres per Person of Dedicated Recharge Area
(for use with residential projects on a single soil group)**

* If an entire site lies within a single Hydrologic Soil Group, Table E-2 may be used to determine if enhanced wastewater treatment is needed. For example: if 30 persons will reside on 10 acres with HSG B soils in an area with 40 inches of rainfall, multiply 30 persons times the threshold value of 0.7 acres-per-person to identify that the project would need 21 acres of dedicated recharge area to avoid needing enhanced treatment. So pretreatment would be needed if this project is to proceed with subsurface disposal.

** Dedicated Recharge Area meaning available abutting land in the same watershed assigned uniquely to a particular project site as perpetually protected and available recharge lands, as detailed further in the text of subsection E.2.

*** Precipitation Rates are available from Randall, A. 1996. Mean Annual Runoff, Precipitation, and Evaporation in the Glaciated Northeastern United States, 1951-1980, USGS Open-file Report 96-395. Newer sources may be consulted.

Model Source Data:

NJDEP/NJDEC aquifer recharge and septic density literature, and Dutchess County Aquifer Recharge Rates & Sustainable Septic System Density Recommendations (Chazen Companies, 2006).

Where central or decentralized sewer systems discharging to an acceptable surface water body with a SPDES permit, or septic systems with denitrifying wastewater treatment processes are proposed, higher density can be considered.

Hydrologic Soil Group	Inches of Annual Precipitation***							
	34	36	38	40	42	44	46	48
	Inches of Annual Aquifer Recharge							
A or A/D	15.5	16.4	17.3	18.2	19.2	20.2	21.1	22.0
B	11.3	12.0	12.6	13.3	14.0	14.7	15.4	16.0
C or C/D	5.8	6.1	6.5	6.8	7.2	7.6	7.9	8.3
D	3.2	34.0	3.6	3.8	4.0	4.2	4.4	4.6

Table E.3 Inches of Annual Aquifer Recharge, by Hydrologic Soil Group and Annual Precipitation

Desktop Modeling using Tables E-2 and E-3

The average population density for an area may be calculated based upon the total adjoining, recharge-contributing, land area in a common watershed of the development. For example, in addition to land that dormitories are on, adjacent playing fields could be included when determining the average population density for a boarding school. Similarly, parks, road right-of-ways, and all other adjacent permanently protected open space — excluding wetlands, streams, ponds or any other open waterbodies — may be included when determining the average population density for a particular area. Care must be taken to ensure that the same recharge area is not assigned to provide dilution to more than one project. In areas served by multiple OWTs, each system cannot claim the dilution benefit of a single recharge area unless the area is determined to be large enough to meet the collective rather than individual dilution needs of the several systems.

E.4. Method to Determine Adequacy of Proposed Enhanced Treatment for a Subsurface Discharge

Step 1: Determine the available recharge acreage in each Hydrologic Soil Group (HSG) assignable to the site. Acreage must be excluded where future impervious runoff will not be allowed to infiltrate on site.

Step 2: Sum the average daily recharges available from the available acreage(s) in each Hydrologic Soil Group. To identify average daily recharge, use the following

equation:

Step 3: Complete the following equation: $Cq = 1,198,264 \times HE/R$

Where:

Cq = estimated effective average groundwater nitrogen concentration, in mg/L

H = proposed number of system users (including adults and children)

E = enhanced treatment function (reduction percentage expressed as a decimal)

R = estimated available onsite or permanently assignable annual recharge in gallons, and equal to annual inches (from Table E-3) times the acreage in a particular HSG times 27,152 (a unit conversion factor).

If the predicted Cq value is over 5 mg/l, additional or enhanced treatment is required before the system can be authorized.

If the predicted Cq value is below 5 mg/l, the project may continue through the review process.

Example 1: A 10 acre residential project is proposed in an area with 40 inches of annual precipitation. The site consists of 4 acres of HSG A and 6 acres of HSG B soils. 2 acres will be covered with roofs or paving, but all stormwater will be retained on site and allowed to infiltrate into HSG A soils. A subsurface disposal system is proposed with *enhanced treatment* to reduce nitrate-nitrogen by 50%. The proposed project population is 34 persons.

Step 1: 4 acres of HSG A soils, and 6 acres of HSG B soils.

Step 2: Using Table E-3:

R for A soils: 4 acres x 18.2" x 27,152 = 1,976,666 gallons/year

R for B soils: 6 acres x 13.3" x 27,152 = 2,166,730 gallons/year

Total R value = 4,143,396 gallons/year aquifer recharge

Step 3: $Cq = 1,198,264 \times HE/R$

$Cq = 1,198,264 \times (34)(0.5) / 4,143,396 = 4.92 \text{ mg/L}$

This project appears likely to be able to meet the groundwater quality protection target of 5 mg/L and so may proceed through the review process **using the proposed enhanced treatment program.**

Example 2: If a business employing 75 persons was proposed on this same site, with the same impervious surfaces, the equation would be modified to estimate groundwater quality, as follows:

Step 1 and 2: Unchanged procedure from Example 1, again identifying 4,143,396 gallons/year aquifer recharge, i.e. *Total R value*.

Step 3: If the 75 persons are employed full-time (2000 hours/year [40 hours x 50 weeks]), approximately 23% of the year [40 hours at work / 7 x 24 total hours], this may be entered into the formula as the *E-value* as a preliminary project review step. It is assumed that employees will release this fraction of their annual nitrogen-nitrate discharge in this location.

$$Cq = 1,198,264 \times 75(0.23) / 4,143,396 = 4.99 \text{ mg/L}$$

This second project appears likely to meet the groundwater quality protection target of 5 mg/L *without the need for enhanced septic treatment design*.

E.5. Distribution Networks

Methods of Distribution

There are three primary methods of distributing pretreated wastewater to subsurface treatment and disposal networks. Gravity distribution, or trickle flow is the most commonly used method for small systems. Pipe and ground slopes convey wastewater downstream from treatment components. Dosing, is also a non-pressurized method of distribution where the effluent is collected in a dosing chamber and a predetermined volume is discharged through the distribution piping by gravity, using siphons, or float/tipping devices. The third, pressurized distribution, is a variation where the effluent is collected in a dosing chamber and a predetermined volume is discharged through a network of small diameter piping under pressure, typically using pumps.

Dosing permits rapid distribution of septic tank/aerobic unit effluent throughout the soil treatment and dispersal system followed by a rest period during which no septic tank/aerobic unit effluent enters. Dosing is recommended on level sites to prevent premature clogging of portions of the absorption system, which can occur with gravity distribution. On restricted sites where usable soils are upslope from the septic tank, a “pump-to-gravity-flow” system can be designed where pumps dose 3- to 4-inch distribution lines. This is also non-pressurized dosing. A dosing system example follows the pressurized distribution example below.

Pressurized distribution provides a uniform application of the wastewater through-out the soil-based treatment area (STA). A pressurized distribution system outline and example follows the distribution network descriptions.

Dosing or pressurized distribution is required for:

- (1) large wastewater flows ($\geq 5,000$ gpd) in slowly permeable soils (slower than 60 min/inch);
- (2) alternative systems (e.g.: raised systems, mounds, and sand or media filters) with large wastewater flows ($\geq 5,000$ gpd) or where even distribution is critical to the performance of the system (e.g.: sloped sites);
- (3) site conditions where a gravity system cannot provide even distribution (e.g.: rapidly permeable soils with percolation rates less than 10 min/inch; or,
- (4) systems with a total absorption trench length exceeding 500 feet or if laterals are over 100 feet long (also see *Section E.9 Absorption Trenches/Beds*).

Note: Dosing or pressurized distribution is recommended for all soil treatment and dispersal systems since it promotes improved treatment of wastewater and system longevity as compared to gravity flow systems.

Distribution Network Types

- 1) Single-line networks can be used in trenches with gravity flow or dosing. If the line is greater than 100 ft. long, the inlet from the pretreatment unit should be at the midpoint of the line. No line shall be longer than 200 feet. (Also see *Section E.9 Absorption Trenches/Beds*.)
- 2) Closed-loop networks can be used for bed or trench systems with the infiltrative surface all at one elevation. Spacing between lines in closed-loop systems should be at least 3 feet (versus 6 feet-on-center for single line networks). Dosing of these systems is strongly recommended.
- 3) Distribution boxes can be used for gravity flow or dosing in multi-trench or bed systems, but

should be restricted to sites with slopes less than 5 percent. Distribution boxes shall not be used with pressure distribution. See Section D.9, Distribution Boxes and Flow Splitters for d-box dimensions and installation.

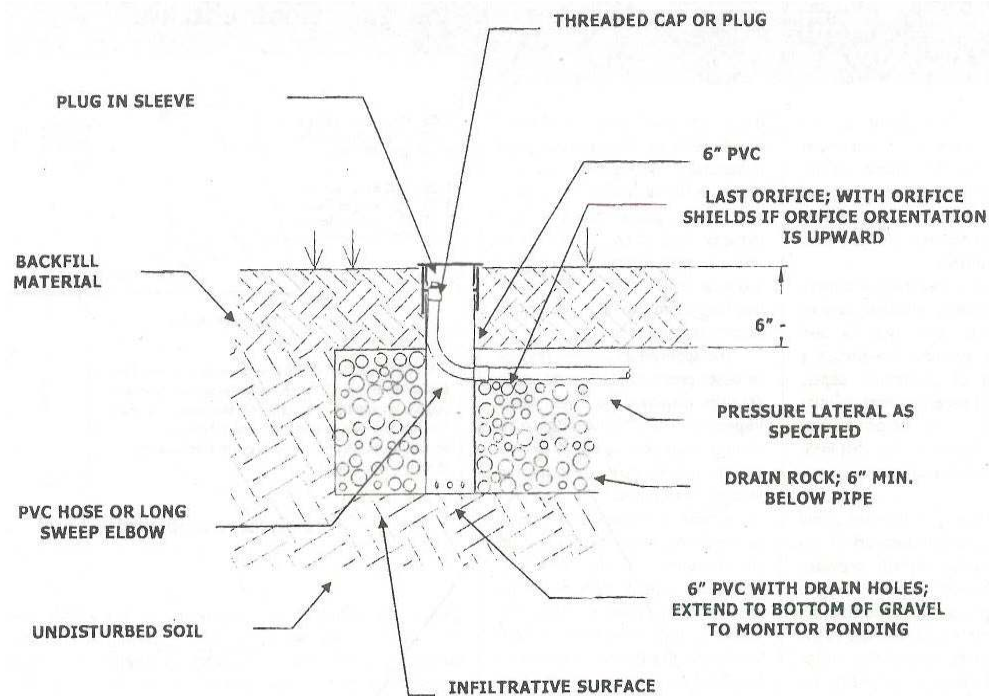
- 4) Drop boxes, or relief line systems using sloped piping rather than vertical drop boxes, are recommended for gravity distribution, or dosing, on slopes greater than 5 percent, as these are less likely to result in surface seepage than distribution boxes. The invert of the overflow should be at the same level, or up to 2 inches above the crown of the distribution line outlet. The inlet invert should be at the same level or above the overflow.

For commercial, institutional and multi-family homes with design flows under 1,000 gpd, drop boxes, or relief lines, may be used for septic tank effluent pipe and non-perforated distributor pipe on slopes greater than 10 percent, per NYSDOH Design Handbook: http://www.health.state.ny.us/environmental/water/drinking/wastewater_treatment_systems/design_handbook.htm.

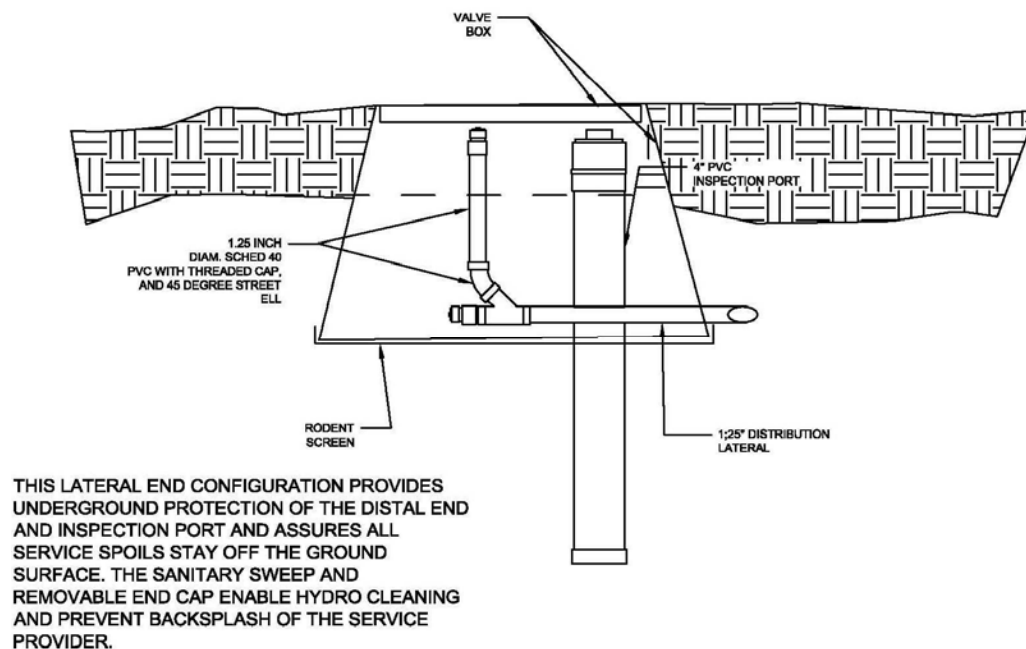
- 5) The piping network used with pressurized distribution differs from those used with gravity distribution and dosing in that it uses smaller diameter piping. Pressurized distribution uses small diameter pipe with perforations. The volume of water that flows out each hole must be approximately equal. This requires that 75 to 85 percent of the head loss in the network must be lost when the water passes through the holes. On sloped sites, the difference in total head within each lateral must be taken into account.

Hole-size should be within the range of 1/8-inch to 5/8-inch; the maximum allowable hole-spacing is 10 feet, but no more than 6 feet is recommended. 1/8-inch holes depend on a well-maintained effluent filter and all burrs removed from drilled holes. The perforation at the end of each lateral should be drilled horizontally in the end-cap near the crown of the pipe to facilitate venting. In beds with pressure distribution the lateral spacing should be approximately equal to the perforation spacing, and holes on adjacent laterals should be staggered so that they lie on the vertices of equilateral triangles.

Root penetration may be reduced and soil aeration increased by orienting the orifices to spray upwards into orifice shields, or inverted half-pipes, or by “sleeving” the small diameter pipe into a 4” pipe. Monitoring and clean-out ports shall also be used with pressurized distribution networks. *See Figure E-1.*



MONITORING PORT DETAIL I



MONITORING PORT DETAIL II

Figure E-1. Monitoring and Clean-out Ports

Below is a simplified design procedure for rigid pipe pressure networks as presented by Richard Otis (1982) as cited in the 2002 EPA OWTS Manual, page 4-27:

1. Lay out the proposed network.
2. Select the desired orifice size and spacing. Maximize the density of orifices over the infiltration surface, keeping in mind that the dosing rate increases as the orifice size increases and the orifice spacing decreases.
3. Determine the appropriate lateral pipe diameter compatible with the selected orifice size and spacing using a spreadsheet or sizing charts from Otis (1982).
4. Calculate the lateral discharge rate using the orifice discharge equation $Q = 0.8 (11.79 d^2 h_d^{1/2})$, where Q equals the discharge in gallons per minute, d equals the orifice diameter in inches, and h is the operating pressure in feet of water.
5. Determine the appropriate manifold size based on the number, spacing, and discharge rate of the laterals using a spreadsheet or sizing table from Otis (1982), Berkowitz (1985) as cited in the 2002 EPA OWTS Manual, pages 4-25 and 4-27.
6. Determine the dose volume required. Use either the minimum dose volume equal to 5 times the network volume or the expected daily flow divided by the desired dosing frequency, whichever is larger.
7. Calculate the minimum dosing rate (the lateral discharge times the number of laterals).
8. Select the pump based on the required dosing rate and the total dynamic head (sum of the static lift, friction losses in the forcemain to the network, and the network losses, which are equal to 1.3 times the network operating pressure).

Note: If duplicate pumps are not provided, the dosing chamber should have a reserve capacity above the active dosing volume equal to one day's average flow.

E.6. Pressure Distribution Design Example

Design a pressure distribution network for a mound as described in **Pressure Distribution Network Design**, (Converse, 2000)². The absorption area is 113 ft long by 4 ft wide. The force main is 125 ft long and the elevation difference is 9 ft with three 90° elbows.

Steps in the Design of a Pressure Distribution Network

1. Configuration of the network.

This is a narrow absorption unit on a sloping site.

2. Determine the lateral length.

Use a center feed, the lateral length is:

$$\begin{aligned}\text{Lateral Length} &= (B / 2) - 0.5 \text{ ft} \quad \text{Where: } B = \text{absorption length.} \\ &= (113 / 2) - 0.5 \text{ ft} \\ &= 56 \text{ ft}\end{aligned}$$

3. Determine the perforation spacing and size.

Perforation spacing -

Each perforation covers a maximum area of 6 ft². The absorption area is 4 ft. wide.

² Converse, J.C., 2000. *Pressure Distribution Network Design*. Small Scale Waste Management Project. SSWMP guidance document #9.14. 345 King Hall, University of Wisconsin-Madison, 1525 Linden Drive, Madison, WI 53706.

James C. Converse, Professor, Biological Systems Engineering, University of Wisconsin-Madison. Member of Small Scale Waste Management Project. Funded by Small Scale Waste Management Project. www.wisc.edu/sswmp/online_publications.htm.

Option 1: Two laterals on each side of the center feed

$$\begin{aligned}\text{Spacing} &= (\text{area/orifice} \times \text{no. of laterals}) / (\text{absorption area width}) \\ &= (6 \text{ ft}^2 \times 2) / (4 \text{ ft.}) = 3 \text{ ft.}\end{aligned}$$

Option 2: One lateral down the center on each side of the center feed:

$$\text{Spacing} = \text{area per orifice} / \text{width of absorption area} = 6 \text{ ft}^2 / 4 \text{ ft.} = 1.5 \text{ ft}$$

Best option:

Ideally, the best option is to position the perforations to serve a square but that may be difficult to do. In Option 1, each perforation serves a 2' by 3' rectangular area while in option 2, each perforation serves a 1.5 by 4 area. With an absorption area of 6 ft wide with one lateral down the center, perforation spacing would be 1 ft apart and the perforation would serve an area of 6 by 1 ft **which would be undesirable**. Laterals should be within 2.0 ft of the edge of the soil-based treatment system absorption area to eliminate designs laterals with spacing too close.

Perforation size:

Select from 1/8, 3/16 or 1/4". Use 3/16" as per three-paragraph discussion in the "Design Procedure" section of *Pressure Distribution Network Design* (WI document outline item A.3).

4. Determine the lateral diameter.

Using Fig. A-2a & 2b (for 3/16" orifices) in the "Design Procedure" section, outline item A-3 of *Pressure Distribution Network Design*:

Option 1: For two laterals on each side of the center feed and lateral length of 56 ft and 3.0 ft spacing, the lateral diameter = 1.5"

Option 2: For one lateral on each side of center feed and lateral length of 56 ft and 1.5 ft spacing, the lateral diameter = 2".

5. Determine number of perforations per lateral and number of perforations.

Option 1: Using 3.0 ft spacing in 56 ft yields:

$$N = (p/x) + 0.5 = (56 / 3.0) + 0.5 = 19 \text{ perforations/lateral}$$

$$\text{Number of perforations} = 4 \text{ lateral} \times 19 \text{ perforations/lateral} = 76$$

Option 2: Using 1.5 ft spacing in 56 ft yields:

$$N = (p/x) + 0.5 = (56 / 1.5) + 0.5 = 38 \text{ perforations/ lateral}$$

$$\text{Number of perforations} = 2 \text{ laterals} \times 38 \text{ perforations/lateral} = 76$$

Check - Maximum of 6 ft² / perforation =

$$\text{Number of perforations} = 113 \text{ ft} \times 4 \text{ ft} / 6 \text{ ft}^2 = 75 \text{ so ok.}$$

6. Determine lateral discharge rate (LDR).

Using network pressure (distal) pressure of 3.5 ft and 3/16" diameter perforations,

Table A-1 gives a discharge rate of 0.78 gpm regardless of the number of laterals.

$$\text{Option 1: LDR} = 0.78 \text{ gpm/ perforation} \times 19 \text{ perforations} = 14.8 \text{ gpm}$$

$$\text{Option 2: LDR} = 0.78 \text{ gpm/ perforation} \times 38 \text{ perforation} = 29.6 \text{ gpm}$$

7. Determine the number of laterals.

This was determined in Step 3 and 4.

Option 1: Two laterals on each side of center feed = 4 laterals spaced 2 ft apart.

Option 2: One lateral on each side of center feed = 2 laterals down center of absorption area.

8. Calculate the manifold size.

Option 1: The manifold is same size as force main as it is an extension of the force main or it could be one size smaller. For larger systems, there is a table available by Otis, 1981 and in the Wisconsin Administrative Code.

Option 2: There is no manifold.

9. Determine network discharge rate (NDR)

$$\text{Option 1: NDR} = 4 \text{ laterals} \times 14.8 \text{ gpm/lateral} = 59.2 \text{ or } 60 \text{ gpm}$$

$$\text{Option 2: NDR} = 2 \text{ laterals} \times 29.6 \text{ gpm/lateral} = 59.2 \text{ or } 60 \text{ gpm}$$

Pump has to discharge a minimum of 60 gpm against a total dynamic head yet to be determined.

10. Total dynamic head.

Sum of the following:

$$\begin{aligned}\text{System head} &= 1.3 \times \text{distal head (ft)} \\ &= 1.3 \times 3.5 \text{ ft} \\ &= 4.5 \text{ ft}\end{aligned}$$

Elevation head = 9.0 ft (Pump shut off to network elevation)

Head Loss in Force Main => Use Table A-2 for plastic pipe friction loss, and A-3 for friction losses through plastic fittings, in **Pressure Distribution Network Design** for 60 gallons per minute and 125 ft of force main and 3 elbows.

Equivalent length of pipe for fittings - Table A-3 in *Pressure Distribution Network Design*

Option A: 2" diameter force main = 3 elbows @ 9.0 ft each = 27 ft of pipe equivalent.

Option B: 3" diameter force main = 3 elbows @ 12.0 ft each = 36 ft

Head Loss - Table A-2 in *Pressure Distribution Network Design*.

Option A: 2" diameter force main = $7.0 (125 \text{ ft} + 27 \text{ ft})/100 = 10.6 \text{ ft}$

Option B: 3" diameter force main = $0.97(125 \text{ ft} + 36 \text{ ft})/100 = 1.6 \text{ ft}$

Total Dynamic Head (TDH)

Option A: TDH = $4.5 + 9 + 10.6 = 24.1 \text{ ft}$ (2" force main)

Option B: TDH = $4.5 + 9 + 1.6 = 15.1 \text{ ft}$ (3" force main)

11. Pump Summary

Option A: Pump must discharge 60 gpm against a head of 24.1 with 2" force main.

Option B: Pump must discharge 60 gpm against a head of 15.1 ft with 3" force main.

These are the calculated flow and head values. The actual flow and head will be determined by the pump selected. A system performance curve plotted against the pump performance curve will give a better estimate of the flow rate and total dynamic head the system will operate under. The next section gives an example.

E.7. Design of the Force Main, Pressurization Unit, Dose Chamber and Controls Steps

1. Calculate the system performance curve.

Use the following table from *Pressure Distribution Network Design* to develop a system performance curve. Follow procedures (a) through (g) listed below the table. Orifice is synonymous to perforation. *This example uses Option A. Option B can be calculated similarly.*

Pump System Curve Table

Total Flow	Orifice Flow	Elevation Difference	Force Main	Network Head	Total Head
----- (gpm)-----	----- (gpm)-----	----- (ft)-----	----- (ft)-----	----- (ft)-----	----- (ft)-----
40	0.526	9	5.0	2.1	16.1
50	0.658	9	7.6	3.3	19.9
60	0.789	9	10.6	4.7	24.3
70	0.921	9	14.2	6.4	29.6
80	1.053	9	18.1	8.4	35.5

Procedure:

- Select 5 flow rates above and below the network discharge rate of 60 gpm.
- Calculate the orifice (perforation) flow rate for each of the flows. This is done by dividing the flow rate by the number of orifices in the network. For the 30 gpm and 76 orifices, the orifice flow rate is 0.395

gpm.

c. The elevation head is the height that the effluent is lifted.

d. The force main head is the head loss in the force main for the given flow rate.

Table A-2 in *Pressure Distribution Network Design* gives the friction loss. Need to select a force main diameter. For this example use 2" force main. For the 60 gpm the friction loss is 7.0 ft x 1.52 for head of 10.6 ft.

e. The network head is calculated by $H = 1.3(Q/(11.79d_2))^2$. H is head in ft, Q is orifice flow rate in gpm, and d is orifice diameter in inches. The 1.3 is an adjustment factor for friction loss in laterals. For 3/16" diameter orifice the equation is $H = 1.3(Q/0.4145)^2$.

f. The total head is the sum of the elevation, force main and network heads.

2. Determine the force main diameter.

Force main diameter:

Option A: = 2" (determined in Step 1 of Section B).

Option B: = 3"

3. Select the pressurization unit.

Plot the performance curves of several effluent pumps and the system performance curve (Figure A-4 in *Pressure Distribution Network Design*). For the system curve plot the flow rates vs. the total head. On the system curve, using an X where the flow rate intersects the curve (in this case 60 gpm). Select the pump, represented by the pump performance curve, located next along the system performance curve just after 60 gpm (Pump B) as that is where the pump will operate.

Pump C could be selected but it is over sized for the unit.

4. Determine the dose volume.

More recent thinking is that the dose volume should be reduced from the larger doses recommended earlier.

Use 5 times the lateral void volume. Use void volume from Table A-4 in *Pressure Distribution Network Design*.

	Option 1:	Option 2:
Lateral diameter =	1.5"	2.0"
Lateral Length =	56'	56'
No. of laterals =	Option 1: 4	Option 2: 2
Void volume =	0.092 gal/ft	0.163 gal/ft

Net dose volume

Option 1: $= 5 \times 56 \times 4 \times 0.092 = 103 \text{ gal./dose}$

Option 2: $= 5 \times 56 \times 2 \times 0.163 = 91.3 \text{ gal./dose}$

Flow back from force main

Option A: 2" force main @ 125 ft @ 0.163 gal./ft = 20.4 gal./dose

Option B: 3" force main @ 125 ft @ 0.367 gal./ft = 45.9 gal./dose

Set the floats to dose the combination selected:

Dose volume with Option 1 and Option A = $103 + 20 = 123 \text{ gal./dose}$

Dose volume with Option 1 and Option B = $103 + 46 = 146$

Dose volume with Option 2 and Option A = $91 + 20 = 111$

Dose volume with Option 2 and Option B = $91 + 46 = 137$

The net dose volume to the mound will be 91 or 103 gal./dose with either 20 or 46 gallons flowing back into pump chamber. No check valve is used to prevent flow back in cold climates due to freezing potential. If the dose is limited to 20% of the design flow, Option 1 with net dose of 91.3 is very close to

90 gpdose (450 gpd x 20%). Option 2 does not meet the 20% criteria.

5. Size the dose chamber.

Based on the dose volume, storage volume and room for a block beneath the pump and control space, 500 to 750 gallon chamber will suffice. If timed dosing is implemented, then a larger tank will be required to provide surge storage. Use the daily design flow for surge capacity.

6. Select controls and alarm.

Demand Dosing: Controls include on-off float and alarm float. An event recorder and running time meter would be appropriate to install. If the pump is calibrated and dose depth recorded, these two counters can be used to monitor flow to the soil unit.

Time Dosing: The advantage of time dosing provides more frequent doses and levels out peak flows to the soil treatment/dispersal unit. In mounds with longer laterals and larger orifices, compared to shorter laterals and smaller orifices in sand filters, time dosing may not be as appropriate as it is in sand filters.

7. Select Effluent Filters.

Filters must be installed on the septic tank to minimize solids carry-over to the pump chamber. A second filter, located on the pump outlet, will keep any solids falling into the pump chamber from being carried over. Converse (1999) provides information relative to filters.

8. Construction and Maintenance

In addition to designing pressure distribution or dosing systems, a design professional should also supervise construction, prepare an operating manual, and implement start-up. Good common sense should prevail when constructing and maintaining these systems. Good quality components should be used. There is no lack of good components today. Water tight construction practices must be employed for all tanks. Surface runoff must be diverted away from the system. Any settling around the tanks must be filled with the soil brought to grade or slightly above to divert surface waters. Provisions must be incorporated

into the lateral design, such as turn-ups (Figure E-1), to provide for easy flushing of the laterals as solids will build up and clog the orifices. *DO NOT ENTER THE TANKS WITHOUT PROPER SAFETY EQUIPMENT.* For specific pressure distribution design details, including the development of a system performance curve, and lateral/orifice sizing Tables, see University of Wisconsin SSWMP guidance document by James C. Converse, cited on page E-13.

E.8. Design of Dosing Systems:

E.8.a. General

Care must be taken in sizing siphons. Siphons have discharge rates ranging from 25 to 3,000 gallons per minute (gpm). The pipe network receiving the dose from the siphon must be at an elevation four feet lower than the siphon.

The head that the network operates against has to be developed in the force main by backing effluent up in the pipe. If the discharge rate out the perforations is greater than the siphon flow rate, the distal pressure in the network will not be sufficient. Some manufacturers recommend that the force main be one size larger than the siphon diameter to allow the air in the force main to escape. However, this will reduce the distal pressure in the network which may be below the design distal pressure. Falkowski and Converse, 1988, discuss siphon performance and design.

The dosing siphon (or pump, or other dosing device) shall have a capacity sufficient to fill at least 75-percent of the interior volume of the distribution laterals lines being dosed.

A dosing device shall have a minimum storage capacity of 125 percent (preferably 200 percent) of the maximum rate of inflow to the dosing chamber.

Dosing siphons, pumps, tanks and devices shall be arranged for convenient access and inspection. If possible, dosing devices should be designed to function by gravity in case of mechanical or power failure.

If multiple dosing devices are used, valves and fittings shall be provided to allow each section of the system to be dosed by any one of the dosing devices.

Dosed systems (pump and siphon types) shall have provisions to prevent the flow of wastewater out of vents in the piping network.

The design of dosing tanks for domestic-type sewage shall, conform to the design standards of a septic tank (see Section D.6). Those standards specify, in part, that:

(a) The dosing station must be watertight and constructed of materials that will not corrode or decay (ASTM or IAPMO standards);

(b) A manhole/access hatch must be installed to-grade and the access opening must be lockable or securely fastened to prevent unauthorized entry; and

(c) If the tank is to be installed where the water table is high, buoyancy should be taken into account. Dosing tanks or devices shall be arranged for convenient access and inspection. If possible, dosing devices should be designed to function by gravity in case of mechanical or power failure.

If multiple dosing devices are used, valves and additional pipe and fittings shall be provided to allow each section of the system to be dosed by any one of the dosing devices.

Dosed systems (pressure and siphon types) shall have provisions to prevent the flow of wastewater out of vents in the piping network.

E.8.b Dosing System Design Example (sample calculation of siphon dose volume)

Given: a treatment and dispersal field with 240 linear feet of 4-inch PVC pipe, and a pipe-network volume to be dosed to 75% -85% full.

At 75% and 85% full, four (4) inch pipe contains 0.537 gallons per foot and 0.607 gallons per foot respectively.

Total Network Volume (75%) = 240 lineal feet * 0.537 = 128.88 gallons

Total Network Volume (85%) = 240 lineal feet * 0.607 = 145.68 gallons

Dose Volume shall be between 128.88 gallons and 145.68 gallons

Sizing the dosing chamber

The dosing network reserve capacity requirements should be based on the type of facility being serviced and whether or not dual pumps are used. With a duplex pump system, a limited reserve capacity is necessary. With a single pump system, it is recommended that the reserve capacity be not less than 25% of the daily average flow. The volume **below** the working level shall include an allowance for the volume of **all drainage** which may flow back to the chamber when pumping has ceased.

The pump dosing chamber (Fig. A-5 in *Pressure Distribution Network Design*) must be large enough to provide:

- a. The dose volume.
- b. The dead space resulting from placement of the pump on a concrete block.
- c. A few inches of head space for floats
- d. Reserve capacity ($\geq 25\%$ of the average daily flow)

For systems using dosing with gravity flow through the network, either a pump or dosing siphon may be used and shall have a capacity sufficient to fill at least 75 percent of the interior volume of the lines being dosed.

Dosing Chamber Effluent Flow requirements

A dosing device should have a capacity of at least 125 percent, and preferably 200 percent, of the maximum rate of inflow to the dosing chamber.

The 'maximum rate of inflow to the dosing chamber' means the peak flow being delivered to the dosing chamber by the preceding system device, i.e. an equalization tank, a polishing filter following an RBC or filter for BOD removal. The design engineer needs to determine what the flow entering the dose chamber would be. This would usually be in gpm - not peak gallons per hour – over the delivery period. The dosing chamber capacity must exceed that volume to prevent it from being flooded. The outflow rate of the dose (distribution rate) has to be significantly larger than the inflow (or delivery) rate.

For example:

A children's camp has a daily design flow of 5,000 gallons and a peak flow of 2,000 gallons over a one hour period. What is the maximum rate of inflow?

A: $2,000 \text{ gallons}/60 \text{ minutes} = 33 \text{ gpm} \times 1.25 \text{ to } 2$

Using the given example, the 2,000 gals over an hour equals 33 gpm. Therefore, the dosing rate has to be at least 42gpm, and preferably 67 gpm, assuming that the 33 gpm flow actually is the rate going to the chamber. If there are septic tanks, or some other unit ahead of the dosing chamber, the flow peaks may be attenuated, and a lower inflow figure used. The device that removes wastewater from the dosing chamber should always be able to remove it faster than it comes in. So, the dosing device design factor would be a peak or maximum flow rate. If the maximum inflow rate is x gallons per minute, the outflow pump should be able to distribute at least x times 1.25 gallons per minute to the soil treatment area.

Siphon-dosing (or pump dosing) **absorption trenches:**

Absorption trenches totaling over 500 feet require dosing.

Absorption trenches totaling over 1,000 feet require alternate dosing.

Maximum length of absorption lines used in either case shall be 100 feet.

Alternating Siphon-dosing:

Absorption systems requiring 1000 feet of trench shall be constructed in two or more sections such that no section contains over 1000 feet of pipe.

The capability to dose each section separately shall be provided.

Siphon-dosing *absorption beds:*

Maximum lateral length shall be 75 feet.

Dosing frequency based upon absorption bed soil type.

Minimum dosing of 3times per day is recommended.

Dose volume is equal to 75% to 85% of the volume of the lateral pipes.

Siphon-dosing of mounds is prohibited.

Siphon-dosing (or pressure distribution) is recommended for NYSDEC *Fill Systems, Cut and Fill Systems, and Raised Systems* under 5,000 gpd. Siphon-dosing (or pressure distribution) is required for NYSDEC Fill Systems with design flows of 5,000 gpd or greater. See **Sections E.12 and E.13** for further information on fill/raised systems, and mound systems, respectively.

For non-pressurized systems (gravity distribution and dosing), 4-inch perforated plastic pipe is recommended. A wide variety of PVC (ASTM D2665), or ABS (ASTM D2661) pipe designated by ASTM as sewer or drainpipe may be considered.

A pressurized system should use 1 to 3 inch PVC (ASTM D 1785, ASTM D 2241, and PVC 1120 and PVC 1220) or HDPE pipe. A minimum in-line pressure of 2.5 feet will allow for deviations from level of up to 3 percent. Pressurized system must be placed so that the laterals and manifold drain after every dose.

E.9. Absorption Trenches/Beds

Absorption trenches/beds are used to distribute sewage over a wide area to enhance treatment and dispersal by seepage into the ground. Installations with more than 1000 feet of trench shall be constructed in two or more sections such that no section contains over 1000 feet of pipe. The capability to dose each section separately shall be provided. See Dosing System Design Example above.

A typical absorption trench layout is shown in Figure E-2. Absorption beds are similar to absorption trenches, except that they are wider, and contain more than one distribution lateral as shown in Figure E-3.

Trenches are strongly recommended instead of beds, especially in soils with high clay content or where groundwater flow patterns are horizontal. Long distribution laterals parallel to the site contour are recommended. Trenches may be placed on slopes of up to 20%, but beds should be limited to sites with slopes no greater than 5%. If it can be demonstrated that the site has the necessary assimilative capacity, trench systems may be built on slopes steeper than 20%. Under these conditions, special construction techniques (e.g. terracing or hand digging) may be necessary.

Trenches must be sized in accordance with the application rates given in Table E-1 above.

Application rates for absorption beds shall be no more than 75% of the rate allowed for trenches on the same site.

The effective area for wastewater application will be the bottom area (only) of the trenches or beds.

The design trench width shall be 2 feet, while the design bed width shall be a minimum of 3 feet. The

maximum width of an absorption bed should be 10 to 15 feet depending on the soil characteristics (e.g. the firmer the soil consistence [silty/clayey], the narrower the bed). Trench depth should be between 18 and 30 inches; in most cases depths of less than 24-inches are preferred. NYSDOH allows single-family homes and duplexes to have absorption bed systems built 20 feet wide http://www.nyhealth.gov/regulations/nycrr/title_10/part_75/appendix_75-a.htm; commercial and institutional systems should all be 10 to 15 feet.

See Section D.12 for greywater irrigation trench systems.

Traditionally, severe restrictions have been imposed upon trench length because of fears of root penetration, uneven settling, or pipe breakage disrupting the flow. However it has been shown that the aggregate actually transmits the wastewater, and so laterals up to 100 ft. shall be considered acceptable. Longer lengths may be permitted if site conditions allow. Pressurized distribution or dosing are recommended if laterals are longer than 60 feet, and shall be required if laterals are longer than 100 feet.

The minimum distance between walls of adjacent trenches should be 4 feet. Separations of 6 feet are desirable to provide some additional aerated soil for the horizontal flow component of sewage between trenches. Trenches must be laid out parallel to the site contours. When stepped trenches are used, it is important that the first length of all distribution lines leading from the distribution box to the trenches be laid with the same slope. The bottom of the trench shall be dug level in the longitudinal and transverse directions, and should be raked prior to placement of the washed gravel or crushed stone. In a dosed or pressurized trench system the distribution lines shall be level; otherwise a slope of between 1/32 and 1/16 inch/foot (0.25 to 0.5%) should be maintained. The ends of the laterals must be capped.

For beds, center-to-center spacing of distribution pipes shall be 5 feet or greater. The floor of the bed and the distribution lines shall be level and the ends of the laterals shall be interconnected. Distribution systems should be hand-leveled. At least 6-inches of graded gravel shall be placed beneath the distribution pipes, and an additional 2 inches shall be placed above the pipes. A barrier material must be placed above the stone to prevent the backfill from clogging the aggregate. This material may be synthetic drainage fabric (permeable geotextile) or for systems under 1,000 gallons per day, untreated building paper. Backfill over the barrier material shall be at least 6-inches and no more than 12-inches deep, and should consist of natural soil.

Absorption trenches/beds shall not be built under paved areas. Also, every effort should be made during construction to avoid smearing or compacting the bottom area or sidewalls.

Backfilling should be done carefully to avoid pipe breakage.

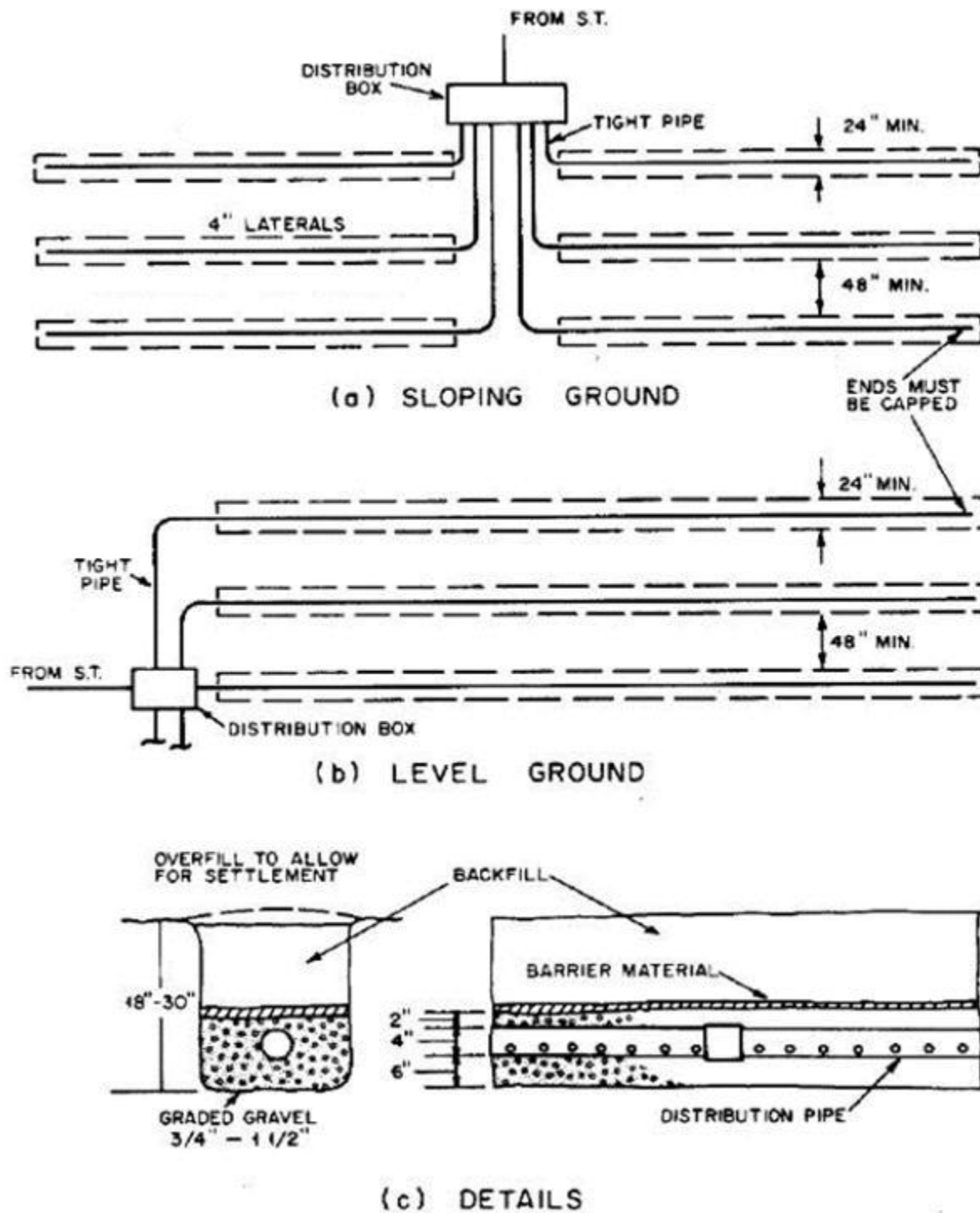


Figure E-2. Conventional Absorption Trench

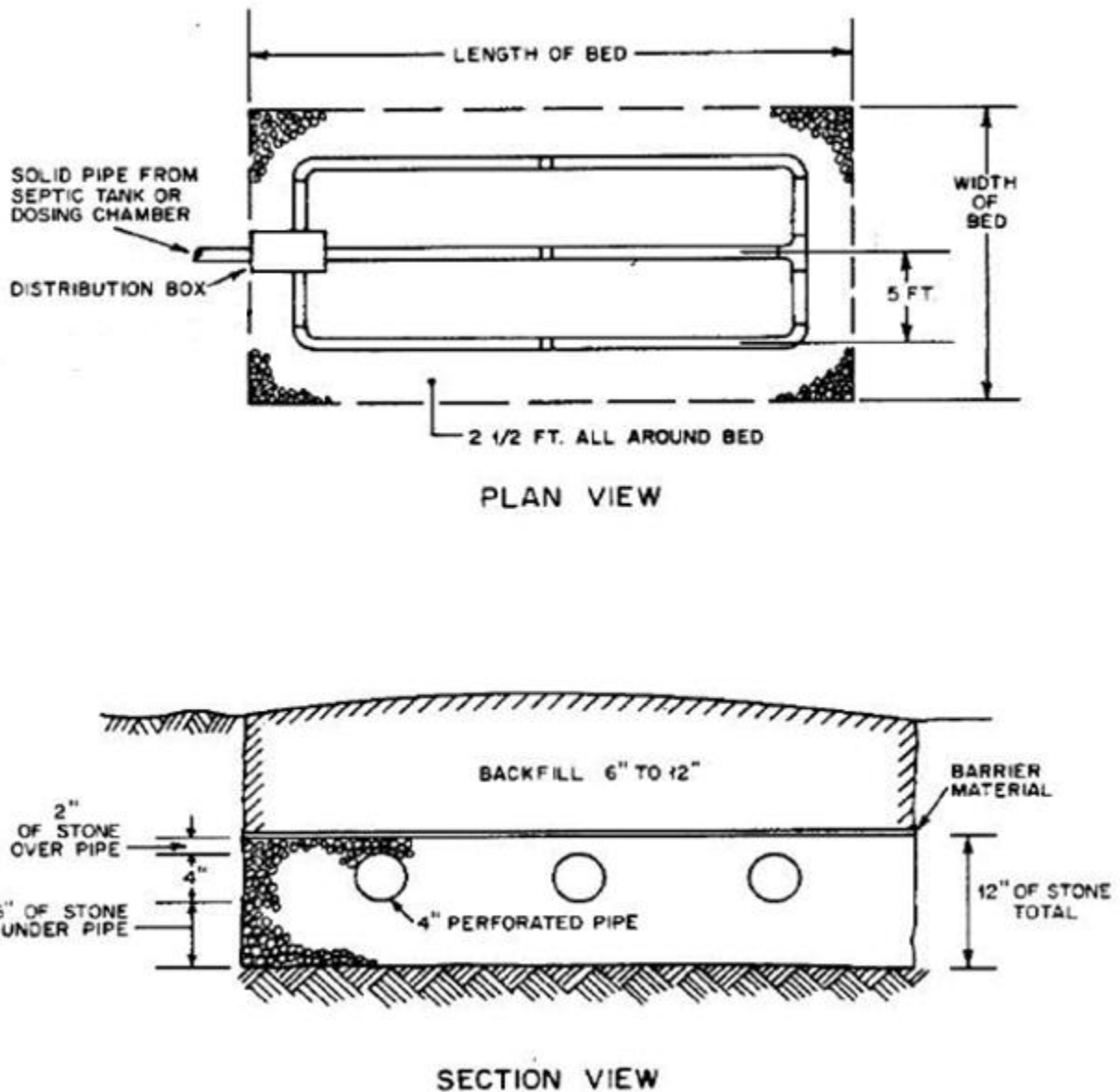


Figure E-3. Conventional Absorption Bed

Reserve Area

Consideration should be given to constructing the absorption area in three sections, with each section capable of handling 50 percent of the design flow. The third section should be alternated into service on a semiannual or annual schedule. This will extend the life of the system, and provide a standby unit in case of failure. The reviewing engineer may allow a 20 percent reduction in total absorption area for alternating systems. A valving system for a three-bed or three-field soil-based treatment system is shown

in Figure E-4. If the three-section option is not chosen, then the full-field (100%) replacement area (Figure E-5) will be required.

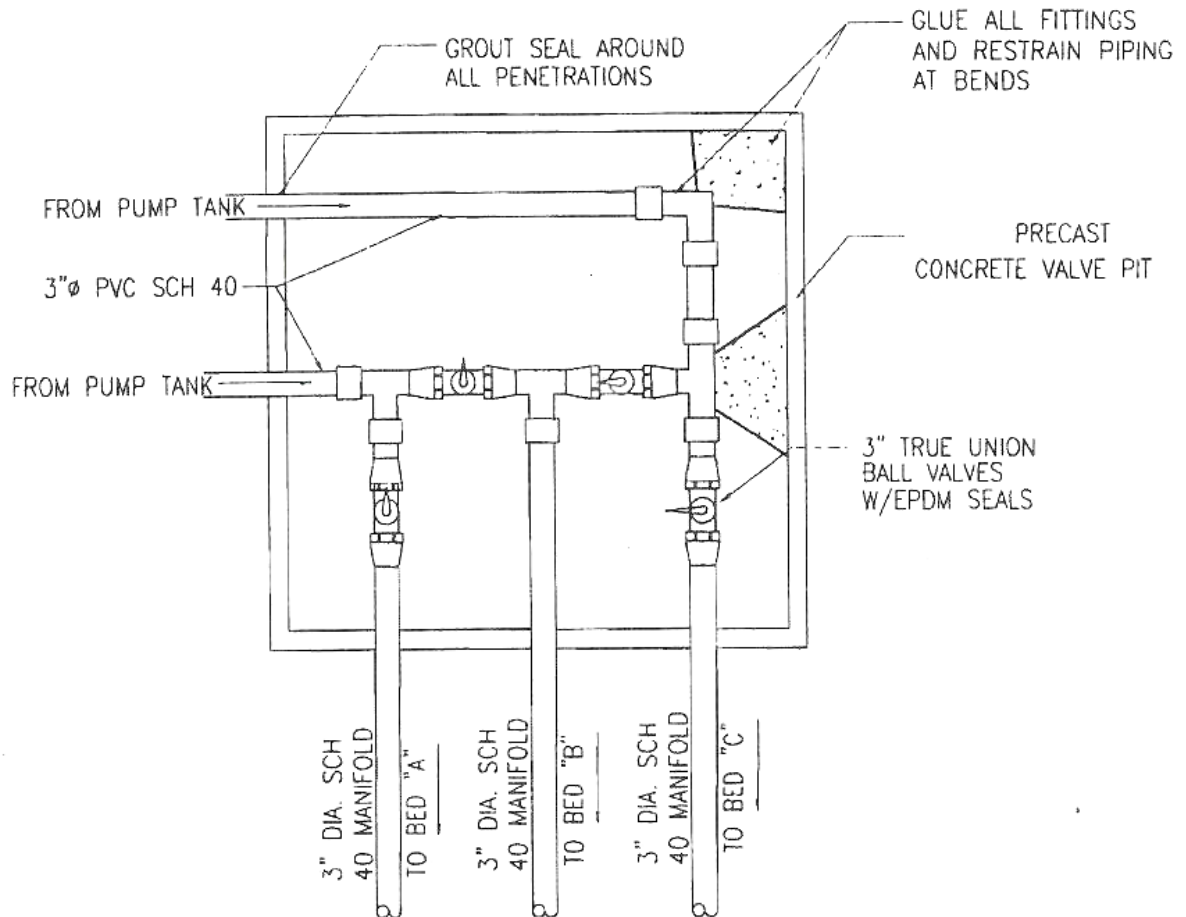


Figure E-4 Typical Valving for Three-Bed or Three-Field Soil-based Treatment Systems

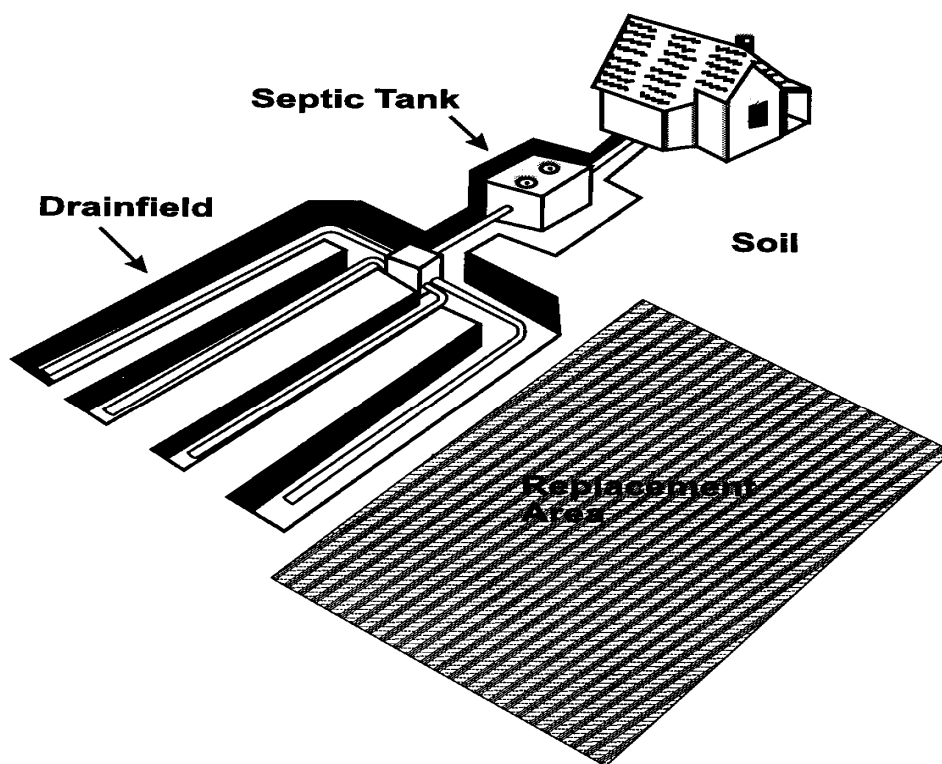


Figure E-5 Full-field Reserve Area

E.10. Shallow Absorption Trenches and Shallow Narrow Drainfields

Shallow Absorption Trenches

Shallow absorption trenches are particularly useful in areas where permeable soil is present above moderately high groundwater, porous or creviced bedrock, and/or an impermeable layer. The site evaluation (see Section B) must show that there will be sufficient depth to these boundary conditions from the bottom of the proposed system. Construction and sizing shall be the same as for conventional absorption trenches, except that the trench shall be only 6 to 12 inches deep as shown in **Figure E-6**. Backfill above the system should be of, or similar to, the native soil. Side slopes of the resulting hummock shall be no steeper than 3 feet horizontal to 1 foot vertical.

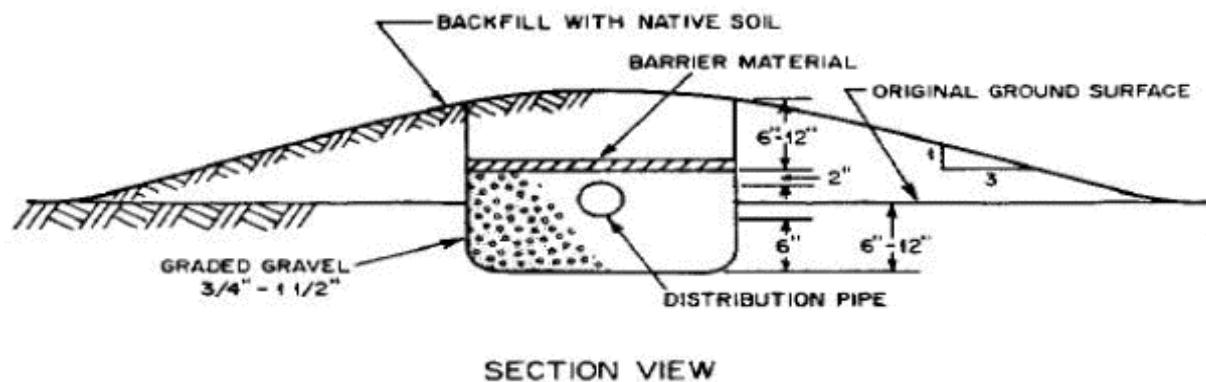


Figure E-6. Shallow Absorption Trench

Shallow Narrow Drainfields

Where the required depths of usable soil as given in Section B of this document are not available, a Shallow Narrow Drainfield system may be allowed where the wastewater is pretreated biologically (secondary and tertiary treatment as discussed in Sections F and G) and distributed using pressure distribution (see Pressure Distribution Network Design in Section E-3).

Criteria for a shallow narrow drainfield system:

1. The treated effluent from a secondary or advanced/tertiary treatment unit shall meet SPDES surface discharge quality: 30 mg/l BOD and 30 mg/l TSS.
The maximum FOG concentration must be less than 5 mg/l.
2. The shallow, narrow drainfield pipes shall be parallel and located 2.5' on center.
Without disinfection, a 2 foot minimum vertical separation is required to the seasonally high water table.
3. The discharge into the soil at any percolation rate will further polish the treated effluent. Fast percolating soils, unless saturated, will attenuate the pathogens and viruses when allowed a two (2) year time of travel to the well source. Distances to any down-gradient well sources should be determined. Using a modification of Darcy's Law for travel time, determine the length of time effluent will take to travel through the soil to the well source; if less than 2 years, disinfection shall be required.

4. Virus and pathogen protection may be provided by UV disinfection before discharge to the soil-based treatment system. If disinfection is provided a 1.5 foot minimum vertical separation is required to the seasonally high water table.

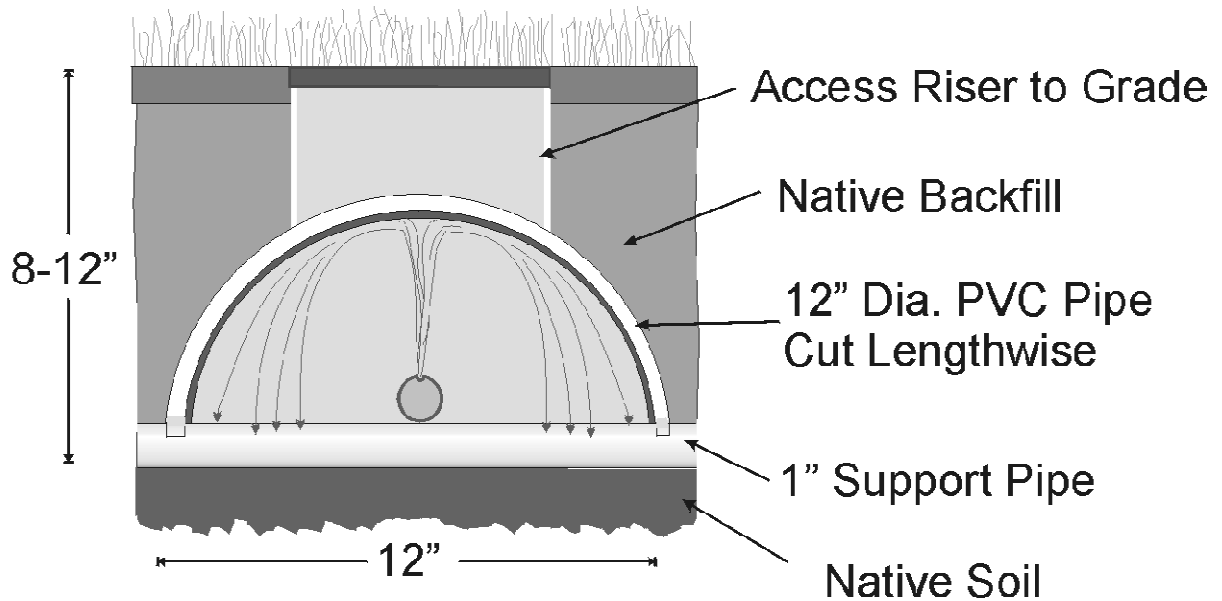


Figure E-7. Pressurized Shallow-Narrow Drainfield (cross section)

E.11. Gravelless (Aggregate-Free) Absorption Systems and Alternative Aggregates

Gravelless absorption systems are designed to receive wastewater from a septic tank or other treatment unit and transmit it into the soil for additional treatment and dispersal. All requirements of Sections E.1 through E.3, including the ADVISORY FOR FAST SOILS IN SPECIFIC AQUIFER AREAS are also applicable to all gravelless absorption systems. Allowable soils for gravelless absorption systems include those soils suitable for conventional, shallow-trench, or shallow narrow (pressurized) drainfields with the specified degree of pretreatment (Sections E.9 and E.10). All gravelless systems must be capable of withstanding typical construction equipment and residential use loads without deformation.

E.11.a. No Reduction in Field Size Allowed for New or Expanding Facilities

Gravelless wastewater distribution products used to serve facilities where no reduction in field size is allowed must meet the design, sizing, material and construction specifications of Subsection E.11.c of these Standards, with the exception that no reduction in field size shall be granted. Pretreatment is

required to be filtered septic tank effluent or better

E.11.b Allowance for Reduction in Field Size for Renovations on Existing Properties

Gravelless wastewater distribution products used to serve facilities where a reduction in field size may be allowed must meet the design, sizing, material and construction specifications of Section E.11.c of these Standards, or an alternative design shall be provided by the design engineer. Designs proposing to use gravelless products receiving secondary treatment or greater, proposing to use proprietary products not meeting the specifications of Section E.11.c, or proposing reduced drainfield sizing will be judged on a case-by-case basis by the reviewing engineer. *The department's regional office having jurisdiction shall be contacted prior to construction regarding the acceptability of specific products for use as a gravelless distribution system.*

E.11.c Specifications for Gravelless (Aggregate-Free) Absorption Systems

As with any soils treatment area, consideration should be given to constructing gravelless soil treatment areas in three sections, with each section capable of handling 50 percent of the design flow. The third section should be alternated into service on a semiannual or annual schedule. This will extend the life of the system, and provide a standby unit in case of failure. The reviewing engineer may allow a 20 percent reduction in the total soil treatment area for alternating systems.

(i.) Open-bottom gravelless chambers, galleys, or flow diffusers may be installed without aggregate backfill. One linear foot of these products shall be equivalent to one linear foot of conventional (24 inch wide) absorption trench. The product shall have:

- (1.) a minimum open-bottom infiltration area of 1.6 square feet per linear foot;
- (2.) a minimum volumetric capacity of 7.5 gallons per linear foot, and
- (3.) an open sidewall area for aeration and infiltration.

In very sandy soil, product may need a water-permeable, geo-textile fabric, to allow wastewater effluent to disperse and eliminate sand accumulation in the excavated trench (follow manufacturer's recommendations).

(ii) Gravelless media-wrapped corrugated pipe sand-lined systems may be installed. One linear foot of these products shall be equivalent to one linear foot of conventional (24 inch wide)

absorption trench. The product shall be:

- (1.) Comprised of corrugated plastic pipe with a minimum diameter of 12 inches;
- (2.) Wrapped in a media that allows wastewater dispersal and prohibits sand infiltration; and
- (3.) Installed with a minimum of six inches of washed concrete sand surrounding the pipe or pipe assembly (sand that meets NYSDOT specification 703-07 is recommended).

(iii) Gravelless geotextile “sand” filters may be installed. Soil treatment area designs may use a trench bottom sizing criteria of up to 6.0-square feet per linear foot of trench when the product demonstrates the following pre-manufactured features and installation criteria:

- (1.) A minimum unit width of three (3) feet;
- (2.) A minimum unit storage capacity of 12 gallons per linear foot;
- (3.) A minimum of six (6) square feet per linear foot of bio-substrate surface area per linear foot of unit, and
- (4.) A minimum of six (6) inches of sand installed below and on the sides of each unit (sand that meets NYSDOT specification 703-07 is recommended).

(iv) Site requirements: These systems shall be used on sites that have been classified as having a design percolation rate of one to sixty (1-60) minutes per inch, and meet the vertical and horizontal separation distances in Table 2 of Section B of these Standards.

(v) Construction requirements:

- (1) Gravelless trench sidewalls shall be separated by a minimum of four feet of undisturbed soil. For absorption bed applications, follow requirements and recommendations of Section **E.2 Application Rates** and **E.9 Absorption Trenches and Beds** in these Standards, and manufacturer recommendations.
- (2.) All gravelless trenches shall be equal in length. The total trench length shall be increased if necessary.

Proprietary Equipment

When proprietary gravelless equipment not conforming to the requirements in this subsection is proposed, e.g. low profile, low pressure mat systems, data which supports the capability of the equipment to meet structural and storage volume requirements under design conditions must be provided. Such equipment

shall be reviewed on a case-by-case basis at the discretion of the regional or local regulatory agency. A third-party certification (e.g. provided by NSF, International) of gravelless absorption systems or products would be a favorable factor in the determination. NSF, International has a gravelless system certification program (post-2010).

Concrete galley systems are prohibited.

E.11.d. Drip And Low-Profile Dispersal Systems

A subsurface drip dispersal system consists of a pretreatment system, a pump tank, filtration system, subsurface drip tubing, and controller. *Other low-profile dispersal systems operate similarly, but differ in the type of conduit and the orifice size, shape and spacing. The design professional should consult with the manufacturer of the specific product being considered.*

Appendix E includes detailed design guidance in support of what is given here. In addition, information is given on the installation, operation, maintenance and monitoring of drip dispersal systems specifically; several reference documents are also listed. Similar guidance applies to other low-profile dispersal products and may be obtained from manufacturers. TR-16 Section 9.4.2.3 Drip Disposal also refers to National Onsite Wastewater Recycling Association (NOWRA) *Recommended Guidance for the Design of Wastewater Drip Dispersal Systems*, 2006, as its primary reference.

For drip-dispersal systems, the pump tank stores the effluent until the controller turns on the pump to dose pretreated wastewater through a filtering system into the soil. The filtration system removes the large solids from the effluent and flushes the solids back to the pretreatment device. The drip tubing is placed directly into the soil without the use of media-filled trenches. The system relies on drip tubing with emitters to reduce the water pressure before the effluent enters the soil. Each emitter in the line should have the same emission rate to apply wastewater uniformly to the dispersal field. The drip tubing is placed approximately 2 ft apart in the landscape so the emitters are on a grid pattern within the existing landscape. The drip lines are buried relatively shallow so the soil can provide treatment, the landscape plants can use the nutrients and water, and the system can maximize evaporation.

Wastewater quality and quantity should be evaluated when selecting soil loading rates and mechanical equipment. Pretreatment criteria depend on the component equipment used, system design, and characteristics of the receiving environment. Primary settling or septic tank treatment is the minimum level of preconditioning necessary. Additional preconditioning to remove specific pollutants, such as

FOG, which may adversely impact the soil or receiving environment, may be necessary. Aerobic treatment is recommended to extend the life of the dispersal system.

The manufacturers of dripper-line or other low-pressure/low-profile dispersal systems should be consulted for recommendations regarding specifications for maximum particle size that can be discharged through the filtering device to adequately protect the emitters. The screens or filters must be cleaned periodically and the residuals returned to the pretreatment unit. Both manual and automatic cleaning methods are used. Providing adequate access to the filtering device in the design of the system is required for routine servicing. Pretreatment may include sand/gravel or media filtration. In addition to this pretreatment, an in-line filter on the discharge side of the pump may also be required by the manufacturer to keep emitter free from clogging solids. Follow manufacturer's instructions.

The drip dispersal system typically includes the components shown in **Figure E-8**.

Design Flow

A demand analysis of water use at the building(s) to be served should be conducted to estimate the average daily flow, expected daily peak flows and diurnal and weekly variations. See **Table B-3 Typical Per-Unit Hydraulic Loading Rates**, in **Section B.6.b Design Flow**. Flow estimates obtained from using unit values from Table B-3 represent maximum daily flows. While most dispersal systems must be designed to distribute the maximum expected peak flows, drip dispersal systems are usually designed to distribute the average daily flow with peak flows controlled by flow equalization (storage, timed dosing or alternating zones).

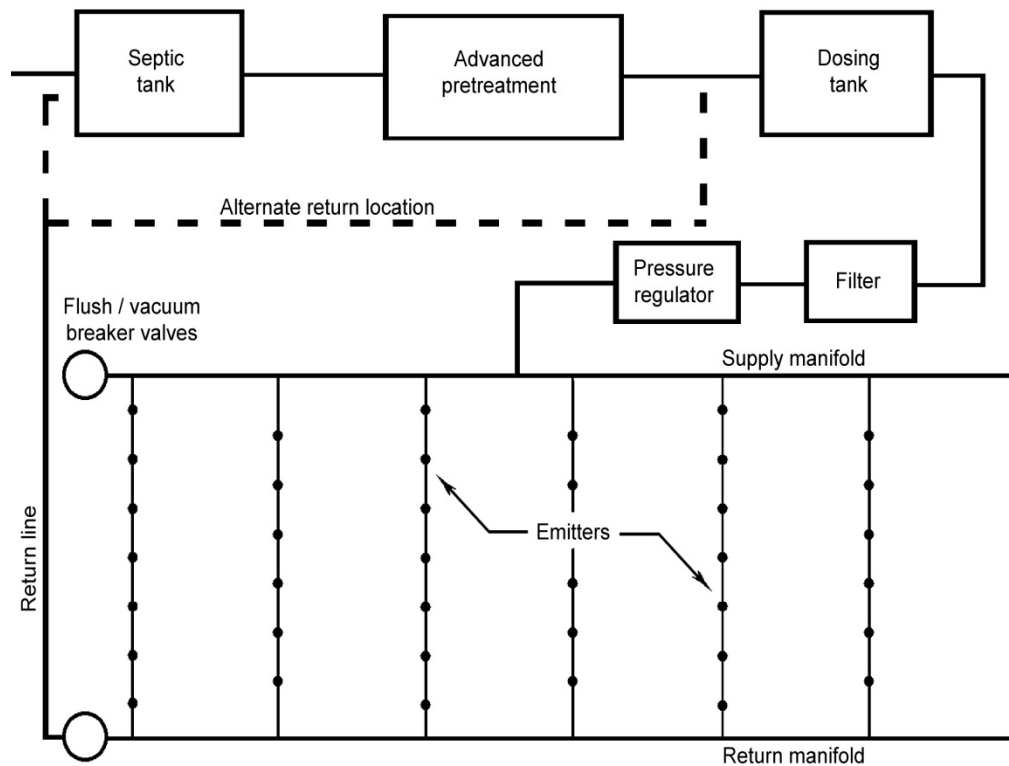


Figure E-8. Drip-Dispersal Diagram (from www.onsiteconsortium.org/graphics)

All components of the drip arrangement must work together for the successful, long-term, reliable operation of a drip dispersal system. The function of each component of the system, in regard to flow rates and pressures, should be appropriately integrated and designed to meet the design requirements given in Appendix E. All components in a drip dispersal system should be rated to withstand contact with wastewater and recommended for drip dispersal by the manufacturer or supplier. Additional components may be used as deemed appropriate by the manufacturer or designer to treat and evenly disperse the wastewater to prevent emitter clogging, prevent physical damage, monitor operation, or otherwise enhance system performance.

E.11.e. Aggregate and Alternative Aggregate

1. Aggregate

Aggregate shall mean washed gravel or crushed stone $\frac{3}{4}$ - $1\frac{1}{2}$ inches in diameter.

Larger diameter material and run-of-bank gravel are unacceptable. Fines passing the #200 sieve must be less than 5%; less than 3% is recommended.

2. Filter Fabric

The aggregate shall be covered with a material that prevents soil from entering the aggregate after backfilling, yet must permit air and moisture to pass through. The preferred material for covering the aggregate is a permeable geo-textile. Untreated building paper is acceptable only for systems under 1,000 gpd. Polyethylene and treated building paper are relatively impervious and shall not be used.

3. Alternate aggregate

Materials may be used as a substitute for conventional gravel or stone aggregate when it can be demonstrated that the material provides at least the equivalent soil infiltration area and storage volume as conventional gravel or stone aggregate. Materials shall also maintain structural integrity and be non-degradable by wastewater effluent. Fines passing the #200 sieve must be less than 5%; less than 3% is recommended.

4. Tire Derived Aggregate (TDA)

Properly manufactured tire chips have physical characteristics similar to conventional gravel or stone aggregate. Tire Derived Aggregate (TDA) may be used as a substitute for gravel or stone aggregate on a one-to-one volumetric basis, when;

- a. The TDA manufacturer shall have a written case-specific beneficial use determination (BUD) from the New York State Department of Environmental Conservation (NYSDEC) that allows a specific facility to produce and offer for sale TDA to be used in onsite wastewater treatment systems, and
- b. The TDA meets the following size and gradation requirements:
 - (i) Two-inch nominal size, and
 - (ii) Maximum dimension in any direction does not exceed four inches, and minimum dimension in any direction is not less than ½ inch, and
 - (iii) Exposed wire does not protrude more than ½” from the chip, and
 - (iv) Fine particles (3-5% passing the #200 sieve) and foreign materials are prohibited, and
 - (v) At least 95% of the TDA shall comply with the above specifications.

E.12. Fill Systems and Raised Systems

Fill System

A fill system may be used when:

1. Slowly permeable soils such as clay or clay loam overlie more permeable soils, or
2. Soils are so rapidly permeable that insufficient treatment will occur before the wastewater reaches porous or creviced bedrock or groundwater.

The site evaluation (see Section B) must show that it is possible to maintain the required depths to seasonally high groundwater, porous or creviced bedrock, and/or an impermeable layer beneath the proposed system. The necessity of installing curtain drains, underdrains, or vertical drains to prevent the flow of water into the filled area from shallow, laterally flowing groundwater or perched water tables should be investigated. Figure E-7 shows some examples of these drainage methods.

When slowly permeable soil overlies more rapidly permeable soil, the slowly permeable soil should be stripped away and replaced with a fill that is similar in texture to the underlying soil. To enhance treatment in fast soils, the fill should have a percolation rate of 10 to 30 minutes/inch, and should extend to a depth of at least two feet below the bottom of the proposed trench. The filled area should extend at least 5 feet in each direction from the sidewalls of the proposed trenches as shown in Figure E-5, to allow some lateral movement of the wastewater.

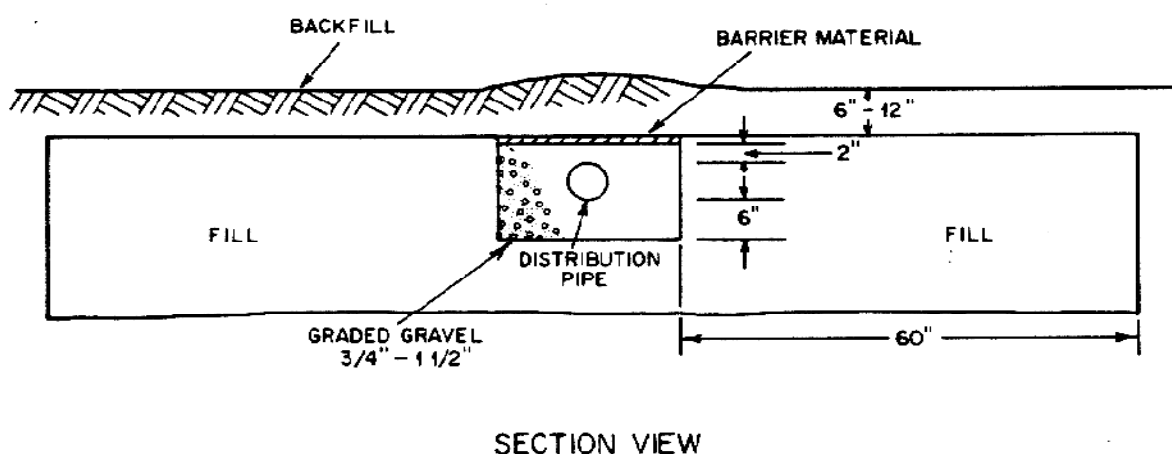


Figure E-9

Fill System

If the fill depth is greater than 4 feet, the fill must be allowed to settle before construction. Depending on the fill type and depth, as much as a full year may be necessary for natural settling. To avoid delay the fill may be spread in thin layers and mechanically compacted, but care must be taken to avoid creating layers of different density.

Coarse sands and gravels (up to a maximum size of 1-1/2 inches) should settle to at least 85 percent of the standard proctor density. Fine sands, silty sand, sandy clay, and sandy loam should settle to at least 90 percent of the standard proctor density. The percent standard proctor density can be determined by measuring the in-place dry density of the fill (using ASTM Test Method D 1556) and dividing the result by the maximum dry density of the fill (ASTM Test Method D 698). An alternative testing method for soil density (as specified by ASTM) may be acceptable if it can be shown to be more appropriate to local conditions.

Construction and sizing of the trenches in the fill shall be the same as for conventional absorption trenches. The application rate should correspond to the natural percolation rate of the fill. The texture of the backfill material used to cover the system shall not be coarser than the fill material. The original slowly permeable soil may be used.

Dosing or pressure distribution is encouraged to achieve uniform distribution throughout the absorption area and thus ensure adequate treatment of the wastewater.

Cut and Fill System

A cut and fill soil absorption system designed according to Appendix 75-A and the NYSDOH Design Handbook may only be used for systems with design flows less than 10,000 gpd where:

Slowly permeable soils (60 m.p.i. percolation rate or slower) such as clay or clay loam overlies more permeable, useable soils (1-60 minutes per inch).

A cut and fill system may only be used where the site evaluation (see Section B) shows that there is a three foot depth of useable soil below the tight soil and above seasonally high groundwater, porous or creviced bedrock, and/or an impermeable layer beneath the proposed system. These systems are generally used where the impermeable overlying soil is one to five feet deep. Conventional absorption field systems may be used when the overlying impermeable soil is no more than one foot deep and usable soil

is placed above the aggregate.

All vertical and horizontal separation distances must be maintained as described in Table 2 of Appendix 75-A, or Table B-2 of these Design Standards, whichever are stricter.

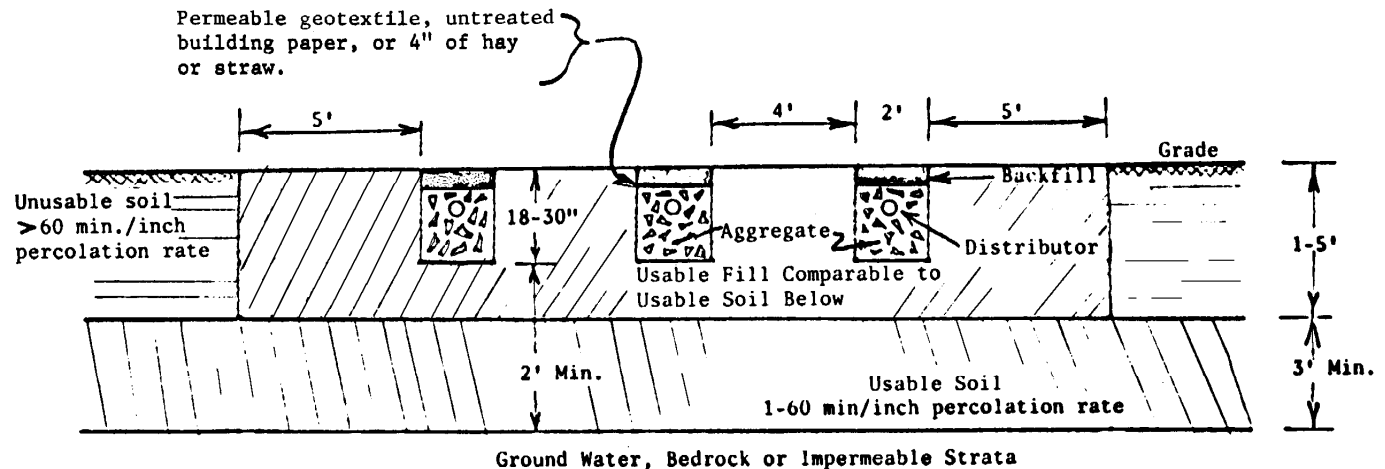


FIGURE 26

CUT AND FILL SYSTEM

Notes: On sloped sites, a diversion ditch shall be constructed uphill from the fill and trench area to prevent surface runoff from entering the absorption area.

Figure E-10: Cut and Fill System

Following NYSDOH Design Criteria in Appendix 75-A and the NYSDOH Design Handbook:

The design shall provide for the removal of the overlaying unusable soil and replacement with soil having a percolation rate comparable to the underlying soil;

An absorption trench system is designed as described in Section 75-A.8 (b);

The required length of absorption trench is based upon the percolation of the underlying soil or the fill material, whichever has the slower percolation (lower permeability).

A cut and fill system shall be constructed according to 75-A.8 (f) as follows:

- The area excavated and filled must provide at least a five foot buffer in each direction beyond the trenches.
- Careful excavation is necessary to assure that the usable underlying soil is not made unusable through compaction, and impermeable overburden does not remain in the bottom of the excavation (i.e., on top of the permeable underlying soil).
- The material placed above the trenches shall have a percolation rate faster than 60 minutes per

inch and slower than 1 minute/inch.

- A conventional absorption field system (i.e., trenches with distribution lines and aggregate) is constructed in the upper 18 to 30 inches of the permeable fill/underlying soil.
- Original surface material shall not be used as backfill above the trenches.
- The surface area of the fill system must be mounded and graded to enhance the runoff of rainwater from the system and seeded to grass.
- On sloped sites, a diversion ditch or berm shall be constructed on the uphill side of the fill area to prevent entrance of surface runoff.

Note: If the bottom of all trenches are not in or at the permeable underlying soil (i.e., the bottoms are in fill), the fill must undergo stabilization and testing prior to constructing the trenches. Stabilization may be achieved by natural settlement for at least six months including at least one freeze-thaw cycle. If the underlying permeable soil and the comparable permeable fill comprise only granular material (i.e., sand and sandy loam similar to fill material for mound systems), stabilization may be achieved by mechanical compaction in six inch lifts to the approximate density of the undisturbed underlying granular soil.

Raised System

A raised system is an absorption trench system constructed in fill material with acceptable permeability *placed above the natural soil* on a building lot. A raised system designed according to 75-A.9 (b) may only be used for systems with design flows less than 10,000 gpd.

A raised system may be used where all the following conditions are found on the site:

- There is at least one foot of original soil with a faster than 60 minutes percolation rate, above any impermeable soil layer or bedrock, but not more than two feet.
- The maximum high groundwater level must be at least one foot below the original ground surface.
- Slopes shall not exceed 15%.
- All minimum vertical and horizontal separation distances can be maintained as described in Table 2 of 75-A or Table B-2 of these Design Standards, whichever are stricter.

Following NYSDOH Design Criteria in Appendix 75-A and the NYSDOH Design Handbook:

- Percolation tests shall be conducted in the fill material at the borrow pit and after placement and settling at the construction site. The slower percolation rate of these tests shall be used for design purposes.

- The total area beneath the absorption trenches, extending 2.5 feet in all directions from the outer edge of all trenches, is defined as the basal area. The minimum size of the basal area of the raised system shall be calculated based upon an application rate of 0.2 gpd/sq.ft. A conventional absorption trench system as described in subdivision 75-A.8 (b) is to be designed using the percolation rate of the fill material. The use of slowly permeable soils for the fill material will result in a trench system that will have a basal area larger than the minimum area calculated using 0.2 gpd/sq.ft.
- The minimum size of the basal area of a raised system may be calculated based upon 0.3 gpd/sq ft if designed to receive effluent from a treatment system providing secondary or tertiary treatment, and located in an area under the jurisdiction of a Responsible Management Entity, a local sanitary code, or watershed rules or regulations that incorporate the requirement to maintain and service the treatment system in accordance with the manufacturer's recommendations. A conventional absorption trench system as described in subdivision 75- A.8 (b) is to be designed to distribute effluent evenly over the fill material basal area.
- Sufficient fill material with a percolation rate of between 5 - 30 min/in is required to maintain at least two feet separation between the proposed bottom of the trenches and any boundary condition such as groundwater, bedrock, clay or other relatively impermeable soil or formation.
- The edge of the fill material shall be tapered at a slope of no greater than one vertical to three horizontal with a minimum 20 foot taper.
- Horizontal separation distances shall be measured from the outside edge of the taper.
- The system shall incorporate siphon dosing, pump dosing or pressure distribution. Gravity distribution may be allowed where both the following conditions are met:
 - The local health department or Regional DEC office has a program incorporating site evaluation, system design approval, and construction inspection/certification, and
 - A minimum of two feet of fill material with a percolation rate of 5 - 30 min/in shall be placed between the bottom of the trenches and the existing ground.
- Curtain drains may be used to intercept and carry underground water away where high groundwater levels exist. Curtain drains shall be upslope from the system and at least 20 feet from the toe of slope of the fill material.

Raised systems shall be constructed according to 75-A.9 (b) as follows:

- Heavy construction equipment shall not be allowed within the area of the system. The underlying soil shall be undisturbed although the surface may be plowed with at least a double bottomed blade/furrow plow and the furrow turned upslope.
- A system shall not be built in unstabilized fill material. The fill material shall be allowed to settle

naturally for a period of at least six months to include one freeze-thaw cycle, or may be stabilized by mechanical compaction in shallow lifts if a fill material consisting of only a granular sand or sandy loam is used.

- The absorption trenches shall be constructed in the fill material.
- The entire surface of the system including the tapers shall be covered with a minimum of six inches of topsoil, mounded to enhance the runoff of rainwater from the system and seeded to grass.
- On sloping sites a diversion ditch or curtain drain shall be installed uphill to prevent surface water runoff from reaching the raised system area.

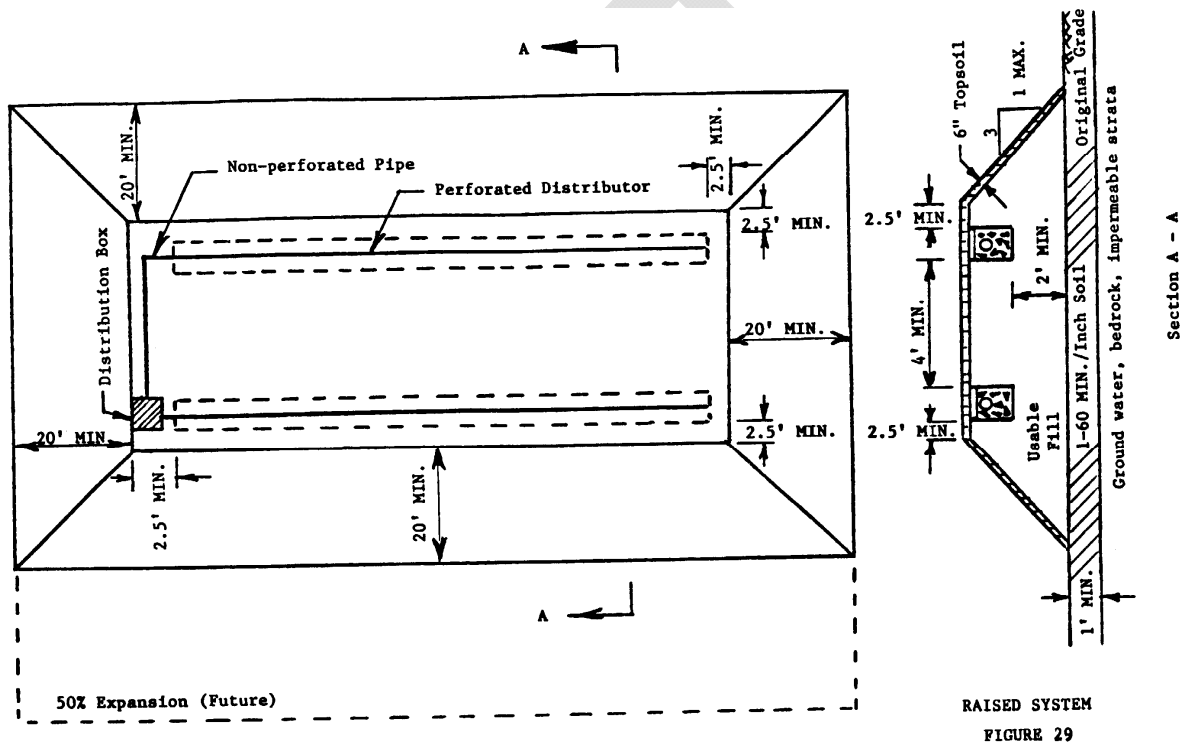


Figure E-11: Raised System

E.13. Mound Systems

A mound is a soil absorption system that is elevated above the natural soil surface in fill material. Mounds may be used when conditions preclude the use of conventional absorption system. These conditions include slowly permeable soils, shallow soils over pervious or creviced bedrock, and soils with high water

tables. The use of mounds is not encouraged, and will only be allowed when no other method of subsurface disposal is feasible.

The design of a mound is complicated and should be done only by a licensed professional engineer experienced in designing mounds. The services of a qualified hydrogeologist or other soil scientist may also be necessary. Guidelines provided by the University of Wisconsin Small Scale Waste Management Project³ should be considered when designing a mound, although the requirements and allowances herein shall be considered to supercede them.

Failure or success of a mound is highly dependent on construction quality. It is strongly recommended that the design engineer make provision for continual surveillance of construction activities.

Site Evaluation

Table E-4 lists site criteria for severe and non-severe sites. Experience with mounds is limited on severe sites. The use of mounds on severe sites is not recommended, but may be allowed on an experimental basis provided that the owner and/or developer are aware of the potential risks involved.

Systems less than 5,000 gpd on non-severe sites shall be considered small mound systems. Systems less than 5,000 gpd on severe sites, and systems greater than 5,000 gpd shall be considered large mound systems, and require a more detailed design technique.

Soils and site testing must be sufficient to characterize the hydraulic capacity of the site to treat and transmit the flow, to determine the ultimate destination of the groundwater (wells, surface water, etc.), and to demonstrate that the groundwater and its ultimate use will not be adversely impacted. For small mound systems (page E-52) at least two percolation tests for every 1000 sq. ft. of absorption area shall be performed in holes spaced uniformly within the basal area of the mound. At least one backhoe pit must be dug to accurately establish the nature of the subsurface layers and maximum groundwater level. If soil stratigraphy is variable across the site, core samples may be required to verify the soil conditions shown in the backhoe pit(s). Section B.4 also has specific percolation test depths for mounds.

For large mound systems, an extensive site evaluation is required, and shall be sufficient to establish surface

³ **Wisconsin Mound Soil Absorption System: Siting, Design And Construction Manual** by James C. Converse and E.Jerry Tyler, January , 20002, SSWMP guidance document #15.24.

water conditions, infiltration rate, vertical and horizontal saturated hydraulic conductivity, zones of permanent and perched water tables, and groundwater conditions for each of the soil horizons present. The hydraulic conductivity of the expected clogging layer should also be considered. It is recommended that this data be used to model wastewater flows using the lower confidence limits for soil hydraulic characteristics.

Mound Construction Materials

The fill should be a sand that is coarse or on the coarse side of medium according to the USDA classification system (0.35 to 1.0 mm). The aggregate should be 3/4 to 2-1/2 inch non-deteriorating rock or crushed gravel. Geo-textile drainage fabric is preferred as a barrier material for all mounds. Untreated building paper may also be used for small systems. The cap soil should be of a finer grained material such as topsoil, silt loam, or clay loam. Good quality topsoil should be used to cover the entire mound.

Table E-4. Site Criteria for Mounds

<u>Criteria</u>	<u>Non-severe Sites</u>	<u>Severe Sites</u>
Landscape Position	Well drained level or sloping areas. Crests of slopes or convex slopes are preferred. Avoid depressions, bases of slopes, and concave slopes.	Depressions, bases of slopes, concave slopes may be considered if suitable drainage is provided
Percolation Rate	0-60 min/inch	60-120 min/inch
Slope	0-6% If percolation rate slower than 30 min/inch	Up to 20%

Depth to Water Table (Minimum) 12 inches > 0 inches

Depth to Pervious or Cracked Bedrock (Minimum) 24 inches 24 inches

Depth to Impermeable Barrier (Minimum) 3 feet^{a,b} 3 feet^a

a) Slowly permeable soils, cold climates, and square mounds require greater depth.

b) Less than three feet may be allowed for very small facilities (less than 2,000 gpd) if substantial proof that the site has the hydraulic capacity to treat and transmit the flow is provided by the design engineer.

Mound Design – General

Beds or trenches shall be oriented along the site contour, or along the contour of the bedrock if it is shallow and sloped differently from the ground surface. Trenches shall be used in slowly permeable soils. Trench/bed bottoms must be level. **Figure E-12** shows the configuration of a typical mound. **Figure E-13** shows one possible subsurface flow pattern.

The fill shall be placed so as to maintain a minimum of 3 feet of unsaturated flow depth including natural soil. This distance shall be increased to 4 feet over impermeable rock.

To form the absorption area, at least 6-inches of aggregate shall be placed below the distribution pipe, and at least 2-inches shall be placed above the pipe. The size of the absorption area shall be based upon the daily wastewater flow and the recommended infiltration rate of the fill material. The width of the absorption area should not exceed 15 feet.

The barrier material should be placed above the fill and aggregate. The cap and topsoil should be at least 1.5 feet thick at the center of the absorption area, and at least 1 foot deep over the edges of the absorption area. The topsoil should be at least 6-inches deep over the entire mound. The mound slopes shall be no steeper than 3 horizontal to 1 vertical.

Multiple disposal fields are recommended for all systems, and shall be required for all large systems. Three mound areas should be built, each designed to carry 50 percent of the design flow.

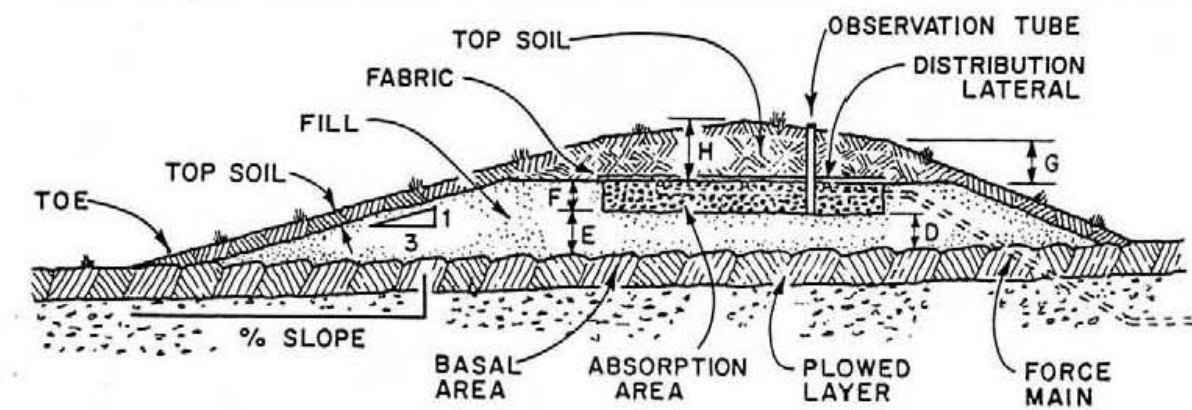
The use of groundwater monitoring wells and observation vents in the gravel bed should be considered for monitoring system performance.

Mound Design - Small Systems

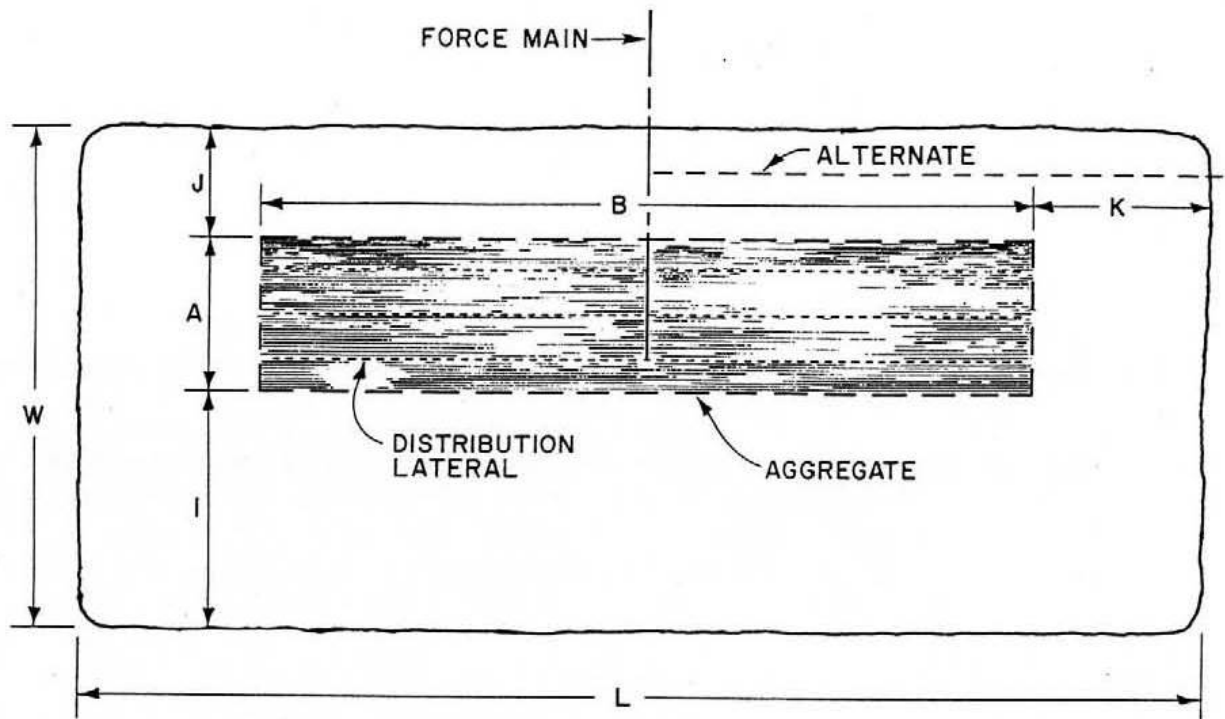
The important design feature is the basal area, which is the total area beneath the mound for level sites ($L \times W$), and the down-slope area for sloped sites ($B \times [A+I]$), see **Figures E-12** and **E-13**. The required basal area shall be determined by the infiltration rate of the natural soil, but may need to be increased to maintain appropriate side slopes. The recommended loading rates from **Table E-1** should be used for sizing the basal area. If actual flow data is available for design purposes, the application rates in **Table E-1** should be reduced by 50 percent. If an impermeable barrier is present within 5 feet of the ground surface, the application rate should be reduced to 0.1 gal/day/sq. ft. unless substantial proof is provided that the site has the hydraulic capacity to treat and transmit the flow at a higher rate.

Mound Design - Large Systems

Large system design is based on determining the maximum vertical and horizontal hydraulic acceptance of the land, and designing within this limit. Steps to be followed are:



Section View



Plan View

Figure E-12.

Mound Configuration (Side and Slope are not Drawn to Scale)

1. Evaluate the site to identify predicted wastewater flow zones in the soil.
2. Establish the horizontal and vertical boundaries of the system, and determine the boundary acceptance rates.
3. Determine the vertical wastewater application width based on the vertical and horizontal boundary acceptance rates.
4. Determine the linear loading rate based on vertical and horizontal acceptance rates. This is the maximum acceptance rate per linear foot, and must not be exceeded.
5. Determine the basal width of each horizon beneath the mound based on the, linear loading rate and the vertical acceptance rate of each horizon. The basal width of the surface horizon will determine the placement of the mound toe. The ground above the widest basal width must be unused for additional water absorption and unrestricted by driveways, ditches, foundations, etc.
6. Determine the absorption trench/bed width based on the linear loading rate and mound fill infiltration rate.
7. Determine the trench/bed length based on the design flow rate and the linear loading rate. An example of this procedure using appropriate design equations is given below.

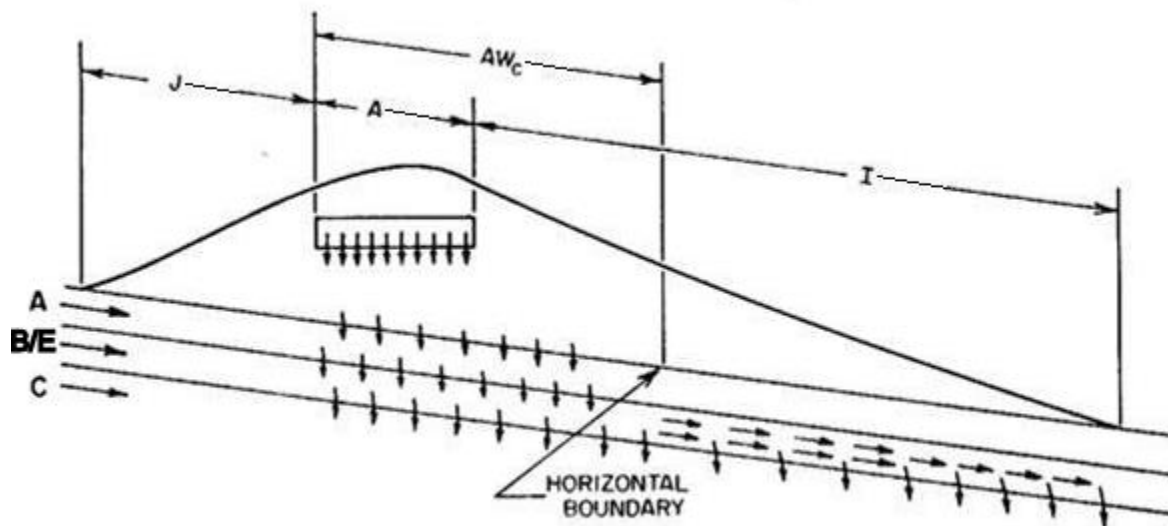


Figure E-13. Horizontal and Vertical Wastewater Travel in a Mound

Distribution Systems in Mounds

Pressure distribution networks are recommended for all mound systems, and are mandatory for large systems. The pump should be able to provide 2 feet of head at the distal end of the laterals.

Table E.5 lists allowable lateral lengths for several pipe diameters and hole-spacing. There shall be one lateral per trench, and no more than 3 laterals per bed.

Mounds should be dosed 2 to 4 times per day, and the dosing volume should be at least 10 times the lateral pipe volume.

Table E.5. Allowable Lateral Lengths (feet) based on hole-spacing and hole-diameter:

Hole Spacing (inches)	Hole Diameter		Pipe Diameter	
	(1 inch)	1 inch	1-1/4 inch	1-1/2 inch
30	3/16	34	52	70
	7/32	30	45	57
	1/4	25	38	50
36	3/16	36	60	75
	7/32	33	51	63
	1/4	27	42	54

Mound System Construction Techniques

Soil moisture at about 7-inch depth should be checked before construction. If the soil can be rolled into a ribbon between one's hands it is too wet, and construction must be postponed.

Trees should be cut to the ground leaving stumps in place. Excess vegetation should be mowed.

The mound site should be plowed perpendicular to the slope with a mold board or chisel plow to a depth of 7 to 8 inches. Roto-tillers shall not be used except on sandy soils.

The fill should be placed on the upslope and side edges of the mound site, and pushed into place using a small track-type tractor with a blade immediately after the site is plowed. A minimum of 6-inches of fill shall be kept beneath the tracks at all times to minimize soil compaction.

Fill should be placed to the desired depth and the side slopes shaped. The trench/bed can then be formed with the tractor blade. The bottom of the absorption area shall be hand-leveled before the aggregate is placed.

Every effort must be made to minimize traffic on the construction site, especially on the down-slope areas. No truck wheels shall be allowed on the plowed area.

E.14. Mound Design Example

Below is a University of Wisconsin Mound Design Example from **Wisconsin Mound Soil Absorption System: Siting, Design and Construction Manual (Wisconsin Mound Manual)**. A copy of the Wisconsin guidance document and some interpretation of the soils data are needed to assure concurrence with application rates of **Table E.1** in these Design Standards.

Effluent Distribution Network in a Mound System:

Pressure distribution network is essential for distributing the septic tank effluent. Gravity flow is unacceptable as it will not distribute the effluent uniformly over the infiltrative surface or along the length of the mound (Converse, 1974, Machmeier and Anderson, 1988). Otis (1981) provides design criteria and examples for pressure distribution. **Pressure Distribution Network Design** by Converse (2000) discusses pressure distribution and **Section E.6** provides a design example.

Design an on-site system based on the following soil profile description.

Site Criteria:

1. Soil Profile - Summary of 3 soil pit evaluations (See **Figure E-13** above).

A Horizon	0 - 6 in.	10YR6/4&2/1; silt loam (Sil); strong, moderate, angular blocky structure; friable consistence.
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E Horizon	6 -11 in.	10YR5/3; silt loam (Sil); moderate, fine platy structure; firm consistence.
B Horizon	11-20 in.	10YR6/3; silty clay loam (Sicl); moderate, fine, subangular blocky structure; firm consistence; few, medium, distinct mottles starting at 11".
C Horizon	20-36 in.	10YR5/3; silty clay (sic); massive structure; very firm consistence; many, medium, prominent mottles.

2. Slope 20%

3. The area available consists of 170 ft along the contour and 50 ft along the slope. There are 3 medium size trees in the area.

4. The establishment generates 300 gallons of wastewater of domestic septic tank effluent per day based on water meter readings.

Procedure:

Step 1. Evaluate the quantity and quality of the wastewater generated.

For all on-site systems a careful evaluation must be done on the quantity of wastewater generated. As indicated earlier, most code values have a factor of safety built into the flows generated daily. These are the values that are typically used for design. It is appropriate for the designer to assess if the code value is appropriate for the given facility and if not, work with the regulators on a suitable number. If metered values are used, a suitable factor of safety must be added to the daily average flow such as 50 to 100%. The average flow should be based on a realistic period of time and not be, for example, an average of six months of very low daily flow rates and 6 months of very high flow rates in which case then the high flow rates should be

used for design plus the factor of safety. It is best to over-design rather than under-design; even though the cost is greater the system performance and longevity should be greater.

Effluent quality must also be assessed. If it is typical domestic septic tank effluent, these sizing criteria may be used. If it is commercial septic tank effluent, lower loading rates (gpd/ft²) must be used (Siegrist, et al., 1985⁴) or the effluent pretreated to acceptable BOD and TSS. Use a factor of safety of 150%.

Design Flow Rate = 300 gpd x 1.5 = 450 gpd.

(Note: In New York State typical design flows are 110 gpd/bedroom; see *Section B.6.b Design Flow*. The State of Wisconsin's experience has shown that some mounds designed at 150 gpd/bedroom have ponded even though the actual flow was probably well below the design.)

Step 2. Evaluate the soil profile and site description for design linear loading rate and soil loading rate.

For this example and convenience the one given soil profile description is representative of the site. A minimum of 3 evaluations must be done on any actual site. More may be required depending on the variability of the soil. The soil evaluator must do as many borings as required to assure that the evaluation is representative of the site. Soil pits are better than borings but a combination are satisfactory. In evaluating this soil profile, the following comments can be made:

The silt loam (A) horizon (0 - 6") is relatively permeable because of its texture, structure and consistence. The effluent flow through this horizon should be primarily vertical.

The silt loam (E) horizon (6 - 11") has a platy structure and firm consistence. The

⁴ Siegrist, R.L., D.L. Andersen, and J.C. Converse. 1985. Commercial wastewater on-site treatment and disposal. In. Proceeding of the Fourth National Symposium on Individual and Small Community Sewage Systems. ASE, St. Joseph, MI 49085

consistence will slow the flow and the platy structure will impeded vertical flow and cause the flow to move horizontally. If this layer is tilled, the platy structure will be rearranged and the flow will be primarily vertical. Thus, tillage must be done at least 11 in. deep on this site to rearrange the platy structure. If the structure in this horizon was not platy, then tillage would be limited to 5-6" in-depth.

The silty clay loam (B) horizon (11-20 in.) is slowly permeable because of the texture and firm consistence. The flow will be a combination of vertical and horizontal flow in the upper portion and primarily horizontal flow in the lower portion of the horizon due to the nature of the next lower horizon. During wet weather the "B" horizon may be saturated with all flow moving horizontally.

The silty clay (C) horizon (20 - 36 in.) will accept some vertical flow as the effluent moves horizontally down slope in the upper horizons. The flow through this profile will be similar to the cross section shown in Fig. 2c and during seasonal saturation as shown in Fig. 2b.

Based on experience a properly designed mound system should function on this site.

It meets the minimum site recommendations found in Table 1, **Wisconsin Mound Manual**.

Linear loading rates range from about 1 - 10 gpd/lf. Since this site has a very shallow seasonal saturation and a very slowly permeable horizon at about 20", and seasonal saturation at 11", the linear loading value for this site should be 3-4 gpd/lf.

Linear Loading Rate = 4 gpd/lf

Note: LLR = 3 could be used for a more conservative design and less risk of toe leakage especially during seasonal saturation.

A basal loading rate for the soil horizon in contact with the sand (basal area) is selected based on the surface horizon (A). Use Table 2 of the *Wisconsin Mound Manual* to determine the design basal loading rate.

Basal Loading Rate = 0.8 gpd/ft²

Step 3. Select the sand fill loading rate.

The section entitled “Sand Fill Loading Rate” and Figure 6 of the **Wisconsin Mound Manual** give guidelines for selecting a suitable sand fill for the mound. Other fills may be used but caution should be used as performance data is very limited with the other fills.

$$\text{Sand Loading Rate} = 1.0 \text{ gpd/ft}^2$$

Note: No absorption area credit is given for use of chambers in mounds.

*Note: For Steps 4 through 13 see **Figures E-12 and E-13** for location of dimensions given.*

Step 4. Determine the absorption area width (A),

$$\begin{aligned} A &= \text{Linear Loading Rate} / \text{Sand Loading Rate} \\ &= 4 \text{ gpd/ft} / 1.0 \text{ gpd/ft}^2 \\ &= 4 \text{ ft} \end{aligned}$$

(Since this appears to be the weak point in the mound, consider making it 6 ft wide. A 6 ft wide absorption area would give a sand loading rate of 0.67 gpd/ft². The linear loading rate will remain at 4 gpd/lf. However, increasing the area will require more orifices in the pressure distribution network).

Step 5. Determine the absorption area length (B).

$$\begin{aligned} B &= \text{Design Flow Rate} / \text{Linear Loading Rate} \\ &= 450 \text{ gpd} / 4 \text{ gpd/lf} \\ &= 113 \text{ ft.} \end{aligned}$$

Step 6. Determine the basal width (A + I).

The basal area required to absorb the effluent into the natural soil is based on the soil at the sand/soil interface and not on the lower horizons in the profile. An assessment of the lower horizons was done in Step 2 when the linear loading rate was estimated.

$$\begin{aligned} A + I &= \text{Linear Loading Rate} / \text{Basal Loading Rate} \\ &= 4 \text{ gpd/ft} / 0.8 \text{ gpd/ft}^2 \\ &= 5.0 \text{ ft (The effluent should be absorbed into the native soil, within about 5 ft.)} \end{aligned}$$

Since $A = 4 \text{ ft}$

$$I = 5.0' - 4.0' = 1 \text{ ft. ("I" will also be calculated based on side slope)}$$

Step 7. Determine the mound fill depth (D).

Assuming the code requires 3 ft of suitable soil and soil profile indicates 11 in. of suitable soil then:

$$D = 36" - 11" = 25 \text{ in.}$$

Step 8. Determine mound fill depth (E).

For a 20% slope with the bottom of the absorption area level then:

$$\begin{aligned} E &= D + 0.20(A) \\ &= 25" + 0.20 (48") \\ &= 35 \text{ in.} \end{aligned}$$

Step 9. Determine mound depths (F), (G) and (H)

$$F = 9 \text{ in. (6 in. of aggregate, 2 in. for pipe and 1 in. for aggregate cover over pipe)}$$

$$G = 6 \text{ in.}; \text{ and } H = 12 \text{ in.}$$

These depths for G and H were reduced (in WI) from 12" and 18" to allow more oxygen to diffuse into and beneath the absorption area. Sand filters have only 6" of cover and freezing is not a problem as long as the distribution network drains after each dose. Granted most sand filters are below grade which may be a factor.

Step 10. Determine the up slope width (J)

Using the recommended mound side slope of 3:1 then:

$$\begin{aligned} J &= 3 (D + F + G) \text{ (Slope Correction Factor from Table 3 of the **Wisconsin Mound Manual**)} \\ &= 3(25" + 9" + 6") (0.625) \\ &= 6.25 \text{ ft or 6 ft} \end{aligned}$$

Step 11. Determine the end slope length (K).

Using the recommended mound end slope of 3:1 then:

$$\begin{aligned} K &= 3((D + E)/2 + F + H) \\ &= 3 ((25" + 35")/2 + 9" + 12") \\ &= 12.75 \text{ ft or 13 ft} \end{aligned}$$

Step 12. Determine the down slope width (I)

Using the recommended mound side slope of 3:1 then:

$$\begin{aligned} I &= 3(E + F + G) \text{ (Slope Correction Factor from Table 3 of the **Wisconsin Mound Manual**)} \\ &= 3(35" + 9" + 6")(2.5) \\ &= 37.5 \text{ ft.} \end{aligned}$$

Since the "I" dimension becomes quite large on steeper slopes, it may be desirable to make the down slope steeper such as 2:1 and not mow the mound. If the natural slope is 6% instead of 20% the mound width would be 28 ft (9 + 4 + 15).

Step 13. Overall length and width (L + W)

$$\begin{aligned} L &= B + 2K \\ &= 113 + 2(13) \\ &= 139 \text{ ft} \end{aligned}$$

$$\begin{aligned} W &= I + A + J \\ &= 31 + 4 + 6 \\ &= 41 \text{ ft} \end{aligned}$$

Step 14. Design a Pressure Distribution Network

A pressure distribution network, including the distribution piping, dosing chamber and pump, must be designed. A design example is presented by J. Converse in **Pressure Distribution Network Design**, 2000 (see reference in Section E.6).

Items to consider when designing the pressure distribution network:

- Using 3/16" holes instead of 1/4" holes with an effluent filter in the tank.
- Using 6 ft²/orifice instead of the typical 15 - 20 ft²/orifice used in the past.
- Provide easy access to flush the laterals such as turn-ups at end of laterals.
- Dose volume at 5 times the lateral pipe volume and not to exceed 20% of the design flow and not at the previously recommended dose of 1/4 the design flow or 10 times the lateral void volume.
- Timed dosing which requires surge capacity in the septic tank/pump chamber.

With the configuration of the mound (long and narrow), the dose volume is larger than for sand filter and time dosing may not be appropriate if larger dose volumes are required due to 5 times the lateral volume.

E.15. Seepage Pits

Seepage pits can be used for subsurface disposal of sewage where the soil below a depth of 2 or 3 feet is more porous than above this depth, where the subsoil is fairly well drained, and/or where the land area is too limited for other systems. Seepage pits must be preceded by treatment at least equivalent to a septic tank. *Figure E-14* shows a typical seepage pit.

Seepage pits should not be used where drinking water is obtained from shallow wells, or where the percolation rate is slower than 30 minutes/inch. If the percolation rate is faster than 5 min/inch seepage pits should not be allowed unless extensive pretreatment is provided. Required pretreatment may include biological treatment and disinfection.

The site evaluation must show that it is possible to maintain the required depths (see Section B) to seasonally high groundwater, porous or creviced bedrock and/or an impermeable layer beneath the proposed system.

It is recommended that pits have an effective diameter at least equal to the depth of the pit. The effective diameter shall not be less than 6 feet. Only the sidewall area of the pit structure may be used for sizing the absorption area. Application rates for sizing the necessary sidewall area are given in **Table E-1**. Soil layers with percolation rates slower than 30 minutes/inch must be excluded from the effective depth. **Table E-6** enumerates the effective absorption area of pits of various dimensions.

Where more than one seepage pit is required the pits should be arranged in groups running generally parallel to the site contour lines. The separation distance from sidewall to sidewall between seepage pits shall be at least equal to three times the diameter of the largest pit. The piping from the septic tank must be so arranged as to distribute the sewage uniformly among the pits. The use of a distribution box, with separate laterals each feeding no more than two pits, is recommended. The pits may not be dosed in series, although an equalization pipe between them is considered desirable. Equalization pipes must be laid level, and should be located in the lower half of the pit.

All seepage pits should meet the following construction requirements:

1. During excavation, smearing and compaction of the sidewall shall be avoided.
2. A layer of coarse gravel 6 to 12 inches deep shall be placed in the bottom of the pit prior to placement or construction of the chamber.
3. Precast concrete rings with large perforations are often used for pit lining. Alternatively, good quality building block can be used as the pit lining. Rectangular block may be used but curved block is preferable. The thickness of the seepage pit lining should be at least eight inches. The walls should be laid close, with no attempt to provide openings between the units. The unevenness of the edges will provide all the space necessary for the sewage to seep into the surrounding soil. Pits with no wall openings are more structurally stable than those with appreciable spaces between units. Fieldstone lining (12-inches thick with open joints) is acceptable.
4. When using eight inch curved block, the bottom of the seepage pit should not be more than 15 feet below ground surface; with 8-inch rectangular block, not more than 10 feet below ground.
5. Below the inlet pipe the space between the pit lining and the earth wall should be filled with clean crushed stone or washed gravel to a thickness of at least 12-inches; stone shall be 1- 1/2 to 2-1/2 inches in diameter. Any area between the lining and the wall that is filled with media smaller than 1 1/2 inches shall not be included as part of the effective diameter. A layer of synthetic drainage fabric or untreated building paper should be placed on top of the gravel before soil is backfilled.
6. The cover and walls above the inlet should be watertight. The cover should have the strength to support soil cover and any anticipated load, and must extend at least 12-inches horizontally beyond the excavation. Access to the pit should be provided via a locking watertight manhole extending to the ground surface.

Table E.6 Sidewall Areas of Circular Seepage Pits (square feet)

Diameter of Seepage Pit (feet)	Effective Strata Depth Below Inlet (feet)									
	1	2	3	4	5	6	7	8	9	10
3	9.4	19	28	36	47	57	66	75	85	94
4	12.6	25	38	50	63	75	88	101	113	126
5	15.7	31	47	63	79	94	110	126	141	157
6	18.8	38	57	75	94	113	132	151	170	188
7	22.0	44	66	88	110	132	154	176	198	220
8	25.1	50	75	101	126	151	176	201	226	251
9	28.3	57	85	113	141	170	198	226	254	283
10	31.4	63	94	126	157	188	220	251	283	314
11	34.6	68	104	138	173	207	242	276	311	346
12	37.7	75	113	151	188	226	264	302	339	377

For depths greater than 10 feet, find the area by adding sections.

Example: Area of 15 foot deep pit = Area of 10 foot pit + Area of 5 foot pit.

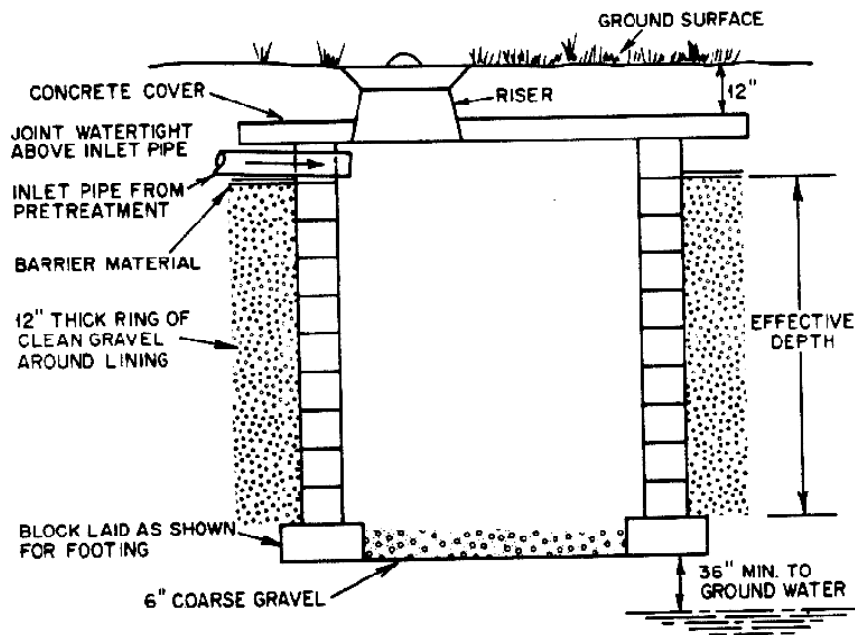
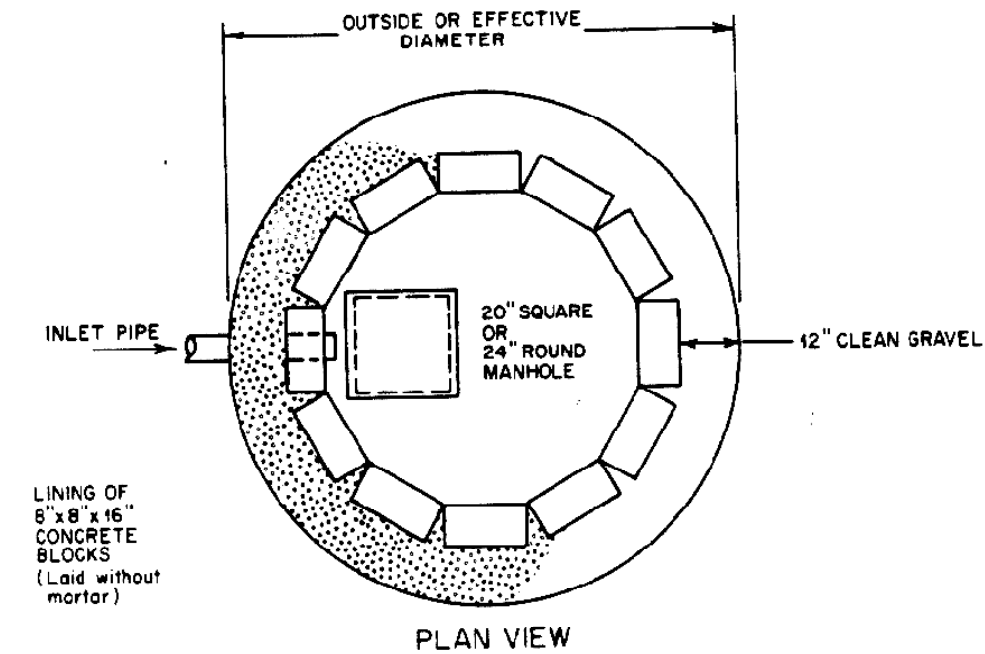


Figure E-14.

Seepage Pit

E.16. Artificially Drained Systems

Artificial drainage lowers high water tables and allows the use of subsurface disposal techniques. Successful design depends on the correct diagnosis of the drainage problem. Four general types of drainage problems are possible. They are: 1) free water tables, 2) water tables over leaky artesian aquifers, 3) perched water tables, and 4) lateral groundwater flow. The initial site evaluation must be extensive enough to distinguish between these problems.

Drainage of an artesian fed water table should not be attempted. Also, sites with slow moving free water tables are not usually practical to drain because closely spaced underdrains with pumped discharges are often necessary.

Shallow, lateral flow problems are the easiest to correct. Curtain drains or vertical drains can be used for this situation. Perched water problems can be corrected using vertical drains or curtain drains. Vertical drains shall only be used if the restrictive soil layer is thin and overlies permeable soil. Vertical drains are not generally recommended because of concern for groundwater quality of deeper aquifers. **Figure E-15** shows some examples of these drainage methods.

It is recommended that the drainage system be installed, and its effectiveness tested, prior to approval of sewerage plans. Effectiveness should be tested during spring months.

The minimum horizontal separation distances given in Table B.2 for interceptor drains must be maintained between the drain and the absorption system.

Outlets of drains must be protected from entry by small animals, and should prevent entrance of floodwaters where submergence may occur. Outlets should be designed to prevent erosion.

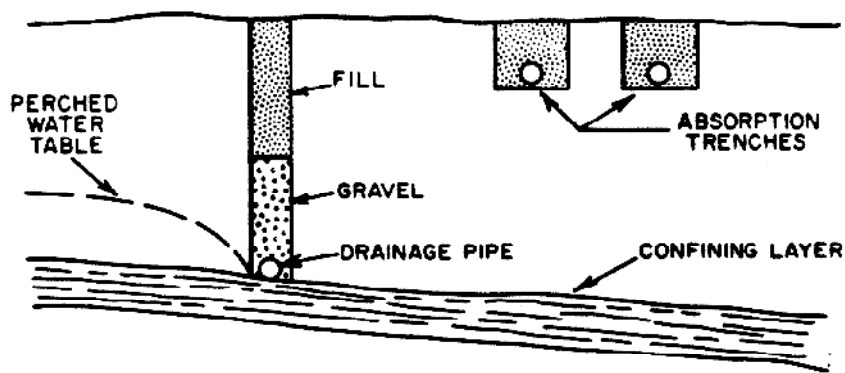
Porous media such as gravel must be placed in drains to a level above the high water table. Fill material at the ground surface should be fine textured to prevent entrance of surface water. Surface inlets via pipes must be avoided wherever possible.

It may be necessary to surround drainage pipes with an envelope filter to prevent clogging. The envelope

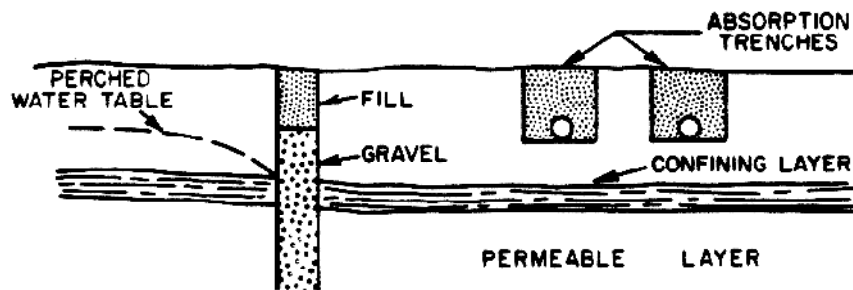
filter may be an aggregate (gravel and/or sand) filter or a geotextile (fabric filter). Aggregate size or pore size for the geotextile will be critical to the functioning of the filter. Also, geotextiles should be chemically compatible with local conditions.

Relief pipes and/or breathers may be necessary on long curtain or underdrains.

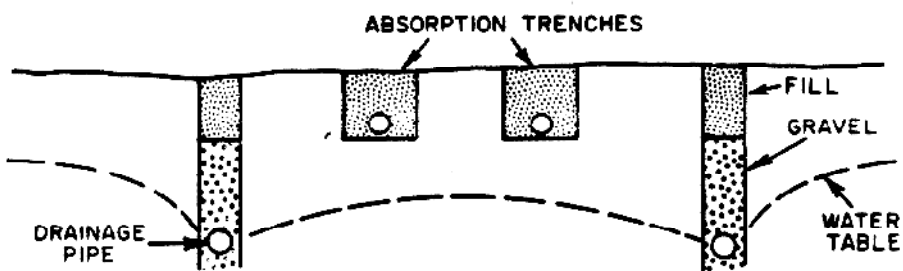
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A) CURTAIN DRAIN



B) VERTICAL DRAIN



C) UNDERDRAINS

Figure E-15. Subsurface Drainage Methods

F. Secondary Treatment

F.1. Introduction

Following the planning and permit application process outlined in Section A, and the project evaluation process in Section B, will bring the design engineer to a point of deciding on the treatment options to use to meet the secondary or intermittent stream treatment limits for surface discharges or to meet treatment requirements for subsurface discharge. This section discusses treatment methods to produce secondary treated effluent. Section G discusses advanced treatment methods that produce effluent of a quality greater than secondary and for wastewater constituents not addressed by secondary limits.

Preliminary and primary treatment by a septic tank or similar unit(s) is required prior to the secondary and tertiary treatment systems in Sections F and G. If the preliminary and primary treatment is pre-designed into a package plant, the design engineer must demonstrate that adequate volume and dimensioning for grit removal, solids separation and solids storage is provided. Flow equalization should be provided for all treatment modes with the exception of septic tanks, single pass (intermittent) sand filters, and lagoons. If flow equalization is not proposed for the system, the engineering report must specify why it is not being proposed.

The Table F-1 reprint of the 2002 USEPA OWTS Manual Table 4-1 (EPA page 4-3) presents secondary and tertiary treatment methods. The USEPA website provides access to the 1980 Design Manual and 2002 OWTS Manual (link to “Guidance, Manuals and Policies”), and to other information on onsite wastewater treatment systems from federal, and state environmental agencies, universities, and industry professionals.

The responsibility for the process to achieve the needed level of treatment rests with the design engineer. For some commercial wastewaters, grease removal may be required (Section D.5) or tertiary treatment techniques such as physical-chemical treatment (Section G) may be more appropriate than biological secondary treatment. New York State's official standards for municipal wastewater treatment and collection facilities standards, the Ten State Standards, should be used for intermediate-sized systems except as amended here. Holding tanks are not considered as treatment; their use under exceptional circumstances is described in Section D.11.

Treatment	Treatment process	Treatment methods
Suspended solids removal	Sedimentation	Septic Tank Free water surface constructed wetland Vegetated submerged bed
	Filtration	Septic tank effluent screens Packed-bed media filters (incl. dosed systems): Granular (sand, gravel, glass, bottom ash), peat, textile or foam. Soil infiltration
Soluble carbonaceous BOD and ammonium removal	Aerobic suspended –growth reactors	Extended aeration Fixed-film activated sludge Sequencing Batch Reactors (SBRs)
	Fixed-film aerobic bioreactor	Soil infiltration Packed bed media filter (incl. dosed systems): Granular (sand, gravel, glass), peat, textile or foam Tricking filter Fixed film activated sludge Rotating biological contactors (RBC)
	Lagoons	Facultative and aerobic lagoons Free water surface constructed wetlands
Nitrogen transformation	Biological Nitrification (N) Denitrification (D)	Activated sludge (N) Sequencing batch reactors (N) Fixed film bio-reactor (N) Re-circulating media filter(N,D) Fixed-film activated sludge (N) Anaerobic upflow filter (N) Anaerobic submerged media reactor(D)

		Vegetated submerged bed (D) Free-water surface constructed wetland (N,D)
	Ion exchange	Carbon exchange (ammonium removal) Anion exchange (nitrate removal)
Phosphorus removal	Physical/Chemical	Infiltration by soil and other media Chemical flocculation and setting Iron –rich packed-bed media filter
	Biological	Sequencing Batch Reactors
Pathogen removal (bacteria, viruses, parasites)	Filtration/Predation/Inactivation	Soil infiltration Packed-bed media filters: Granular (sand, gravel, glass, bottom ash), peat, textile or foam
	Disinfection	Hypochlorite Ultraviolet light
Grease removal	Flotation	Grease interceptor
		Septic tank
	Adsorption	Mechanical skimmer
	Aerobic biological treatment (incidental removal will occur; overloading is possible)	Aerobic biological systems

Table F-1 Suggested Treatment Methods to Meet Treatment Objectives

(reprinted from 2002 USEPA OWTS Manual Table 4-1, page 4-3)

F.2. Media Filtration and Alternative Media

F.2.a. General

Sand filters and alternative media filters must be preceded by properly designed settling facilities. A septic tank, with an effluent filter, is usually sufficient; aeration of the septic tank effluent will extend the life of the filter. Post filtration aeration and/or disinfection may be necessary prior to discharge to surface waters.

Intermittent sand filters can be buried or open. Open filters may be single-pass or re-circulating. Intermittent filters can be used where the soil is impermeable, or where highly polished effluent is desired. Discharge is generally to surface waters, but may be to a soil-based treatment system.

Influent flows and loads

Examples of influent levels recommended for the design of sand filters:

Five-day Biochemical oxygen demand (BOD₅) ≤ 300 mg/l;

Total suspended solids (TSS) ≤ 170 mg/l;

Fats, oils, and grease (FOG) ≤ 25 mg/l; and

Total Kjeldahl nitrogen (TKN) ≤ 75 mg/l.

(RI Sand Filter Guidance, March 2010).

Five-day Biochemical Oxygen Demand (BOD₅) ≤ 220 mg/L;

Total Suspended Solids (TSS) ≤ 150 mg/L;

Fats, Oil and Grease (FOG) ≤ 30 mg/L; and

Total Nitrogen (TN) ≤ 180 mg/L (for Recirculating Sand Filters).

(State of Wisconsin Department of Commerce, Division of Safety and Buildings, sand filter component manuals for private onsite WWTS, 1999).

Sand/Gravel Filtration

Sand filtration, specifically, is a well-established method of wastewater treatment. Single-pass

(intermittent) sand filters operating by siphon-dosing to gravity are preferred as being least costly and requiring the least maintenance. Where a gravity system can be used, single-pass sand filters (SPSFs) will provide reduced energy costs. However, recirculating sand filters (RSFs) require additional operator control, cause fewer odor problems, may result in a more consistent effluent quality than single-pass filters and will have a smaller footprint than single pass filters. Recirculating sand filters may require up to three pumps: to the filter, to recycle, and to the soil-based treatment system (STS). (See Buried and Open filter descriptions below).

Liners

The need for a liner, curtain drain, or other appropriate measure to prevent infiltration to, or exfiltration from, the filter should be evaluated, particularly where high groundwater levels, fractured bedrock, and soils with fast percolation rates are present. If the natural/native soil has a percolation rate faster than 60 minutes/inch it is strongly recommended that the filter have a liner, especially where groundwater contamination is a concern

An impermeable liner is required for sand/media filter systems with design flows over 1,000 gpd. Liners shall be 30 mil per ASTM D 751 (for thickness), ASTM D 412 (for a tensile strength of 1,100 lb., and an elongation of less than 200%), ASTM D 624 (for a tear resistance of 150 lb./in), and ASTM D 471 (for water adsorption range of +8 to -2% mass). The liner must be ultraviolet resistant with a geotextile fabric, or three inches of sand, below it to protect the liner from puncture.

Flow rates

In general, open filters are preferred over buried filters when the wastewater flow rate exceeds 30,000 gpd. Open filters generally may be used for wastewater flows up to 200,000 gpd.

Resting filters

When multiple single pass filter beds are used, a resting period of at least 60 days for every six months of operation is recommended to oxidize the clogging mat and increase the lifetime of the filter. The surface area of multiple filter beds should be adequate to handle the recommended design hydraulic and organic loading listed in each section with the largest filter bed at rest. See Single Pass Sand Filters – Buried Filters for seasonal use and resting of single-bed filters

Winter start-up of filters should be avoided. Open recirculating sand or gravel filters can be designed to avoid the formation of a surface clogging mat, and the resulting required maintenance.

Filter media alternatives

Sand, pea gravel and graded gravel are most often used in the construction of sand/gravel filters; the use of crushed stone should be avoided unless it is washed to remove all fine materials that could clog the filter. Other granular media that have been used are bottom ash, expanded clay, expanded shale, and crushed glass. These media should remove BOD and TSS similar to sand and gravel for similar effective sizes, uniformity, and grain shape. Newer commercial media such as textile materials, open-cell foam, and peat have had limited testing, but based on early testing should be expected to perform as well as the above types.

Alternative media may be manufactured in “pods”, tanks or containers sized by design flow or “per bedroom” for residential use. Any media should be evaluated based on allowable loading rates, durability, performance and cost. It is not possible to provide design data for every material. The design engineer should require the manufacturer to provide sufficient documentation regarding the performance under expected on-site conditions (such as sewage strength, ambient temperature and desired effluent quality).

Sand/gravel media sizing and specification

The following subsections will focus on the use of sand or gravel media. This type of media should be durable, insoluble in water, have rounded grains, and an organic content of less than 1 percent. Only washed material should be used. Fine particles passing the U.S. No.200 sieve (less than 0.074 mm) should be limited to less than 3 percent by weight. Sieve analysis criteria are often given in other state filter design criteria or the design criteria of proprietary media filters. 3/8-inch should be the largest particle size.

A statement from a certified laboratory and/or from the source operator indicating that a sample has been analyzed, and that the indicated sand/gravel media is the material that will be supplied, must be provided to the design engineer prior to construction, and should also be provided for every load delivered. Sufficient media must be supplied for a minimum filter depth of 24-inches. On-going monitoring of media quality during construction is strongly encouraged to assure the proper media is installed. A sample sieve size specification is shown here:

Example Sand Media Gradation

<i>US Standard Sieve</i>	<i>Particle Size (mm)</i>	<i>Limit (%)</i>
3/8	9.5	100/100
4	7.8	95/100
8	2.4	80/100
16	1.2	45/85
30	0.60	15/60
50	0.30	3/15
100	0.15	0/4
200	0.075	0/0

Ranges of media sizes are recommended for the different types of sand/gravel filters in the discussion to follow. The effective size (ES, or d_{10}) of the media refers to the sieve size in millimeters that permits 10 percent by weight to pass. The Uniformity Coefficient (UC) is the ratio between the sieve size that will pass 60 percent by weight and the effective size (d_{60}/d_{10}).

In general, without recirculation, smaller media sizes (d_{10}) combined with low loading rates will result in both a high quality effluent and enhanced nitrification. A properly operated filter (i.e., not overloaded) should be able to achieve nitrification of at least 80 percent of the applied ammonia. From the 2010 Rhode Island Sand Filter Guidance Introduction: "Sand filters when designed, installed, and operated in accordance to this guidance will provide effluent BOD₅ and TSS levels of less than 10 mg/l. Sand filters are efficient nitrifying units, and can reduce septic tank effluent ammonia-nitrogen levels from 35-55 mg/l to less than 5 mg/l by passage through a single pass sand filter," providing 86 to 91% removal." Additional performance data is given in the 2002 EPA OWTS Manual Technical Fact Sheets 10 and 11.

Other material selection

Piping for sand filters may be of a variety of materials, although perhaps the most common is PVC (ASTM D3034). Materials should be appropriate for the anticipated loads and the chemical nature of the wastewater.

Wastewater distribution

Gravity distribution of wastewater resulting in ***trickle flow*** to sand filters shall not be allowed. System dosing, via siphons, floating outlet dosing devices, or pumps must be provided for all media filters. Provision should be made to prevent the flow of wastewater out through any vents when the system is being dosed.

F.2.b. Single-Pass (Intermittent) Sand Filters

Single pass filters can be used as a secondary treatment method or for effluent polishing following package plants. Single pass sand filters are insufficient treatment for intermittent stream effluent limits. Single pass filters and can be buried or open.

Single-Pass (Intermittent) Sand Filters - Buried Filters

Buried filters are intermittent filters constructed below grade in a lined or unlined excavation and covered with soil material. The design limits operator access to the filter. These filters shall not be used directly after package aerobic treatment plants because an upset may cause clogging. Use of an effluent filter in a separate tank following the aerobic treatment unit would prevent the solids carry-over to the buried sand filter if properly installed, operated and maintained.

A buried filter should be constructed in accordance with Figure F.1. Multiple filter beds are strongly recommended when filters are of the buried type, and are mandatory if the flow exceeds 1,000 gpd. Steps shall be taken to divert rainfall and runoff away from buried filters.

Media: The recommended effective media size (E.S.) range is 0.25 to 1.0 mm, and the uniformity coefficient (UC) shall be less than 4. If nitrification is *not required*, effective media size should be 0.5 to 1.0 mm. Media with lower Uniformity Coefficients are preferred as less likely to clog, but are more difficult to obtain or more expensive.

Loading: *Hydraulic* - The hydraulic application rate for buried filters shall be no more than 1.0 gpd/sq. ft. for filters in continuous operation. Application rates up to 2.0 gpd/sq. ft. may be allowed if a bed is operated such that it will rest for an amount of time equal to or greater than that for which it is in use on a yearly basis (i.e., seasonal operation). *When nitrification is required*, the application rate shall not exceed 1.0 gpd/sq. ft. Efficient nitrification cannot be expected with filters that operate on a seasonal basis, recirculation is needed.

Loading: *Organic* – The long term organic loading should be less than 0.005 lbs BOD/day/sq ft.

If nitrification is required, a lower organic loading is recommended e.g. 0.003 lbs BOD/day/ sq.ft.

Base: Approximately 2-inches of gravel shall be placed above and below the distribution lines and the under-drains. Gravel around the under-drains and distribution lines shall be 3/4 to 1-1/2 inches. Pea gravel should be 1/8 to 3/8 of an inch. The ground beneath the filter shall be sloped to the trenches in which the under-drains are laid.

Dosing: This type of single-pass buried filter should be flooded at least twice per day, and the volume of each dose should be at least seventy-five percent of the volume of the distribution lines. Distribution boxes should be used to direct sewage flow. Dosing siphons are acceptable for dosing buried filters, although pressurized dosing systems that provide two feet of head at the distal end of the distribution system are strongly recommended

Arrangement: Under-drains shall not be placed on greater than 12-foot centers, and at least two under-drains must be provided. Under-drains must be sloped to the outlet. Distribution lines shall not be placed on greater than 3-foot centers. For installations with more than 800 feet of distributors, the filter shall be constructed in two or more section such that no section contains more than 800 feet of distributors. Dual siphons or pumps must be provided to alternate the flow to different sections. Venting of the under-drains is also required.

Construction: The filter must be settled by flooding before the distributor is placed at final grade. Before backfilling, a barrier material should be placed above the graded gravel. The barrier material should be a synthetic drainage fabric (permeable geo-textile). Untreated building paper is permitted only for SPSFs with design flows less than 1,000 gpd. Backfilling should be done carefully, and the use of heavy machinery should be avoided.

Approximately 6-inches of topsoil should be mounded over the site with a 3 to 5 percent slope to direct rainwater away from the filter.

Single-Pass (Intermittent) Sand Filters - Open Filters - for filter flooding designs:

Single pass filters can be used as a secondary treatment method or for effluent polishing following. A typical open single pass filter is shown in Figure F.2.

Media: The recommended effective media size (E.S.) range is 0.25 to 1.0 mm, and the uniformity coefficient (UC) shall be less than 4.

For filters that nitrify, effective sizes of 0.14 -0.65, and UCs of 1.5 to 4.0 are acceptable.; see EPA OWTS Manual Technology Fact Sheet 10 – Intermittent Sand/Media Filters for performance.

If nitrification is not required, effective media size should be 0.5 to 1.0 mm. Media with lower Uniformity Coefficients are preferred as less likely to clog, but are more difficult to obtain or more expensive.

Loading: Hydraulic - If nitrification is not required, the hydraulic loading rate shall not exceed 5-gpd/sq. ft. from septic tanks and other primary settling tanks. From trickling filters and package activated sludge plants, application rates up to 10-gpd/sq. ft. are acceptable. The use of an effluent filter in a separate tank following the aerobic treatment unit is required to prevent any solids carry-over from the ATU. Decreased loading rates are recommended when nitrification is required.

For filters that nitrify, loading rates of 0.33 – 3.0 gpd/sq ft, with corresponding dosing rates of 3-24 times per day, and a dosing depth of < 0.5cm are acceptable to achieve full nitrification (denitrification requires recirculation) Lower hydraulic loading rates and greater dosing rates (12-24 times per day) are recommended for better nitrification than flooding (see EPA OWTS Manual Technology Fact Sheet (TFS) 10 – Intermittent Sand/Media Filters).

Loading: Organic – The long term organic loading should be no more than 0.005 lbs BOD/day/sq ft. Media on the larger side of the range, and dosed more frequently (12-24 per day), may allow higher

organic loading, but research is lacking.

If nitrification is required, an organic loading of 0.003 lbs BOD/day/ sq.ft. is recommended; an even lower organic loading may be beneficial, but research is lacking.

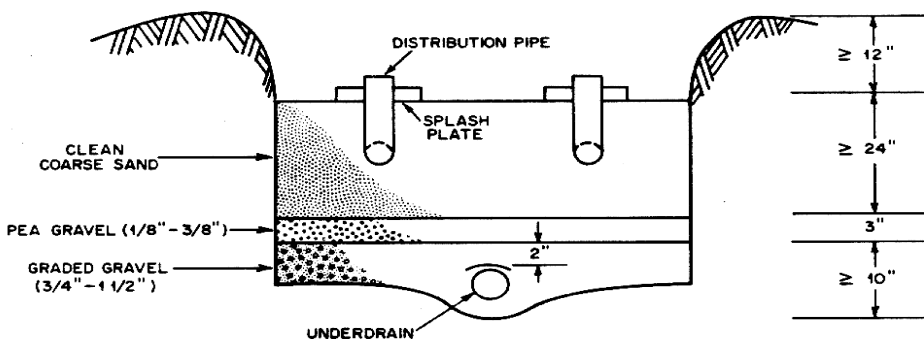


Figure F.2 – Open Single Pass Sand Filter

Dosing: Filters of this design (pre-1980's) should be flooded at least twice each day with a volume equal to a depth of 2- inches over the bed, or to a depth of 4-inches if the effective media size is greater than 0.5 mm. Adequate resting periods must be provided between doses.

Arrangement: Multiple filter units must be provided to allow for maintenance, except for very small facilities (flow less than 2,000 gpd). Rectangular beds should be considered to provide more complete coverage during dosing.

Base: Graded gravel must be placed to a depth of at least 10-inches around the under-drains, and shall be 3/4 to 1-1/2 inches in size. This shall be covered with at least 3 inches of pea gravel, which is 1/8 to 3/8 inches in size.

Under-drains: Under-drains shall not be placed on greater than 12-foot centers, and at least two under-drains must be provided. Under-drains must be sloped to the outlet. Distribution lines shall not be placed on greater than 3-foot centers. For installations with more than 800 feet of distributors, the filter shall be constructed in two or more section such that no section contains more than 800 feet of distributors. Dual siphons or pumps must be provided to alternate the flow to different sections. Venting of the under-drains

is also required.

Rectangular beds should be considered to provide more complete coverage during dosing.

Dosing: If nitrification is not required, and effective media size is 0.5 to 1.0 mm, the filter should be flooded at least twice per day. Filters of this design should be flooded with a volume equal to a depth of 2- inches over the bed, or to a depth of 4-inches if the effective media size is greater than 0.5 mm. Adequate resting periods must be provided between doses.

Where nitrification is required, lower hydraulic loading rates and greater dosing frequencies (12-24 times per day) are recommended; see EPA OWTS Manual Technology Fact Sheet (TFS) 10 – Intermittent Sand/Media Filters. Doses of less than 0.5 cm (< 0.02 inches), at regular one hour intervals provide a better environment for treatment, and results in a 3 gpd/sq.ft. application rate.

Distribution: For flooding, troughs, pipelines, central inlets, and spray nozzles may be used for distribution of the settled sewage over the filter surface, and shall be located so that the maximum lateral travel is not more than 20 feet. Ridge and furrow methods are recommended for winter operation.

Siphons, floating outlet dosing devices, or pumps, at minimum head, shall have a discharge capacity of at least 100 percent in excess of the maximum rate of inflow to the dosing tank; and at the average head, at least 90 gallons per minute per 1,000 square feet.

For flooding filters, the discharge lines to the beds shall have sufficient capacity to permit the full rate discharge of the siphon, float-based dosing device or pump.

For lower hydraulic loading rates and greater dosing frequencies, either pressure distribution with drilled orifices and orifice shields, or drip dispersal is recommended. Designs using smaller and more frequent doses will use both smaller pumps and discharge lines and require less energy to operate.

For both distribution methods, the discharge pipe must be drained between doses to prevent frost damage. This can be done by providing a drain-line from the distribution line back to the recirculation tank, or by providing small weep holes in the discharge pipe. At least 18-inches of sand must be present below the weep holes. For a pumped system, the discharge pipe can be sloped back to the dosing chamber and the check-valve at the pump eliminated. In this case the dosing volume must be sized to account for this backflow.

Under-drains: Under-drains must be sloped to the outlet, and shall not be placed on greater than 12 foot centers. At least two under-drains must be provided with venting.

Walls: Provision must be made to prevent soil or stormwater from washing onto beds. Walls that are exposed directly to the air in cold climates should be insulated. The walls also prevent creeping weed growth from entering the filter area.

Maintenance: Winter start-up in any location should be avoided. Open sand filters must be raked and weeded as needed. The distribution network should be flushed annually, and the dosing pump should be calibrated at least annually. Multiple filter units must be provided to allow for maintenance, except for very small facilities (flow less than 2000 gpd). Regular filter bed rotation is also recommended for resting (draining and reaerating).

F.2.c. Recirculating Sand/Media Filters (RSF)

Recirculating filters can be used as a secondary treatment method or for effluent polishing following package plants. Depending on the design, an effluent filter may be required to prevent occasional solids carry-over from some aerobic treatment units that may precede the sand or media filter. A typical recirculating filter is shown in Figure F-3.a.

Recirculating Sand/Media Filters - Partially Buried Filters

Most RSFs are constructed aboveground and with an open filter surface; however, in cold climates, they can be placed in an in-ground excavation to prevent freezing. Placing a cover over an RSF is recommended to reduce odors and to provide insulation in cold climates. Walls preventing eroded soil or stormwater from entering the filter must be constructed, and insulated in cold climates. Covers must provide ample fresh air venting because re-aeration of the filter media occurs primarily from the filter surface.

Use of a wier box for recirculating is required for visual flow confirmation and determination of the quantity of recirculating flow.

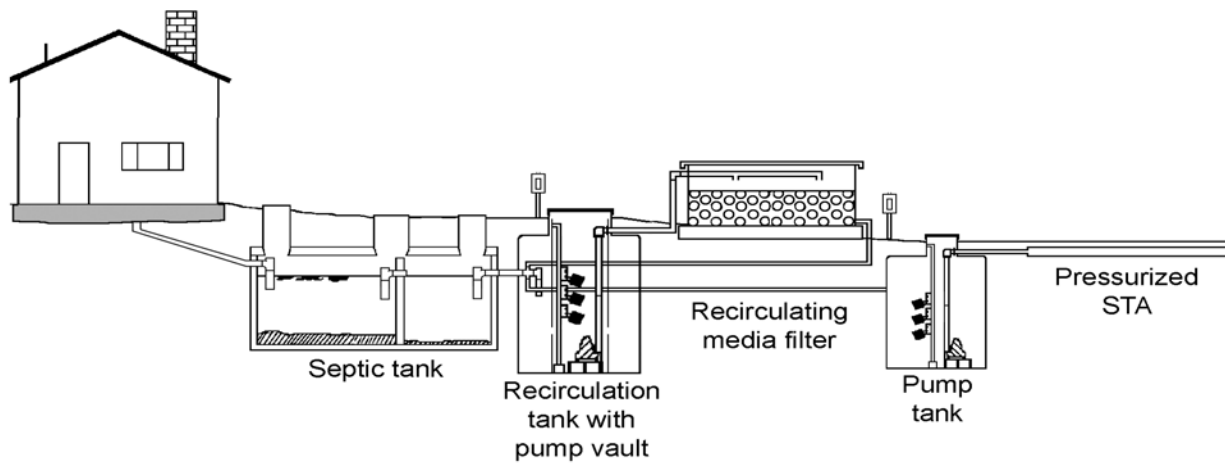


Figure F.3.a Recirculating Sand/Media Filter with Separate Recirculation Tank

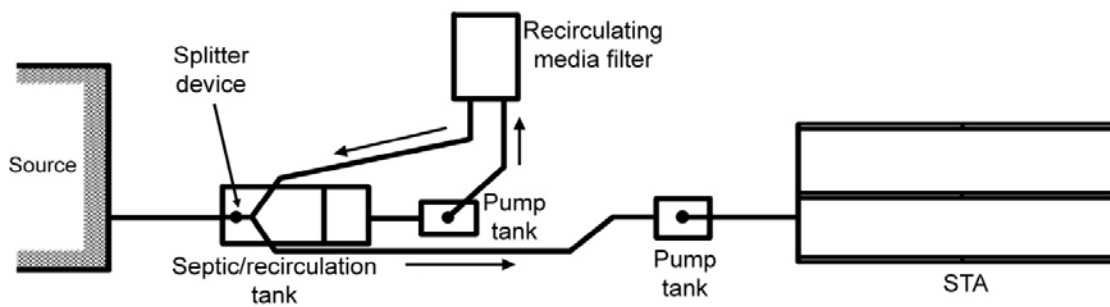


Figure F.3.b Recirculating Sand/Media Filter with Splitter Device (CIDWT Graphics)

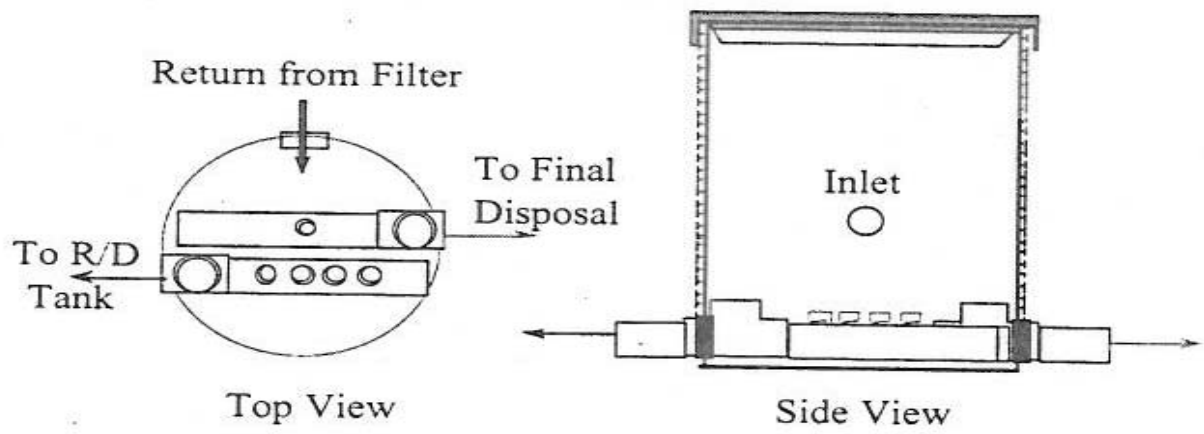


Figure F.3.c

Splitter Device

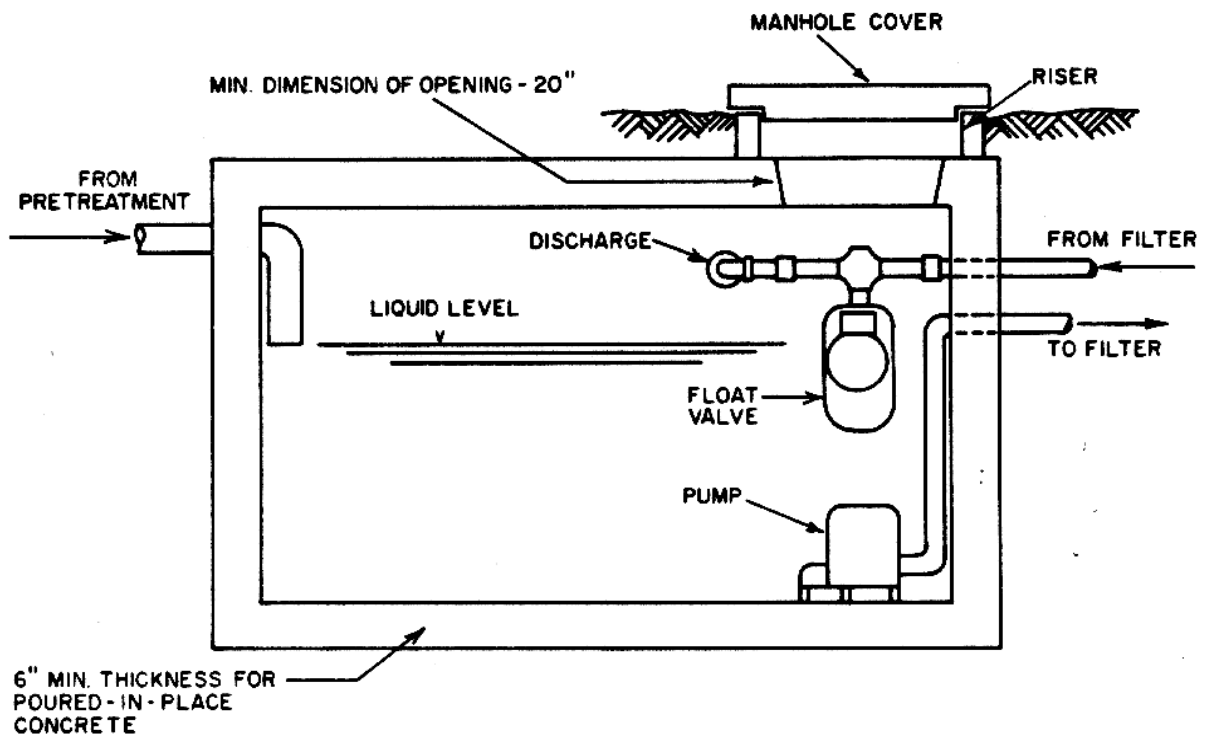


Figure F.3.d.

Recirculation Tank

Recirculating Sand/Media Filters - Open Filters - *for flood dosing designs:*

Recirculation tank: Recirculation tank volume (Figure F.3.d) should be at least equal to one-and-one-half (1.5) times the daily wastewater flow, or as determined by the Recirculation Tank Volume Calculation Worksheet below. A recirculation ratio of 3:1 to 5:1 should be maintained, and a loading rate of 5gpd/sq. ft. based on forward and not recirculated flow, shall not be exceeded.

Effluent from the filter should be returned to the recirculation tank, or discharged depending on liquid level in the recirculation tank. Acceptable recirculation ratios may be maintained by using a float valve arrangement, moveable gates, check valves, or a flow splitter box as shown in **Figures F.3.b & c**, above.

Dosing: For small systems dosing should last 5 to 10 minutes every 30 minutes. For larger systems dosing should last up to 20 minutes every 2 to 3 hours. Filters should be flooded with a volume equal to a depth of 2-inches over the bed.

Media: The effective size of the media shall be 0.3 to 1.5 mm with a uniformity coefficient less than 4. The use of coarse media with a low uniformity coefficient is strongly recommended.

Arrangement: Multiple filter units must be provided to allow for maintenance and adequate resting periods (60 days per 6 months of operation) except for very small facilities (flow less than 2,000 gpd). The arrangement of the under-drain and distribution piping is similar to single pass open filters.

Recirculation Tank Volume Calculation Worksheet

Recirculation tank sizing

In many types of commercial systems, daily flow variations can be extreme. In such systems, the recycle ratios necessary to achieve the desired treatment may not be maintained unless the recirculation tank is sized properly. During prolonged periods of high influent flows, the recirculation ratio can be reduced to the point that treatment performance is not maintained unless the recirculation tank is sized to provide a sufficient reservoir of recycled filtrate to mix with the influent during the high-flow periods.

To size the tank appropriately for the application, assess the water balance for the recirculation tank using the following procedure:

- 1 Select the dosing frequency based on the wastewater strength and selected media characteristics.

- 2 Calculate the dose volume based on the average daily flow:

$$V_{\text{dose}} = [(\text{recycle ratio} + 1) \times Q_{\text{ave. daily}}] \div (\text{doses/day})$$

$$Q_{\text{dose}} = V_{\text{dose}} \div (\text{dose period})$$

Where V and Q are the flow volume and flow rate, respectively.

- 3 Adjust the dose volume if the calculated volume is less than the required minimum dose volume for the distribution network.

- 4 Estimate the volumes and duration of influent peak flows that are expected to occur from the establishment.

- 5 Calculate the necessary recirculation tank “working” volume by performing a water balance around the recirculation tank for the peak flow period with the greatest average flow rate during that peak period.

$$\text{Inputs} = Q_{\text{inf.}} \times T + Q_{\text{recycle}} \times T = Q_{\text{inf.}} \times T + (Q_{\text{dose}} - Q_{\text{eff}}) \times T = V_{\text{inf.}} + V_{\text{recycle}}$$

$$\text{Outputs} = V_{\text{dose}} \times (T \div \text{dose cycle time})$$

Where T is the peak flow period duration.

If the inputs are greater than the outputs, then $Q_{\text{eff}} = Q_{\text{dose}}$ and the peaks are stored in the available freeboard space of the recirculation tank. If the inputs are less than the outputs, then $Q_{\text{eff.}} = Q_{\text{inf.}}$

To provide the necessary recycle ratio, sufficient filtrate must be available to mix with the influent septic tank effluent. The filtrate is provided by the return filtrate flow and the filtrate already in the recirculation tank.

$$\text{Recycle ratio} \times Q_{\text{inf.}} \times T < Q_{\text{recycle}} \times T + \text{minimum tank working volume}$$

$$\text{Where minimum tank working volume} = \text{Recycle ratio} \times (Q_{\text{inf.}} - Q_{\text{recycle}}) \times T$$

- 6 Calculate the necessary freeboard volume for storage of peak flows when the influent volume is greater than the dosing volume during the peak flow period.

$$\begin{aligned}\text{Freeboard volume} &= Q_{\text{inf.}} \times T + Q_{\text{recycle}} \times T - Q_{\text{dose}} \times T \\ &= Q_{\text{inf.}} \times T(Q_{\text{dose}} - Q_{\text{eff.}}) - Q_{\text{dose}} \times T\end{aligned}$$

- 7 Calculate the minimum total recirculation volume.

Total tank volume = minimum tank working volume + freeboard volume

(The above was adapted from Ayres Associates, 1998.)

Recirculating Sand/Media Filters - Open Filters - *For pressure distribution designs:*

Recirculation: Adjustable recirculation ratio covering 2:1 to 4:1 should be available to the system operator. Recirculation may be maintained by using weir-boxes, flow splitter boxes, moveable gates, check valves, or the float valve arrangement as shown in **Figure F.3** above.

The recirculation tank volume should be at least equal to one-and-one-half (1.5) times the daily wastewater flow, or as determined by the Recirculation Tank Volume Calculation Worksheet above. The hydraulic storage capacity above the working level of the tank (freeboard volume) is provided to accommodate power outages or servicing of the system.

Dosing: Filters should be pressure dosed every 30 minutes to one hour using programmable timers. Small, regular (intermittent) doses provides uniform distribution: a better environment for treatment.

Pumps shall be sized to provide a minimum of five (5) feet of head (water pressure) at the distal end of each distribution lateral in the filter. The pump dosing the sand filter should be located in the recirculation tank that follows the septic tank, and shall be placed in a screened pump vault.

Flow from the septic tank to the recirculation tank is by gravity. All effluent should be prescreened using an effluent filter/screen before it is dosed onto the sand filter. This screen/filter assembly helps protect the pump and sand filter surface from excessive solids.

Media: The effective media size shall range from 1.5 to 3 mm, with a uniformity coefficient less between 1.3 and 2.5.. Washing is required to remove fine material (fines) from the media. The use of

larger media size is encouraged because it extends the longevity of the filters and reduces operator maintenance time. Smaller media may provide better treatment (pathogen removal), however increasing the recirculation can help address that issue if adequate resting time provides the necessary aerobic conditions. UV disinfection may also be added to reduce pathogens.

Loading: Hydraulic - The hydraulic loading rate is based on *forward flow* only (not recirculated flow), and shall not exceed 5-gpd/sq. ft. when distributing effluent from septic tanks and other primary settling tanks. With a recirculation rate of 2-4, the actual wastewater loading rate on the sand filter is 3-5 times the forward flow from the building.

Wastewater from trickling filters and package activated sludge plants equipped with effluent filters to prevent solids carry-over, at loading rates up to 10-gpd/sq. ft. are acceptable. Decreased loading rates are recommended when nitrification is required.

Loading: Organic - The long term organic loading should be less than 0.005 lbs BOD/day/sq ft. If nitrification is required, a lower organic loading of 0.003 lbs BOD/day/sq ft is recommended.

Base: Graded gravel must be placed to a depth of at least 10-inches around the under- drains, and shall be 3/4 to 1-1/2 inches in size. This shall be covered with at least 3 inches of pea gravel, which is 1/8 to 3/8 of an inch in size.

Arrangement: Multiple filter units must be provided to allow for maintenance, except for very small facilities (flow less than 2,000 gpd). Rectangular beds should be considered to provide more complete coverage during dosing.

Distribution: Using large media size eliminates the option of flooding the filter. PVC pipelines with drilled orifices or spray nozzles may be used for distribution of the settled sewage over the filter surface. 1 to 1.5 inch Class 200 PVC manifolds should deliver wastewater to a distribution pipe grid of 0.75 to 1 inch Schedule 40 PVC pipe.

Temperate climate systems typically drill upward facing orifices with every fifth orifice drilled at both the 6- and 12-o'clock positions. Systems in colder climates typically have all orifices pointing down with slotted orifice shields providing free drainage. An alternative to orifice shields is to use 3 to 6" PVC or corrugated plastic foundation drainage pipe as an outer sleeve to allow free drainage and prevent filter media blockage of the orifices.

Orifice spacing is determined by the dosing requirements. RSFs should receive 24 to 48 equal doses of wastewater per day. The grid dimensions (created by the pipe centering and orifice spacing) should be 2 to 2.5 feet.

The discharge pipe to the distribution system shall be a 1 ¼ to 2 inch PVC (Class 200 minimum) pipe, the actual size depending upon such factors as distance, pump head, friction loss, and desired pressure at distal orifices. The discharge pipe must be drained between doses to prevent frost damage. This can be done by providing a drain-line from the distribution line back to the recirculation tank, or by providing small weep holes in the discharge pipe. At least 18-inches of sand must be present below the weep holes. For a pumped system the discharge pipe can be sloped back to the dosing chamber and the check-valve at the pump eliminated. In this case the dosing volume must be sized to account for this backflow.

The distal ends of laterals in a RSF, which are readily accessible by pushing aside a small amount of pea stone, do not need sweep elbows (turn-ups). These lateral ends should have threaded ball valves onto which a distal head measurement pipe can be attached. This ball valve will also be the location through which lateral cleaning will occur.

Under-drains: Four (4) inch diameter Schedule 40 PVC slotted under-drain collection pipes must be sloped to the outlet, and shall not be placed on greater than 10 foot centers. The under-drain may lay level or on a maximum slope of 0.5%. The slots should be oriented upwards, sized 0.25 by 2.5 inches, and spaced 4 inches apart.

At least two under-drains must be provided with venting. Venting should be provided by bringing the distal end of the under-drain pipe to the surface of the filter and supplied with a removable cap. In addition to venting, the pipe to the surface can be used as a clean-out and an observation port.

The under-drain pipe shall leave the concrete or lined filter enclosure via a watertight, sealed penetration.

A minimum of 4 inches of 0.5 to 0.75-inch clean washed stone shall be placed between and over the under drain pipes. If a plastic liner is used, sharp, angular stone should be avoided, to prevent liner punctures.

Eight inches of 3/8-inch clean washed pea-stone shall be placed carefully over the under-drains and drainage stone to assure that the filtering media is not washed down into the under-drain.

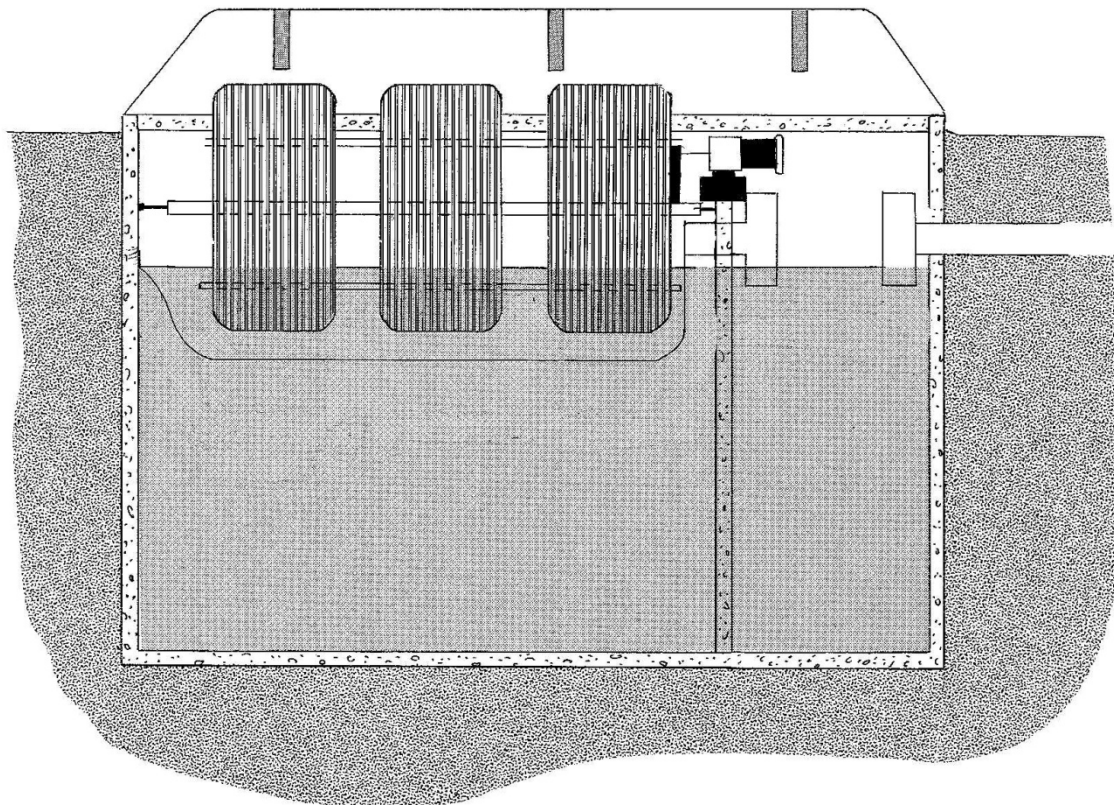
Walls: Provision must be made to prevent soil from washing onto beds. Walls that are exposed directly to the air in cold climates should be insulated. The walls also prevent creeping weed growth from entering the filter area.

Cover: For a RSF, cover the filter with 3/8-inch pea-stone to a level of 2 to 4 inches over the top of the lateral (distal) end ball valve. (No topsoil cover shall be placed over the pea-stone.)

Maintenance: Winter start-up should be avoided. Regular rotation with resting is also recommended.

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F.3. Rotating Biological Contactor (RBC)



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Figure F.4. Rotating Biological Contactor (RBC) Schematic

The Ten State Standards do not address RBCs. TR-16 (2011) Section 6.2.4 addresses pretreatment, media, unit sizing, designing for operational flexibility, and weather protection. The TR-16 Section 6.2.4 design criteria shall serve as New York State's standards for RBCs, as amended here:

1. Pretreatment - RBCs should be preceded by properly designed settling facilities. Fine screening is also acceptable as pretreatment, provided compliance with removal and effluent requirements can be demonstrated. Efficient grease and scum removal devices should be provided, especially when influent grease loads are high. Flow equalization must be provided when the ratio of peak (maximum instantaneous) to average daily flow exceeds 2.5.
2. Staging - At least 3 stages shall be provided for secondary treatment applications. Additional stages may be necessary for nitrification and further BOD removal. For small installations, up to 4 stages can be provided on a single shaft by installing inter-stage baffles in the tank, with the

direction of flow parallel to the shaft.

3. Media - On a normal 27 ft. shaft with media diameter of 12 ft., standard density media is considered to be 100, 000 sq. ft. of surface area, and high density is 150, 000 sq. ft. of surface area. High-density media should not be used in the first two stages of the system. At least 35% of the media should be submerged.
4. Organic Loading - The organic loading of the RBC system shall be accurately defined, including loading from recycle flows. The design of an RBC unit for secondary treatment shall not exceed a load of 3.0 lbs of soluble BOD/day/1000 sq. ft. and 5 lbs of total BOD/day/1000 sq. ft. on any stage based on average flow.
5. Nitrification - Four or more stages are usually necessary when nitrification is desired in addition to BOD removal, because maximum nitrification rates will not be obtained until the level of soluble BOD drops to 10 mg/l or less. For design purposes the average maximum removal rate should not exceed 0.3 lb NH₃-N/day/1000 sq. ft. of media.

Where large and/or frequent peaks in flow or organic loading are anticipated, consideration should be given to providing either additional media or flow equalization to ensure consistently low ammonia nitrogen levels.

The temperature of the system should be maintained at or above 55°F. If this is not possible, additional media should be provided to compensate for reduced removal rates. The system pH must be held between 7.1 and 8.6. If the wastewater is poorly buffered the system should have the capacity for the addition of alkaline chemicals.

6. Pilot Plant Studies - When possible, full-scale diameter media should be used for pilot plant studies. If small diameter units are used, each stage should be loaded at or below the oxygen transfer capability of a full-size diameter unit to minimize scale-up difficulties.
7. Monitoring - Load cells for measuring total shaft weight must be provided for the first stage of the standard density and the first stage of the high density shafts, at a minimum. Electronic strain gauge load cells are preferable, but hydraulic load cells may also be used.

Dissolved oxygen levels should be monitored in at least the first stage of the RBC system.

Supplemental air should be considered in the original design. Supplemental air helps increase DO levels, control filamentous bacteria growth, and can be used to strip excess growth from an RBC, and may increase treatment efficiency

8. Enclosures – Either permanent buildings or covers shall be used to protect the RBC units from sunlight and winter weather. In some instances covers have been used inside a building to help minimize the corrosive and humid atmosphere.

Enclosures must provide ready access to all components of the RBC units for observation and minor repair. Covers should be made in removable sections, or have some other means of allowing removal and replacement of the shaft /media assembly.

Buildings shall have adequate ventilation, heating and humidity control, and an internal hoisting device is recommended for the removal of the shaft/media assembly. Mechanical and electrical equipment shall be properly rated for the building environment or electrical components shall be located in a completely separate area.

9. Equipment - Drive systems should be variable speed and may be mechanical or air drive, although mechanical systems are preferred. Air drive systems should have positive airflow metering and control to each RBC unit. Aeration of a mechanical drive system may increase treatment efficiency.

Bearing units shall be self-aligning and should be located outside of media covers to allow easy access for lubrication and maintenance.

Provision for auxiliary power during power outages is recommended as overloads may occur when the discs do not rotate.

10. Design/Configuration - Operation and maintenance requirements (including bio-film control, drive train and radial support arm maintenance and repair, and media/shaft repair and replacement) must be considered in the design and layout of RBC treatment systems. Provision should be made for positive flow control to each stage, allowing flexibility in feeding and discharge. Tank depth/configuration should be such that solids are not deposited in the tank. Also, provision should be made for draining the tank.

Large installations with closely space RBC units may need a crane for shaft/media removal. System layout must account for crane reach and size. Wastewater flow perpendicular to the shaft should be encouraged to develop uniform loading over the entire length of the shaft.

11. Flexibility - Overloads generally can be avoided if flexibility is designed into the RBC system to strip excessive biofilm growth from the media or to even the organic load to all stages. Flexibility can be achieved by having variable rotational speed, the ability to reverse rotational direction, supplemental aeration, or the potential for chemical addition. The ability to increase surface area in the affected stage should also be considered.

The process must include the ability to step feed, bypass and isolate individual RBC stages. If the first stage is being overloaded, these provisions will allow a portion of the flow to be diverted to alternative lower density/growth stages.

12. Settling - Final settling shall provide a detention time not less than 90 minutes, with maximum surface settling rate of 600 gpd/sq. ft. and weir overflow rate not greater than 5,000 gpd/lineal ft. Higher surface settling and weir overflow rates may be used if the contactor is to be followed by tertiary treatment. The surface settling rates of the final clarifier(s) following the RBC process shall be based on peak hourly flow rates.

F.4. Plastic Media Biotower: an improved trickling filter

The Ten State Standards and TR-16 have design standards for Trickling Filters. Both require pilot testing, or empirical equations based on full-scale operation to determine unit sizing for either. TR-16 has a table of Trickling Filter characteristics for four classes of TFs; the Ten State Standards do not.

Plastic Media Biofilters should be designed based on the TR-16 Table 6-1 for ranges of hydraulic and organic loading and other operational information based on the intended function of the filter and media type used. Documentation of design criteria differing from the TR-16 Table 6-1, and based on alternative manufactured media and specified recirculation rates must be provided by the design engineer.

BOD Roughing and Secondary Treatment

For pre-treatment of high-strength wastes or BOD reduction prior to further treatment for nitrification, the surface area of the manufactured media per unit volume shall not exceed $30 \text{ ft}^2/\text{ft}^3$. This limit is to provide large non-clogging passages, maximum re-distribution points, and good ventilation.

Nitrification

Bio-towers intended for ammonia oxidation following BOD roughing can use higher surface area media of up to $45 \text{ ft}^2/\text{ft}^3$ having smaller passages. Thinner bio-films in the nitrification process are less likely to cause plugging of the narrow passages.

F.5. Activated Sludge

The Ten State Standards provide design criteria based on standard modes of activated sludge and TR-16 provides design guidance based on the use of selectors or reactors. The design engineer may follow either of the above guidance documents based on the selected design, and as amended here:

Pretreatment

For all activated sludge modes, both the Ten State Standards and TR-16 provide pretreatment guidance, with or without the use of primary settling tanks. Without, screening devices with clear openings of $\frac{1}{4}$ inch or less should be provided. When primary settling tanks are provided, by-passing is recommended by the Ten State Standards for plant start-up and initial stages of the facility's design life. If the treatment train uses only one activated sludge tank, then primary treatment/settling tanks are recommended.

Extended Aeration

The Ten State Standards provides some specific design criteria for the extended aeration mode of activated sludge treatment. Those are summarized and amended for intermediate-sized systems as follows:

1. Start-up and Use Restrictions - The extended aeration process can be utilized where a highly nitrified effluent is required. Its use should be governed by the realization that it is a delicate

biological process subject to distress caused by surge loadings, variations in organic content, and periods of low or no flow. It takes approximately three months from start up to stabilization of effluent quality within design parameters. *Therefore, it is not recommended that extended aeration facilities be used for schools or seasonal facilities.*

2. Flexibility of Operation - Duplicate units are not mandatory, but the piping should be arranged to permit at least primary sedimentation in the event that any of the treatment units must be taken out of operation. Additional flexibility should be built into the system to allow switching to the contact stabilization mode of operation, particularly if the wastewater flow rate or quality will have significant seasonal variations.
3. Aeration Tank Dimensions - Water depths in vertically mixed aeration tanks may not be less than 10 feet; and not less than 5.5 feet for horizontally mixed aeration tanks. Tank width-to-depth ratio shall be between 1.0 and 1.7 to 1. Minimum ratio of tank length-to-width shall be 2.0:1. A minimum of 18 inches of freeboard shall be provided. Mechanical surface aerators require a minimum 3 feet of freeboard.
4. Aeration Tank Volume - Total aeration tank volume shall equal the design capacity of the plant or 1,000 cubic feet per 15 pounds of 5-day BOD applied on a daily basis, whichever results in the greater volume.
5. Diffuser - The air diffuser system and supply piping shall be capable of supplying 2,050 cubic feet of air per pound of 5-day BOD.
6. Blowers - Air blowers may be of the positive displacement or of the centrifugal type. Air requirements may be met by two interconnected blowers, each capable of supplying normal air requirements. Three blowers, each capable of supplying one-half of the normal air requirement are recommended. Consideration should be given to providing dedicated blower(s) for air lift pumps, to allow separation of aeration and sludge return operations.
7. RAS - Return activated sludge facilities shall be capable of returning from 50 to 150 percent of the plant design flow. Return shall be to the head of the plant. Provision shall be made for rate regulation and measurement.
8. WAS - Provision shall be made to control the mixed liquor suspended solids in the aeration tank

by wasting return activated sludge.

9. Final Settling - Final settling shall be designed to provide a surface settling rate not greater than 1,000 gpd/sq. ft. and weir overflow rates not greater than 10,000 gpd/linear ft.
10. Sludge Holding Tank - A sludge holding tank, preferably with supernatant decant capability, shall be provided. A minimum, of 1,000 gallons capacity per 15,000 gallons design flow is recommended. There must be access to the sludge draw off piping. Sludge generation and wasting is very site-specific and could be necessary every 3 to 12 months, and more frequently with phosphorus removal.

A sludge holding tank should be aerated to reduce odors, stabilize the sludge, and reduce its volume. Dispersal of waste sludge must be in accord with currently accepted practice as described in Section J.7, Residuals Dispersal (Part 360), of this document.

Contact Stabilization

The Ten State Standards provides some specific design criteria for the contact stabilization mode of activated sludge treatment. Those are summarized and amended for intermediate-sized systems as follows:

1. Start-up and Use Restrictions - Contact Stabilization shall be considered only when the design flow is at least 50,000 gallons per day. Process efficiency for dilute wastewater, and wastewater with high levels of soluble BOD is questionable.
2. Tank Volume - Total contact tank plus sludge-re-aeration tank volume shall provide not less than 1,000 cubic feet per 50 pounds of 5-day BOD applied to the aeration tanks on a daily basis, or a total volume equivalent to six hours detention time based on rated design capacity, whichever results in the greater value.

The number of tanks and flow pattern shall be such that $\frac{2}{3}$ of the total volume will be available for sludge-re-aeration and $\frac{1}{3}$ for contact with sewage at the design rate. Tank duplication requirements will be satisfied by three tanks arranged so that any one may be de-watered for service while one of the remaining tanks is used for sludge-re-aeration and one is used for sewage contact. Additional flexibility should be built into the system to allow switching to the extended

aeration mode of operation, particularly if the wastewater flow rate or quality will have significant seasonal variations.

Aeration tank dimensions and design standards for diffusers, blowers, return activated sludge, wasting activated sludge, final settling, and sludge holding tanks are the same as those listed above for extended aeration.

F.6. Sequencing Batch Reactor (SBR)

The Ten State Standards provides some specific design criteria and redundancy requirements for sequencing batch reactors in Section 92.6. TR-16 Section 6.3.7 provides design guidance for SBR systems that includes preliminary and primary treatment requirements, the SBR component processes, and downstream unit considerations. Both documents should be used for intermediate-sized systems except as amended here:

SBRs use batch-type suspended-growth (activated sludge) processes to carry out secondary treatment in a single tank, in step order: fill, treat, settle, decant and draw. SBRs may be approved at the discretion of DEC on a case-by-case basis. The design must be based on experience at other facilities and shall meet the applicable requirements under Ten State Standards Chapters 50, 70 and 90 in particular Sections 53.413, 72.43 and 92.6, and TR-16 Section 6.3.7.

Section 65 of the Ten State Standards describes flow equalization. The exception to Section 65.2 for SBRs is that flow equalization will follow the SBR units. The need for flow equalization will be determined by downstream treatment units, if any.

Continuity and reliability of treatment equal to that of the continuous flow-through modes of the activated sludge process shall be provided. Supplemental treatment units may be required to meet applicable effluent limitations and reliability guidelines. DEC should be contacted for design guidance and criteria in addition to Ten State Standards and the following:.

1. More than two tanks should be provided per Section 92.6.b. If a two tank system is proposed, the engineering justification should be documented and approved by the reviewing engineer.
2. In addition to the requirements of Section 92.6.g., the blower design shall also provide for varying the volume of air delivered in proportion to the load demand of the plant. Aeration

equipment shall be easily adjustable in increments and shall maintain solids suspension within these limits. DO probes, connected to the blowers, should be provided to allow for more efficient blower usage (i.e., prevent over or under aeration).

3. The plant design shall include the ability to chlorinate for filament control.
4. Sludge Holding Tank - A sludge holding tank, preferably with supernatant decant capability, shall be provided. A minimum, of 1,000 gallons capacity per 15,000 gallons design flow is recommended. There must be access to the sludge draw off piping. Sludge wasting from the sludge holding tank is very site-specific and could be necessary every 3 to 12 months, more frequently with phosphorus removal.

Sludge holding tanks should be aerated to reduce odors, stabilize the sludge, and reduce its volume. Dispersal of waste sludge must be in accord with currently accepted practice as described in Section J.7, Residuals Dispersal (Part 360), of this document.

SBRs are aerobic growth reactors that can also provide advanced treatment removals of phosphorus, nitrification and denitrification of nitrogenous compounds. Additional information on nitrogen removal can be found in Section G “Advanced Treatment” of this document and in TR-16 Section 6.3.7.

The EPA 2002 Onsite Wastewater Treatment Systems Technology Fact Sheet (TFS) 3 describes both Intermittent Flow (IF) SBRs (that uses a true “batch treatment” scheme), and Continuous Flow (CF) SBRs. The “Performance” section of TFS 3, (Chapter 4 pages TFS-13 thru TFS-16) gives ranges of effluent pollutant levels expected for both *intermittent-flow* and *continuous-flow* SBR systems. The USEPA website provides access to the 2002 OWTS Manual and the 1980 OWTS Design manual (link to “Guidance, Manuals and Policies”), and other information on onsite wastewater treatment systems to state and local governments, and industry professionals.

F.7. Oxidation Ditches

The Ten State Standards provides no specific design criteria for oxidation ditches. The only applicable guidance is limited to generally applying the extended aeration mode of activated sludge treatment (Section 92). The TR-16 standards Section 6.3.10 and Table 6-12 provides design guidance on the use of oxidation ditches for Biological Nitrogen Removal (BNR). Both Ten State Standards and TR-16 should be used for intermediate-sized systems as amended here:

1. Raw sewage shall be comminuted or fine-screened prior to flowing into the oxidation ditch. Primary settling is not required.
2. Design of the ditch or ditches shall be based upon 24-hour retention of the design flow or 1,000 cubic feet per 15 pounds BOD₅ applied to the oxidation ditch, whichever results in the greater volume.
3. Duplicate units are not mandatory.
4. Aerators are usually of the partially submerged rotating-brush type. When provided with this type of aeration the submergence shall be adjustable, and at least two complete units shall be provided (either located in the ditch or stocked as a spare unit). Alternative aeration schemes may be acceptable, but only upon demonstration of proper aeration and mixing capabilities by the design engineer.
5. Aerators must maintain a minimum of 2.0 mg/l of dissolved oxygen in the mixed liquor at all times throughout the aeration tank; maintain all biological solids in suspension by maintaining a velocity of at least 1.0 fps; meet maximum oxygen demand and provide for varying amounts of oxygen transferred in proportion to the load demanded; provide motors, gear housing, bearings, grease fittings that are easily accessible; be provided with equipment replacement parts that shall suffice as duplication of the unit.
6. Final settling shall be designed to provide a detention time of not less than four hours, a weir overflow rate not greater than 10,000 gpd/lineal ft., and a surface settling rate not greater than 1,000 gpd/sq. ft.
7. Pumps or airlift may be used for return sludge. Pumps should have at least 2-1/2 inch suction and discharge openings. Return piping should be at least 3 inches in diameter. Airlifts should be at least 3 inches in diameter. Return sludge rates should be between 50- 200 percent of plant design flow.
8. Provisions should be made for rate regulation and measurement.
9. Waste sludge storage for at least six months volume should be provided.

11. A sludge holding tank, preferably with supernatant decant capability, shall be provided. A minimum, of 1,000 gallons capacity per 15,000 gallons design flow is recommended. There must be access to the sludge draw off piping. Sludge generation and wasting is very site-specific and could be necessary every 3 to 12 months, and more frequently with phosphorus removal.

Sludge holding tanks should be aerated to reduce odors, stabilize the sludge, and reduce its volume. Dispersal of waste sludge must be in accord with currently accepted practice as described in **Section J.7**, Residuals Dispersal (Part 360), of this document.

F.8. Lagoons (Wastewater Treatment Ponds or Stabilization Ponds)

The Ten State Standards provides specific design criteria for three types of wastewater treatment ponds (Section 93), but does not address ponds utilized for equalization, percolation, evaporation, or sludge storage. TR-16 standards Section 6.5 provides similar design criteria for the same three types: aerated-facultative, flow-through, and controlled-discharge stabilization pond systems, and their components. Either may be used for intermediate-sized systems as amended here:

Lagoons may also be approved by DEC as an alternative effluent dispersal system when a soil-based treatment system (STS) or a surface discharge is not feasible. In these instances, the lagoon is not relied upon for any further treatment (the treatment train preceding it would provide all required treatment), but only used as a final discharge point rather than discharging directly to surface waters. DEC may choose to require more than one intermittent discharge lagoon/point depending upon the hydraulic capacity of the lagoon and the effluent flow rate. Lagoons should be designed and constructed in accordance with standard engineering practice, and shall have an engineered overflow structure to provide for controlled release during wet weather events. Guidance regarding controlled releases is in Section I.3.d.

1. A comminutor or bar screen shall be provided upstream from the influent line conveying raw sewage or waste into an aerated pond system. Primary treatment is not required.
2. Minimum separation from habitation or other occupied area for an aerated pond should be 1,000 ft. Also see Table B.2, for separation distance in developing areas.
3. Multiple cells designed to permit both series and parallel operation are recommended for all aerated ponds. For unaerated ponds, series operation is preferable to parallel operation.

4. For very small installations, a dike top width of less than 8 feet may be considered.
5. Influent lines or interconnecting piping to secondary cells of multiple-celled ponds operated in series may consist of pipes through the separating dikes.
6. The use of multiple inlets/outlets, baffles, and dikes is encouraged to prevent short circuiting. Influent lines to rectangular ponds should terminate at approximately the third point farthest from the outlet structure, if only a single inlet or outlet is provided.
7. For unaerated ponds, interconnecting piping for multiple unit installations operated in series should be valved or provided with other arrangements to regulate flow between structures and permit flexible depth control. The interconnecting pipe to the secondary cell should discharge horizontally near the lagoon bottom to minimize need for erosion control measures and should be located as near the dividing dike as construction permits.
8. Control manholes or other such flow-splitting facilities should be provided between cells of aerated ponds to provide a positive visual means of directing and controlling the flow.
9. Overflow structures should consist of a manhole or box so designed that flow from the pond during ice-free periods could be taken from below, but near, the water surface so as to select for release the best quality effluent available and insure the retention of floating solids.
For unaerated ponds the draw off lines or an adjustable overflow device should permit pond operation at depths of 2 to 5 feet, with the lowest draw off 12-inches above the bottom to control eroding velocities and avoid pick-up of bottom deposits.
10. A locking device should be provided to prevent unauthorized access to and use of the level control facilities. Wherever possible, the outlet structure should be located on the windward side to prevent short circulating. Consideration must be given in the design of all structures to protect against freezing and ice damage.
11. Stream hydrograph-controlled-release lagoons (HCRs) discharge effluent according to the current assimilative capacity of the stream. Release is usually based on stream flow, but water quality and temperature also may be considered. A review of site specific stream information and predicted effluent quality will be necessary to determine at what stream flow rates a discharge will be allowed.

12. In a multiple cell facility with a diffused air aeration system and submerged air headers, consideration must be given to arranging the overflow structure and piping to allow for independent drainage of each cell down to or below the level of the air header.

Alternatives to diffused aeration can result in energy savings. The New York State Energy Research and Development Authority (NYSERDA) developed guidance in 2010 entitled “Water & Wastewater Energy Management Best Practices Handbook,” pages 57-60.

13. Surface runoff shall be diverted around ponds and protect the pond embankments from erosion.
14. Separation distances should be maintained between the water table and bedrock.
15. The Basis of Design for Controlled Discharge, Flow Through, and Aerated Pond Systems shall be as described in the Ten State Standards- Section 93.3, as highlighted in 16 through 21 below.
16. A minimum of two (2) cells is required. Two feet of freeboard is required under average operating conditions.
17. Dikes shall be constructed of relatively impervious soil and compacted to at least 95 % standard proctor density. Vegetation and other unsuitable materials shall be removed from the area where the dike is to be constructed.
18. Facultative ponds should be between 2- 6 ft. in depth. Aerated ponds may be 10- 15 ft deep.
19. Lagoons shall be sealed and seeded as described in the subsections 93.417 and 93.422.
20. The pond shall be enclosed with adequate fencing. A locked vehicle gate should be provided to accommodate mowing equipment. Warning signs shall be provided.
21. Flow measurement and pond level gauges shall be provided.

The 2002 EPA Onsite Wastewater Treatment Systems Manual, Technology Fact Sheet (TFS) 7 describes Stabilization Ponds, Free Water Surface (FWS) Constructed Wetlands, and Other Aquatic Systems. Typical design guidance is given for facultative and aerated lagoons.

G. Tertiary Treatment

G.1. Introduction

Tables B.4-A and B.4-B of these Design Standards list the required effluent limits for secondary treatment and intermittent streams, respectively. Stricter limits may apply for some water-quality restricted waters e.g. TMDL waters. Table F-1 listed secondary and tertiary treatment methods, or technologies, used to accomplish treatment objectives. Some secondary treatment technologies can be used to achieve advanced or tertiary treatment levels and some are required as pretreatment prior to tertiary treatment. Table G-1 shows per capita mass loading of nutrients.

Table G-1. Per Capita Mass Loading of Nitrogen and Phosphorus

Typical Data on the Unit Loading Factors and Expected Wastewater Contaminant Loads from Individual Residences (From WI state regulations Section A-83.43 (7) COMMERCIAL FACILITIES, Comm 83 Appendix)

Contaminant	Unit Loading Factor lb/capita per day	Value		
		Unit	Range	Typical
BOD5	0.180	mg/L	216-540	392
SS	0.200	mg/L	240-600	436
NH3 as N	0.007	mg/L	7-20	14
Org. N as N	0.020	mg/L	24-60	43
TKN as N	0.027	mg/L	31-80	57
Org P as P	0.003	mg/L	4-10	7
Inorg. P as P	0.006	mg/L	6-17	12
Grease		mg/L	45-100	70
Total Coliform		cfu/100mL	107-1010	108

Table F-1 may be referred to in determining the tertiary treatment methods needed to accomplish the necessary treatment goals. Also see Chapter 7 of TR-16, “Advanced Wastewater Treatment.”

Tertiary treatment technologies may be considered for attaining more stringent water quality limits, or maintaining current water quality effluent standards on a smaller facility footprint (i.e. plant treatment capacity increase without physical footprint increase). Tertiary treatment may include suspended solids (colloid) and nutrient removal. Nitrogen removal can be accomplished with variations of secondary treatment (Sections F and G.4) - primarily activated sludge processes, recirculating media filters, and other fixed film unit processes. Phosphorus removal can be accomplished both biologically and physically/chemically.

TABLE G-2
Conceptual Processes for Nutrient Removal – WEF Manual of Practice No. 8 – Vol. II, Table 15-1.

Conceptual process selection for nutrient removal.						Effluent quality ^a				
Process	Secondary ^b	5mg/L BOD	5mg/L TSS ^c	Nitrification	10mg/L nitrate nitrogen	3 mg/L total nitrogen ^e	1.0 mg/L total phosphorus ^e	0.5 mg/L total phosphorus ^e		
Activated sludge	X	M	X	M						
Extended aeration (oxidation ditch)	X	M	X	X	M					
A/O TM	X	M	X	M	X		M			
Modified Ludzack-Ettinger (MLE)	X	M	X	X						
Operationally modified activated sludge	X	M	X	M	M		M			
PhoStrip TM	X	M	X	M	M		X			
University of Cape Town (UCT) and VIP	X	M	X	X	X		M			
A ² O TM	X	M	X	X	X					
Trickling filters	X			M						
Fluidized bed ^d	M			M						
Postaeration anoxic tank ^d					X	X				
Two-sludge process ^d	X	M	X	X	X	X				
Three-sludge process with chemical addition ^d	X	M	X	X	X	X	X			
Denitrification filters ^d										
Bardenpho TM	X	M	X	X	X	X				
Modified Bardenpho TM	X	M	X	X			M			
Simple TM	X	M	X	X	X	M				
Bionutre TM	X	M	X	X	X	M				
OWASA nitrification	X	M	X	X	M	M				
Sequencing batch reactors	X	M	X	X	M	M				
Phase isolation ditches	X	M	X	M	X	M				
Chemical addition (alum, lime, or iron salts)	X	M	X	M	M	M	X			

^a X-process capable of producing effluent meeting indicated standard. M-process should be capable of meeting indicated standard with proper design, acceptable influent characteristics, and/or tertiary filtration.

^b 20-30 mg/L effluent BOD₅ and total suspended solids (TSS).

^c Filtration recommended to meet indicated standard.

^d Requires methanol addition for denitrification

The Ten States Standards tertiary treatment technology selection is limited to “Phosphorus Removal by Chemical Treatment” and “High-Rate Effluent Filtration” (Chapter 110). TR-16 includes advanced treatment in Chapter 6 (biological) and Chapter 7 (physical-chemical). These standards together shall govern, except for requirements and allowances listed herein, which shall be considered to supersede them. Any conflicts between standards will be reviewed on a case-by-case basis at the discretion of the regional or local regulatory agency. Proprietary technologies will be approved on a case by case basis according to the procedure in Section H.

Additional references include the Water Environment Federation’s 1992 Manual of Practice #8, Chapters 15 and 16, and the 2002 EPA Onsite Wastewater Treatment Systems Manual.

The 2002 EPA, OWTS Manual, Technology Fact Sheet 8 “Enhanced Nutrient Removal of Phosphorus,” and Technology Fact Sheet 9 “Enhanced Nutrient Removal of Nitrogen” give descriptions of available technologies, and design, operation, maintenance, management and cost considerations.

For phosphorus removal the NYSDEC Division of Water Technical Operation and Guidance Series TOGS 1.3.6 [Phosphorus Removal Requirements for Wastewater Discharges to Lakes & Lake Watersheds](#), should be referenced through the DEC website. Treatment technologies are recommended there for three ranges of flow from facilities discharging 1) less than 10,000 gpd, 2) between 10,000 gpd and less than 50,000 gpd, and 3) over 50,000gpd.

G.2. Granular Media Filters: High-Rate/Rapid-Rate Effluent Filtration or Intermittent Filtration

Note: Because operation and maintenance requirements may be significant, high-rate filters shall not be allowed for intermediate-sized treatment facilities except where it can be demonstrated that the required supervision will be provided.

TR-16 Section 7.2.10 on Granular Media Filters distinguishes between intermittent and rapid-rate filters. Design requirements for intermittently-loaded granular media filters, e.g. Intermittent Sand Filters, are discussed under secondary treatment methods (Section F.2 in these Design Standards) but can also be used for tertiary treatment of secondary effluent, including nutrient removal. Intermittently-loaded granular media filters may use recirculation, but are not “backwashed” on a routine operational basis as high-rate filters are.

Both TR-16 Section 7.2. and 7.3 and the Ten States Standards Section 112 distinguish between high-rate “gravity filters” and high-rate “pressure filters” and provide design requirements for both. The design guidance for pressure filters (microfiltration and ultrafiltration) is much more extensive in TR-16 Section 7.3.

The granular media filtration processes addressed here include deep-bed stratified granular media units using single-, dual-, or multi-media filters with down-flow regimes. Other rapid filter units may be used if the design engineer has sufficient documented experience or performance data to provide a sound basis for the design.

Granular media filters (intermittent, re-circulating, or rapid-rate) may be used for secondary treatment or as a tertiary/advanced treatment device for the removal of residual suspended solids from secondary effluents. Filters may be necessary where effluent concentrations of less than 20 mg/L of suspended solids and/or 1.0 mg/L of phosphorus must be achieved, or to obtain adequate turbidity reduction for urban water reuse.

A ***pretreatment process*** such as chemical coagulation and sedimentation or other acceptable process should precede the filter units where: (1) effluent suspended solids requirements are less than 10 mg/l, (2) secondary effluent quality can be expected to fluctuate significantly, or (3) filters follow a treatment process that generates significant amounts of algae. Section 7.2.3 of TR-16 has additional information on pretreatment options specifically for post-secondary suspended solids removal.

Design Considerations

Care should be given in designing pumping equipment, and pipes or conduits ahead of filter units, if applicable, to minimize shearing of floc particles. Consideration should be given in the plant design to provide flow-equalization facilities (also see Backwash Surge Control) to moderate influent quality and quantity, protect the downstream filter components, and preserve treatment effectiveness. Each filter shall be designed and installed so that ready and convenient access to all components and to the media surface is provided for inspection and maintenance.

Filter Types

High-rate filters may be of the gravity type or pressure type. Where greases or similar solids which result in filter plugging are expected, filters should be of the gravity type, and shall be preceded by a grease interceptor where food preparation or food service is provided (see Section D.5 of these Design Standards).

Filtration Rates

Total rapid-rate filter area for secondary treatment shall be provided in two or more units, and the filtration rate shall be calculated on the total available filter area with one unit out of service (TR-16, Section 7.2.8, and Ten States Standards, Section 112.3), except when justified by volume and size of installation. The expected design maximum suspended solids loading to the filter should also be considered in determining the necessary filter area. Engineering justification must be documented by the design engineer and approved by the reviewing engineer.

The hydraulic loading rate is a function of the media design, but generally the maximum hydraulic loading rate should not exceed 5 gallons per minute (gpm)/sq. ft. (3.4 L/m²s) based on the design peak hourly flow rate applied to the filter units.

The hydraulic loading rate for Rapid- or High-Rate filters should not exceed 5 ***gallons per minute*** (gpm)/sq. ft. (3.4 L/m²s).

However, the typical maximum intermittent (single-pass) filtration rate may be:

5 to 6 ***gallons per day*** per sq. ft. (within a range of 2.3 to 18.4 gpd/sq ft.) following secondary treatment; or

1 to 3 ***gallons per day*** per sq. ft. for intermittent media filters receiving primary effluent.

Refer to Ten State Standards 112 and TR-16 Sections 6.2.2, 6.2.6 and 7.2.10 for more on hydraulic and organic filter loading rates.

Selection of proper media type and size will depend on required effluent quality, the type of treatment provided prior to filtration, the filtration rate selected, backwash rate, and filter configuration. In dual or multi-media filters, media size selection must consider compatibility among media. Media shall be selected and provided to meet specific conditions and requirements relative to the project under consideration. The selection and sizing of the media shall be based on demonstrated satisfactory field experience under similar conditions. All media shall have a **uniformity coefficient (UC) of 1.7 or less**. The uniformity coefficient, effective size, depth, and type of media shall be set forth in the specifications (2004 Ten States Standards, page 110-7).

The designer has the responsibility for selection of media to meet specific conditions and treatment requirements of the project under consideration. However, **Table G-2** provides a list of typically acceptable media sizes and minimum media depths for intermittently-backwashed, high-rate gravity filters.

Table G-2. Media Sizes and Minimum Depths

	Single Media		Dual Media		Multi-Media	
	Size	Depth	Size	Depth	Size	Depth
	(mm)	(inch)	(mm)	(inch)	(mm)	(inch)
Anthracite	-	-	1.0-2.0	20	1.0-2.0	20
Sand	1.0-4.0	48	0.5-1.0	12	0.6-0.8	10
Garnet or Similar	-	-	-	-	0.3-0.6	2

Filter Appurtenances

Rapid-rate, gravity filters shall be equipped with wash-water troughs, surface wash or air scouring equipment; a means of measurement and positive control of the backwash rate; equipment for measuring filter head loss, positive means of shutting off flow to a filter being backwashed, and filter influent and

effluent sampling points. If automatic controls are provided, there shall be a manual override for operating equipment, including each individual valve essential to the filter operation. The under-drain system shall be designed for uniform distribution of backwash water (and air, if provided) without danger of clogging from solids in the backwash water. Provision shall be made for periodic chlorination of the filter influent or backwash water to control slime growths.

Pressure filters shall be equipped with means of measurement and positive control of the backwash rate, equipment for measuring filter head loss, valves and piping to isolate the filter being backwashed, filter influent and effluent sampling points, and a sight glass on the backwash discharge line. Provision for manual override of automatic controls, facilities for uniform distribution of backwash water, and provision for chlorine application, must be supplied for pressure filters as specified for gravity filters.

When chemical disinfection is not provided at the plant, manual dosage of chlorine compounds is acceptable.

Access and Housing

Each filter unit shall be designed and installed so that there is ready and convenient access to all components and the media surface for inspection and maintenance without taking other units out of service. The need for housing of filter units shall depend on expected extreme climatic conditions at the treatment plant site. If housing for the entire filter units are to be provided, the housing shall be constructed of suitable corrosion-resistant materials, and provided with adequate heating and ventilation equipment to minimize problems with excess humidity.

As a minimum, all controls shall be enclosed, and the structure housing filter controls and equipment shall be provided with adequate heating and ventilation equipment to minimize problems with excess humidity.

Backwash

Gravity filters are typically backwashed intermittently; pressure filters may be backwashed intermittently or continuously. For gravity filters, the backwash rate shall be adequate to fluidize and expand each media layer a minimum of 20 percent based on the media selected. The backwash system shall be capable of

providing variable backwash rates. Minimum and maximum backwash rates shall be based on demonstrated satisfactory field experience under similar conditions. A typical minimum backwash rate would be 20 gpm/sq. ft. (13.6 L/m²s). Neither TR-16 Section 7.2, nor Ten States Standards Section 112.4 specifies minimum or maximum backwashing flowrates, but allows them to be based on design configuration, manufacturer's recommendations, whether the backwashing is manual or automatic, and field experience.

The Ten State Standards requires provision be made for a minimum backwash period of 10 minutes, and a media expansion of 20 percent. Filtered water from the clear well or chlorine tank shall be used as the source of backwash water. Waste filter backwash water shall be treated, e.g. returned to the head of the plant. A combination of filtered effluent and compressed air may also be used to backwash. The impacts of backwash flows and associated pollutant loads must be considered relative to the overall capacity and operation of the facility.

Backwash Pumps

Pumps for backwashing filter units shall be sized and interconnected to provide the required backwash rate to any filter with the largest pump out of service.

Backwash Surge Control

The hydraulic and organic load from waste backwash water shall be considered in the overall design of the treatment plant. The rate of return of waste filter backwash water to treatment units shall be controlled such that the rate does not exceed 15 percent of the design average daily, dry-weather, flow rate to the treatment units. Surge tanks may be eliminated if adequate flow equalization capacity is used in the treatment scheme. The backwash water should then discharge directly to the equalization tank.

If needed, surge tanks shall have a minimum capacity of two backwash volumes, although additional capacity should be considered to allow for operational flexibility. Where waste backwash water is returned to the head of the plant for treatment by pumping, adequate pumping capacity shall be provided with the largest pump out of service.

Backwash Water Storage / Effluent Clear-well

An effluent clear-well, or other unit following filtration, having a minimum capacity of one backwash volume per filter shall be provided as a backwash water supply for multiple filter systems. If flow

equalization is used, volume shall be 1/2 of the above. For single filters, volume for two complete backwash cycles must be provided

Proprietary Equipment

Where proprietary filtration equipment not conforming to the preceding requirements is proposed, data which supports the capability of the equipment to meet effluent requirements under design conditions shall be provided; also refer to Section 7.2 of TR-16. Such equipment will be reviewed on a case-by-case basis at the discretion of the regulatory agency; refer to Section H of these Design Standards.

G.3. Physical-Chemical Unit Processes

The Physical-Chemical processes are used more frequently and effectively to remove phosphorus than nitrogen. NEIWPC TR-16 addresses Physical-Chemical Phosphorus Removal in Section 7.4 and the Chemical Precipitation for Phosphorus Removal in Section 7.4.1. Biological nutrient removal is covered in Section G.4 of these Design Standards; in TR-16 Section 6.3.10; and in the Ten State Standards Chapters 90 and 110.

G.3.a. Colloid (Suspended Solids) Removal

Systems employing the physical-chemical mode of treatment in lieu of, or to compliment, biological treatment should include at least coagulation, settling, and filtration.

1. Chemicals - Coagulation of solids may be accomplished through use of lime, or aluminum or iron compounds (usually alum or ferric chloride). In the use of lime, neutralization of high pH may be required prior to discharge.

Polyelectrolytes (polymers) may be used alone as coagulants, or as aids to other coagulants. Coagulant aids may be used to optimize floc growth and hasten settling, and may allow the dosage of primary coagulant to be decreased. When polymer use is anticipated, the potential effects on solids generation and handling should be considered during design. Treatment systems should have the flexibility to allow the use of polymer as a coagulant aid if it becomes necessary in the future.

The use of a liquid supply of metal salt instead of a dry form should be considered at small plants because handling requirements would be decreased, although transportation costs may be prohibitive.

Safety equipment appropriate for the chemical type and form must be provided. This may include dust masks, respirators, goggles, face shields, and protective clothing. If dry chemicals are used, it may be necessary to install dust collectors in storage and handling areas. Optimum chemical type and dosage should be determined by jar testing, preferably followed by pilot plant work. Dosage equipment should be sized to cover a range up to twice the recommended dosage, and should be constructed of materials that will resist the caustic or corrosive nature of the chosen chemical.

2. Coagulation - Provision for a complete mix of the chemical with the sewage as quickly as possible should be included. Detention time in this facility should be no longer than two minutes. Gentle agitation should be provided for at least 30 minutes, to allow floc to form.

3. Settling - Detention time in the settling facilities should be at least two hours. Facilities should be designed to achieve a surface settling rate no greater than 800 gpd/sq. ft. Sludge withdrawal mechanisms should be designed so as to prevent disruption to or loss of the floc blanket. Inlets and outlets should be designed to dissipate velocity, to distribute and receive flow equally, and to prevent short-circuiting. A baffle should be provided at the outlet end to retain oils, greases, and other floatable material.

4. Filtration - Filtration may be of either the high-rate/rapid rate type or intermittent type. Filter criteria are presented in Section G.2 of this publication. Filter dosage may be up to 5 gallons per minute/sq. ft. for high-rate filters (Section G.2) and 10 gallons per day/sq. ft. for the open intermittent type (see Section F.2).

G.3.b. Physical-Chemical Phosphorus Removal

1. Chemicals - Aluminum or iron compounds (salts) and lime will react with orthophosphate to form insoluble phosphate colloids. Alum is generally preferred as lime may generate excessive sludge and iron compounds may result in iron leakage into the effluent. (Note safety concerns in Section A, above.)

2. Dosing - Chemical addition may occur prior to primary clarifiers (including septic tanks), secondary treatment tanks, or final clarifiers either in a separate mixing basin or in a turbulent portion of the system. TR-16 Section 7.4.1.2 recommends the addition of metal salts to both primary and secondary treatment facilities as the most cost-effective means of phosphorus

removal by chemical precipitation. However, ferrous salts must be added prior to, or in, the secondary treatment (aeration) tank. The ferrous salt must be oxidized to the ferric state before it will precipitate out iron phosphate in the clarifier. Adding ferrous chemicals to the final clarifier will result in increased chlorine demand, increased suspended solids resulting in ineffective UV disinfection, and higher TSS and color in the plant effluent

If nitrification is desired during biological treatment, dosing should occur prior to primary settling, to reduce BOD load on the biological system. If a high percentage of the total phosphorus is present as polyphosphate or organic phosphate, dosing should occur after biological conversion to orthophosphate. When high levels of detergents (polyphosphate) are present, dosing with aluminum or iron compounds should occur after biological treatment to avoid competing side reactions of the detergent with the metal ion.

TR-16 Section 7.4.1.3 provides additional information on chemical application points, where adequate turbulence will ensure complete mixing of the metal salt with the wastewater, and also located downstream of the plant's internal recycle streams, such as recycles from sludge treatment facilities.

Jar tests for dosage estimation should simulate treatment plant conditions. Mixing speed should be adjusted to match the hydraulic regime in the plant, but the duration of mixing should be the same as the jar test. To approximate settling conditions the mechanism should not be motionless, but should turn very slowly. If possible, jar tests should be followed by 30-day pilot-plant or full scale tests.

Phosphorus levels show significant diurnal variations, so it is recommended that the dosage be adjusted regularly (normally 3 to 5 changes in dose rate per day during initial phases of application). Flow equalization may be provided to reduce the number of necessary dosage adjustments.

Overdosing will prevent floc growth and settling, and can dangerously reduce the pH through the formation of hydroxide salts. The addition of lime or sodium hydroxide may be needed to prevent this, especially if aeration is not provided in the aeration tank to oxidize ferrous chemicals to the ferric state.

3. Optimization: To maintain effluent quality, pH adjustment is necessary. Addition of metal salts

or lime can be followed by addition of polyelectrolyte to improve settling.

Multi-media filtration is recommended if consistently low phosphorus levels (below 1 mg/l) are necessary.

4. Sludge: During design, consideration must be given to the generation and dispersal of additional sludge from chemical treatment.

TR-16 Section 7.4.1, “Chemical Treatment - Phosphorus Removal” recommends that the use of chemical precipitation for phosphorus removal process be based on:

- Chemical analysis of the influent wastewater
- Objective effluent criteria
- Capability of alternative wastewater treatment processes such as biological nitrogen and phosphorus removal processes; and
- Overall economies of the alternative processes

TR-16 Sections 7.4.1 and 7.4.2 also provide recommendations for economic evaluation, discussion of the various forms of P in wastewater, a comment on the declining use of lime in comparison to metal salts, and the need to provide adequate alkalinity. TR-16 Section 7.4.3 gives recommendations for chemical storage and feed facilities.

Phosphorus levels show significant diurnal variations, so it is recommended that the dosage be adjusted regularly (normally 3 to 5 changes in dose rate per day during initial phases of application). Overdosing will prevent floc growth and settling. Flow equalization may be provided to reduce the number of necessary dosage adjustments.

The Ten State Standards Chapter 110 provides both duplicative and complementary design guidance for phosphorus removal by chemical treatment (but not for biological removal). Design guidance and specifications from both TR-16 and the Ten States Standards should be followed.

G.3.c. Physical-Chemical Nitrogen Removal

There are two physical-chemical removal methods that can be used in NYS for wastewater treatment of ammonia: selective ion exchange, and breakpoint chlorination. TR-16 Sections 7.6.3.2 and 7.6.3.3 provide guidance for ion exchange, and advanced oxidation processes (AOPs), respectively, for Total Organic Carbon (TOC) removal. There is no guidance on these systems in the Ten State Standards.

Neither of these systems has been used in NYS due to the superior cost-benefit of biological removal. See *Section G.4.a Biological Nutrient Removal including use of Activated Sludge Recirculation in Secondary Treatment Units*.

G.3.d. Membrane Processes and Membrane Bio-reactors (MBRs)

The 2004 Ten State Standards has no information on Membrane Processes or Membrane Bio-Reactors (MBRs). *NYSDEC has drafted guidance for municipal wastewater reuse. For non-municipal, non-industrial wastewater the following guidance from TR-16 should be followed for treatment systems using membrane processes.* TR-16 Chapter 7 provides filtration design guidance for advanced suspended solids removal, chemical phosphorus removal, and total organic carbon removal. Section 7.3 Membrane Filtration, has guidance on four categories of filtration for removal of suspended or dissolved particles:

- Microfiltration (MF): 0.1 to 10 microns
- Ultrafiltration (UF): 0.01 to 0.1 microns
- Nanofiltration (NF): 0.001 to 0.01 microns
- Reverse osmosis (RO): Less than 0.001 microns

Membrane Bio-Reactors (MBRs) are covered in Chapter 6 Biological Treatment in Section 6.3.8, with effluent limits given in Table 6-11 also shown here.

Effluent BOD	mg/L	<5
Effluent TSS	mg/L	<0.5
Effluent NH ₃	mg/L	<1
Effluent TN	mg/L	<10
Effluent turbidity	NTU	<1

1. Metcalf and Eddy, Fourth Edition 2003, p 1127, 1128

G.3.d.1. Microfiltration (or microscreening / microstraining)

Typically membrane systems that remove suspended solids following secondary treatment utilize either microfiltration or, more commonly, ultrafiltration. Microfiltration and ultrafiltration generally operate at lower pressures and can be configured in pressure-driven or immersed, vacuum-driven applications.

Either can be used following secondary or tertiary treatment systems, depending on overall process goals,

with micro-filtration generally targetted towards removal of precipitated suspended solids, and ultra-filtration toward organics removal.

G.3.d.2. Ultrafiltration (or membrane filtration)

Ultrafiltration has the significant benefit of removing residual BOD and bacteria, enhancing the performance of subsequent disinfection processes. TR-16 Section 7.3 outlines the design process and components including pretreatment, redundancy, backwashing and residuals handling.

TR-16 design limitations or preferences for micro- and ultra-filtration units (TR-16 Table 7-1) are for:

- Influent TSS levels between 5 and 30 mg/L; minimized to extend filtration run time.
- System redundancy to allow system backwashing and membrane replacement.
- Surge tank to dissipate the hydraulic and solids impacts of backwash flows on downstream components.

TR-16 design limitations for backwashing and maintenance are for:

- Redundancy for feed, backwash, and waste pumps should be provided, being calculated with the largest unit out of service for each pumping system.
- Multiple modular units are desired from an operating and cost perspective. Adequate redundancy should account for the largest unit being out of service and the other units operating at a flux level of 50 percent of the membrane's useful life.

G.3.d.3. Nanofiltration (or Selective Ion Exchange)

Nanoparticles are being discharged to wastewater treatment plants and OWTs in personal care products and other consumer products. Recent research has been directed toward its fate in WWTPs, specifically its effect on activated sludge, removal and effluent limitation.

Nanofiltration, or Selective Ion Exchange, is typically utilized for the removal of heavy metals or ionically charged organics. Numerous natural and synthetic resins are available typically in bead or granular form. Ion-exchange resins can be classified as either cations or anions. The potential for resin fouling mandates an influent wastewater stream low in suspended solids and organic content. Resins can often be regenerated on-site; however, provisions must be made to deal with the waste regeneration streams. Multiple units are essential for operation and during regeneration periods (TR-16 Section 7.6.2).

Water Environment Research Foundation (WERF) research gives an indication of the potential problem for WWTPs. Treatment methods are considered innovative. Design engineers should refer to Section H for the department approval process.

In *Impact of Silver Nanoparticles on Wastewater Treatment*⁵, the WERF investigators evaluated how silver nanoparticles would affect wastewater treatment systems and anaerobic digestion. Silver nanoparticles (AgNPs, nanosilver) are a frequently used nanomaterial with a wide range of industrial and consumer applications, including fiber coating, detergents, and hydrogels and plastics to prevent bacterial and fungal growth.

Nanoparticles released from various nanotechnology-enhanced consumer products will inevitably enter our sewers and wastewater treatment plants (WWTPs). The researchers set up several lab-scale wastewater treatment modular units using activated sludge processes designed to remove organic matter and nutrients in wastewater. The results demonstrated that nitrifying bacteria were especially susceptible to inhibition by silver nanoparticles.

At a concentration of 0.4 mg/L total Ag, a mixture of positively charged silver ions and AgNPs (50:50 in mass ratio, average size =15-21 nm) inhibited the growth of nitrifying bacteria from the Modified Ludzack-Ettinger (MLE) bioreactor (TR-16 Section 6.3.10) by 11.5 percent. In an experiment on shock loading of 100% AgNPs (lasting for 12 hours), a peak concentration of 0.75 mg/L total Ag in the activated sludge basin (more than 95% associated with biomass) was detected, and about 50% nitrifying bacterial growth inhibition (or nitrification inhibition) accompanied with a slight accumulation of nitrite concentration in wastewater effluent was observed.

Studies of anaerobic digestion indicate that AgNPs at concentrations of 19 mg/L (19,000 ppb) or above in the biomass might inhibit anaerobic microbial activities. Most of the silver particles are in the activated sludge. After considering concentration factor and safety factor, the suggested threshold concentration of total silver including nanosilver in wastewater influent is 0.1 mg/L. This study suggests that accumulation of silver in activated sludge could have a detrimental effect on wastewater treatment, if the concentration reaches threshold levels.

⁵ WERF Report U3R07, Zhiqiang Hu, 01 Apr 2011 • ISBN: 9781843393993, 50 Pages, [WERF Research Report Series](#).

G.3.d.4. Hyperfiltration (or Reverse Osmosis and Membrane Filtration)

This technology provides an exceptional effluent quality but is still limited in the removal of dissolved organics e.g. Total Organic Carbon. A high-quality influent wastewater stream is essential to its operation. Extensive pretreatment is required to remove suspended solids, extreme pH values, oils and grease, and membrane-destructive chemical constituents. Similar to ion-exchange methods, membranes can be cleaned on-site; however, they often generate significant waste streams both difficult to treat and requiring significant detention. Multiple-membrane systems are essential for operation, cleaning, and membrane replacement purposes (TR-16 Section 7.6.2).

G.4. Biological Nutrient Removal

For onsite wastewater treatment systems discharging to Soil-based Treatment Systems (STSs), biological nutrient removal can be accomplished using shallow soil absorption systems for nutrient uptake by the system cover vegetation.

For further down-gradient aquifer protection, permeable reactive barriers can be designed to remove both nitrogen and phosphorus from the subsurface path towards groundwater recharge by serving to increase the travel time, and provide adsorption sites and filtration. Such in-ground barrier technologies are very site-specific, and can be limited by the costs of replacing the reactive media. Replacement frequency due to exhausting adsorption sites should be investigated by the design engineer researching similar applications at locations with similar soils, and geology.

For treatment plant unit processes, several types of treatment processes are available for biological removal of nitrogen and phosphorus from wastewater, including biological nitrogen removal processes, biological phosphorus removal processes, and biological processes for simultaneous removal of nitrogen and phosphorus. Ten State Standards does not discuss any tertiary biological treatment systems for either N or P. TR-16 provides general guidance in Section 6.3.10 Biological Nutrient Removal (BNR) by Suspended Growth Systems and recommends case-by-case reviews based on the use of process simulators, Tables 10-5 and 10-6 in EPA's 2010 *Nutrient Control Design Manual*, and other available information. Information on design considerations, features, performance capabilities, operating requirements, and other factors to consider when designing new plants, plant expansions, and retrofits of

existing plants for nitrogen and phosphorus removal also are included.

In selecting a process, or combination of processes, the design engineer must consider the following criteria:

- Influent wastewater characteristics, including five-day biochemical oxygen demand (BOD₅), chemical oxygen demand (COD), total Kjeldahl nitrogen (TKN), total phosphorus (TP), alkalinity, and temperature
- Seasonal variations in the influent wastewater characteristics caused by industrial contributors, population changes, and septage, where applicable
- Discharge limitations
- Seasonal variation of the discharge limits
- Existing treatment facilities at the plant site
- Cost

G.4.a. Biological Nitrogen Removal

In the first step of the two-step nitrification-denitrification process, ammonia and organic nitrogen present in the wastewater are oxidized to nitrite and then to nitrate by autotrophic nitrifying bacteria (*Nitrosomonas* and *Nitrobacter*) under aerobic conditions. In the second step, the nitrates are reduced to nitrogen gas by heterotrophic bacteria under anoxic conditions. The nitrogen gas produced from the denitrification process escapes into the atmosphere, removing total nitrogen (TN) from the wastewater.

TR-16 presents a brief explanation of the chemistry of the removal processes, including the consumption and generation of alkalinity by the nitrification and denitrification processes, respectively, provision of sufficient organic carbon for denitrification, and control of a low D.O. concentration prior to any return sludge flows to the head of denitrifying units (deoxygenation zone).

Biological nitrification and denitrification are discussed in several subsections of Chapter 6 in TR-16. Nitrogen removal using suspended growth systems, attached growth systems, and hybrids are all described. In suspended growth activated sludge systems, the microbial growth occurs in a mixed-liquor that is kept in suspension by turbulence created by aeration or mechanical mixing devices. In attached growth (fixed-film) systems, the microbial growth occurs on fixed surfaces.

Fixed growth systems (Section 6.2) can be configured using a number of stages, with more stages warranted as treatment requirements become more stringent.

Moving Bed Biofilm Reactors (Section 6.2.5) use media supporting a biofilm growth that can be either aerobic (for carbonaceous removal and nitrification) or anoxic (for denitrification).

Biological Filter Processes (Section 6.2.6) include biologically active filters (BAF) that encompass both aerobic (nitrification) and anoxic (denitrification) filters.

Suspended Growth Systems (Activated Sludge, Section 6.3) use selectors that can be anaerobic, anoxic, or aerobic: anoxic selectors are used in nitrifying, with a portion of the nitrified mixed liquor recycled to the anoxic zone for denitrification. Rotating Biological Contactor Systems (Section 6.4.3) can be configured to accomplish total nitrogen removal, if sufficient aerobic and anoxic SRTs are provided. Sequencing Batch Reactors (Section 6.3.7) can provide a moderately high level of nutrient removal by controlling the overall cycle time and the duration of the steps in the SBR sequence of operation.

Some of the suspended growth processes commonly used for biological nitrogen removal are single-sludge processes. The following are described in TR-16 Section 6.3.10, Table 6-12:

- Modified Ludzack Ettinger (MLE) process,
- Bardenpho process,
- cyclical aeration process, and
- oxidation ditch process.

Integrated Fixed Film/Activated Sludge Systems (Section 6.4.4) can incorporate media configured to work in an aerobic or anoxic environment, but the designer must understand the balance and synergy between the attached and suspended growth biomass components. Nitrification, denitrification and phosphorus removal design examples are provided in WEF's MOP 35: *Biofilm Reactors*.

Chapter 7 of TR-16 (2011) includes information on solids removal processes that may also be beneficial in achieving low TSS and nutrient limits.

Anoxic and Aerobic Reactor Design

The size of the biological nitrogen removal system should be determined based on the degree of treatment required, influent BOD and TKN loads, MLSS concentration, and the expected mixed liquor temperatures. Typically, the anoxic zone volume for domestic wastewater in most nitrogen removal

processes is 30 to 40 percent of the total reactor volume or 40 to 60 percent of the aerobic reactor volume. The nitrate recycle rates in MLE and Bardenpho processes depend on the design effluent nitrogen concentration. The greater the recycle rate, the greater the nitrate removal rate. Tables 6-13 and 6-14 in TR-16 outline other design parameters for nitrogen removal systems.

The following steps should help to determine the design loadings for the BNR system:

- Determine the average annual, maximum weekly, and maximum monthly BOD_5 , COD, and TKN loadings to the BNR systems from a minimum of 1 year of wastewater data or from data from similar plants.
- Determine the maximum weekly and maximum monthly peaking factors and the ratio of maximum weekly to maximum monthly average BOD_5 , COD, and TKN loadings to the BNR system.
- Use the peaking factor that corresponds to the period that the permit will demand.

A safety factor may be applied to the design of the BNR system depending on the uncertainty of the performance of the chosen technology.

When a maximum month peaking factor cannot be determined, a peaking factor of no less than 1.5 should be applied to annual average wastewater loading estimates.

Multiple anoxic and aerobic reactors and step feed should be considered in order to achieve a higher level of performance where configuration of the existing tanks (i.e., in the retrofit designs) is suitable or where permit limits are more stringent. Multiple anoxic and aerobic reactors can be created by constructing baffle walls within the reactors. Baffle walls should be constructed so that scum will not be entrapped in any portion of the reactors. Consider preventing back-mixing between the reactors when designing baffle walls.

Consider a deoxygenation zone at the end of the aerobic reactor if effluent is to be recycled back to the head of an anoxic reactor.

Mixers (i.e., submersible or overhung) should be provided in the anoxic zones. Depending on the tank geometry and mixer design, the mixer horsepower can vary between 0.03 and 0.1 per 1,000 gallons of tank volume.

The use of alternate carbon sources should be evaluated as alternatives for methanol. If methanol or

chemicals that have properties similar to methanol are used as supplemental carbon sources, then appropriate local, state, and federal fire protection codes must be followed when designing chemical storage, off-loading, and feed facilities. The supplemental carbon feed storage capacity should be sufficient to provide treatment for a minimum of 30 days.

Methanol or chemicals that have properties similar to methanol should be stored outdoors in a diked containment area. Venting in the storage tanks should be controlled by conservation-type vent(s).

Overdosing of the supplemental carbon source may result in a BOD₅ violation of the permitted effluent limitation. Therefore, the feed pumps should be calibrated so that the supplemental carbon source is just sufficient to meet the stoichiometric requirement for denitrification. Varying the supplemental carbon feed rate by manual adjustment or flow pacing with on-line nitrate analyzers should be considered.

The aerobic reactor (i.e., the nitrogen stripping zone) that follows the anoxic reactor should have adequate HRT to remove any additional BOD caused by overdosing of the supplemental carbon source. Typically, an HRT of 30 to 60 minutes at design average flow is provided for nitrogen stripping zones.

The aeration equipment design in BNR plants must be sufficient to provide 0.6 to 1.4 lb O₂/lb BOD depending on the MCRT of the system (EPA Design Manual, *Fine Pore Aeration System*) and 4.6 lb O₂/lb TKN present in the influent wastewater to produce a minimum DO concentration of 2.0 mg/L for the following conditions:

- *Maximum Condition:* The maximum oxygen demand should be based on the maximum day TKN and BOD₅ mass loadings for the design condition, maximum mixed liquor temperature, maximum anticipated diffuser fouling, and the low end of the Alpha factor range recommended for the aeration system. In computing the maximum oxygen demand, the organic carbon removed by the denitrification process should not be counted.
- *Minimum Condition:* The minimum oxygen demand should be based on the minimum day TKN and BOD₅ mass loadings for the flow conditions at the time of startup, minimum mixed liquor temperature, minimum anticipated diffuser fouling, and the high end of the Alpha factor range recommended for the aeration system. If the actual wastewater characteristics data are not available to calculate the minimum day conditions, 75 percent of the average anticipated loading at the time of startup should be used to compute the minimum day condition. In computing the minimum oxygen demand, the organic carbon removed by the denitrification process should be accounted for at the rate of 2.86 pounds

of oxygen demand satisfied per pound of nitrates denitrified. The oxygen demand satisfied by the denitrification process should be subtracted from the calculated oxygen demand based on TKN and BOD₅ loadings.

Provisions for pH and DO monitoring and control should be considered. Other provisions regarding aeration tanks and related process equipment as presented in Chapter 6 of TR-16 (1998 edition) also apply.

Clarifier Design

The following design criteria are discussed in TR-16 Section 6.2.7 and should be used ***for clarifiers that follow suspended growth nitrification-only reactors or suspended growth nitrification and denitrification reactors.***

- Multiple units capable of independent operation should be provided. Consider whether the process can be maintained if a clarifier is taken out of service. If a single clarifier system is proposed, the engineering justification should be documented and approved by the reviewing engineer.
- Inlet conditions should provide for effective velocity dissipation with minimum shearing of the biological floc.
- Use of coagulant aids should be considered especially where peak hydraulic loadings in existing clarifiers are beyond the limits recommended herein or where effluent TSS concentrations of less than 15 mg/L are required. The design engineer should compare the capital and long-term O&M costs of the polymer feed system with the capital and O&M costs of building an additional clarifier.
- Effective influent baffling and scum removal equipment should be provided. Scum collected from the clarifiers should not be returned to the aeration tanks. A suitable method for dispersal of the scum should be considered.
- Provisions for measuring, sampling, and controlling the individual clarifier underflow (i.e., RAS) should be provided.
- Provisions for chlorine dosing of the RAS for control of filamentous bacteria should be provided.
- The capability to transfer sludge from one reactor to another and from one stage to another should be included, along with provisions for wasting sludge from the system.
- Clarifier peak overflow rates at maximum hourly flows should be less than 800 gpd/sq.ft. and peak solids loadings should be less than 30 lb/day/sq.ft. Under average conditions, the overflow

rate and the solids loading rate should not exceed 400 gpd/sq.ft. and 20 lb/day/sq.ft., respectively. The sidewater depth of the final clarifiers should be no less than 13 feet.

- The design engineer should consider using sludge deflector baffles, also called Stamford or Crosby baffles, to redirect density currents back to the center of circular clarifiers.

Design guidance for secondary clarification is given in several subsections of Section 6.3.5, TR-16, and 72.23 in the Ten State Standards. Either may be used for intermediate-sized systems as amended under the various modes of activated sludge in these Design Standards (Sections F.3 – F.6).

G.4.b. Biological Phosphorus Removal (BPR)

Biological Phosphorus Removal (BPR) or enhanced biological phosphorus removal (EBPR) is discussed in TR-16 Section 6.3.10 Biological Nutrient Removal (BNR) by Suspended Growth Systems. EBPR is achieved by wasting sludge from a treatment system that has an excess amount of phosphorus stored in the bacterial cells. The two most commonly used EBPR processes are:

- Phostrip process (proprietary)
- A/O process (TR-16 Table 6-13)

These processes are sidestream and mainstream processes, respectively. The Phostrip process uses the return sludge flow for phosphorus removal. In the A/O process, phosphorus removal reactions occur as the wastewater passes through the units of the treatment system. Neither of these EBPR systems has been used in NYS due to the superior cost-benefit of physical-chemical removal. *See Section G.3.b. Physical-Chemical Phosphorus Removal.*

The Biological Nutrient Removal systems in TR-16 Table 6-13 include four treatment systems that remove both nitrogen and phosphorus. EPA's 2010 *Nutrient Control Design Manual* includes two tables (10-5 and 10-6) that provide an excellent starting point for engineers and reviewers evaluating a BNR (total nitrogen and phosphorus) design.

G.5. Constructed Wetlands

The use of natural wetlands as receiving waters is not considered due to the undesirable impacts of fluctuating hydraulic and organic loadings.

Table F-1 shows the use of Vegetated Submerged Beds (VSB) and Free Water Surface (FWS) Constructed Wetlands for suspended solids removal by sedimentation. However, for carbonaceous BOD and ammonium removal, and biological nitrification-denitrification, only Free Water Surface (FWS)

Constructed Wetlands are recommended. Vegetated Submerged Beds (VSB) are recommended for biological denitrification only, following another unit process that provides nitrification.

The 2002 EPA Onsite Wastewater Treatment Systems Manual, Technology Fact Sheet (TFS) 5 describes Vegetated Submerged Beds (VSB) and Other High-Specific-Surface Anaerobic Reactors, but no design standards are given. Vegetated Submerged Beds (VSB) and Subsurface Flow (SF) Constructed Wetlands are equivalent. The 2002 EPA TFS 7 describes Stabilization Ponds, Free Water Surface (FWS) Constructed Wetlands, and Other Aquatic Systems.

TR-16 Chapter 9, Land Treatment, Treated Effluent Disposal, and Treated Effluent Reuse should be followed for additional guidance on the use or selection of constructed wetlands (Section 9.4.7). The primary reference in TR-16 is Constructed Wetlands and Aquatic Plant Systems for Municipal Wastewater Treatment- Design Manual⁶.

The 1998 WEF MOP #8 Design of Municipal Wastewater Treatment Plants contains some information on the three most generally recognized types of constructed wetlands: FWS, SF and Vertical Flow (VF). Section 13 primarily compares Free Water Surface (FWS) systems with Subsurface Flow (SF) systems. The Vertical Flow (VF) wetlands are only described briefly, and their construction is generally similar to either an intermittent sand filter (European) or a vegetated re-circulating gravel filter (North American).

The 1993 “Subsurface Flow Constructed Wetlands for Wastewater Treatment: A Technology Assessment,”⁷ only evaluates the Subsurface Flow (SF) systems.

WEF MOP #8, Chapter 21 and TR-16, Section 11.9.6 both discuss Reed Beds as a sludge dewatering method, not as a constructed wetland for secondary treatment.

The 2006 WERF Report briefly discusses Vertical Flow wetlands and Sludge Dewatering Beds or Reed Beds. The report notes that “Literature on Vertical Flow (VF) wetlands and Sludge Dewatering Beds is readily available.” [Dating back to the 1960’s] “Design information for small-scale Free Water Surface

⁶ Constructed Wetlands and Aquatic Plant Systems for Municipal Wastewater Treatment- Design Manual, U.S. Environmental Protection Agency, Office of Research and Development, Center for Environmental Research Information, Cincinnati, OH, 45268, EPA/625/1-88/022, September 1988.

⁷ U.S. EPA, “Subsurface Flow Constructed Wetlands for Wastewater Treatment: A Technology Assessment,” EPA 832-R-93-008. U.S. Environmental Protection Agency, Office of Water, Washington, DC 1993.

(FWS) and Vegetated Submerged Beds (VSB)/Subsurface Flow (SF) wetlands is not as established”.

There is no information in the references above regarding the use of forced bed aeration as a modification for constructed wetland design. Such a design would be covered under Section H. INOVATIVE SYSTEMS AND VARIANCES FROM THESE DESIGN STANDARDS. (Procedure for new technology review).

DRAFT

H. Innovative Systems and Variances from these Design Standards

H.1. Innovative Systems: Procedure for reviewing technology

An innovative system may be an entirely new technology, one that has been accepted for common use in other states or countries but is new to New York, or one that has been proven effective in a field other than wastewater treatment. It is any wastewater treatment system technology that is either not in these Design Standards, or that is required by these Design Standards to undergo departmental review before approval.

It is a policy of the department to encourage rather than obstruct the development of any acceptable methods or equipment for treatment of wastewater. The absence of some types of wastewater treatment processes or equipment in these Design Standards should not be construed as precluding their use, but only that consideration by the reviewing agency will be on the basis of information submitted with the design.

Innovative technology review is only applicable to projects applying for an individual SPDES permit. The burden of proof is on the licensed professional engineer to demonstrate that a given process, system of unit processes and equipment, or a specific application of either, will have a reasonable and substantial chance of providing adequate, reliable and long-term treatment, without excessive energy consumption or operator attention.

The following outlines DEC's innovative technology review procedure.

1. The system designer needs to show DEC that any system will work under various conditions, and can meet the SPDES permit conditions for the receiving water:
 - The technical information from a sales brochure is not sufficient. Site-specific and application-specific information is needed, and the applicable design criteria of the Design Standards need to be met or exceeded.
 - Often innovative technologies will not be in the Design Standards, so it is necessary for the design engineer to determine what unit process(es) their technology contains and show how it meets or exceeds performance specifications in the Design Standards, e.g. application rates, effluent limits.
 - A technical description must show calculations that the unit will provide the

pollutant removals required by the SPDES permit for the specific facility's wastewater being treated.

2. DEC has witnessed many problems due to a lack of clarity as to the roles of the manufacturer, design engineer and contractor in assuring proper specification and installation of parts, and entire units. Therefore, DEC will require provisions that:
 - prevent contractors from offering substitutions that attempt meet “or equal” specs, but cause problems with the system operation
 - assure availability of necessary equipment and parts should the manufacturer go out of business
 - prevent early failure in design/specifications by incorporating cathodic protection for steel, avoiding the use of inappropriate dissimilar metals where the unit is mated in some way with a contractor-supplied unit or part.
 - indicate thorough review by the design engineer submitting the application
 - indicate the appropriate steps for public or private review, legal liabilities, etc., are being taken.
3. DEC will require contract and specification details supporting system longevity:
 - Are there seemingly minor omissions that may lead to operational problems?
 - Is it designed to last (i.e. durable construction materials, corrosion protection, and
 - Is availability of replacement parts or consumables necessary to the process?
4. Before DEC accepts a design, a record of performance that demonstrates reliability will be required. The full-scale “reference” system must:
 - be in a similar climate and other conditions which may be encountered in the area of the proposed installation (including diurnal variations)
 - have similar effluent limitations and under various ranges of influent strength
 - have sampling results to show removal effectiveness – sets of composite samples over several months, including all seasons
 - provide data appropriate for *all* permit parameters and flow rates, and other appropriate information
5. The design engineer should review the details of the installation and indicate in the permit application that he (or his representative) will spend sufficient time during

construction to make sure everything is properly installed, and that the wastewater treatment system owner will provide an appropriately certified operator, and adequate service time to manage and maintain the system once it is operational.

6. DEC does not give statewide approvals to innovative designs or technologies. Any application of an innovative technology unit will be site-specific. Permits are tailored for specific situations, e.g., receiving stream low-flow, various water body classifications, size of discharge, etc. What might be acceptable in one place may be completely inappropriate in another.
7. DEC may approve use of innovative technology on an experimental or pilot basis only. If conditions are attached to a permit, or order, the permittee would be required to follow them by getting sample results, operational data, test results and engineering evaluations demonstrating the efficiency of the treatment process(es). DEC or another reviewing authority may require a contingency plan including replacement of the innovative/alternative system should it fail to consistently meet discharge limits, or if it creates a nuisance.
8. DEC may ask for data on system performance over time. DEC may require that appropriate testing be conducted at certain intervals and evaluations be made under the supervision of a licensed professional engineer with expertise in the treatment process(es) proposed.
9. DEC may require other permits besides the SPDES permit, so design engineers should be aware of site limitations, e.g., they should not use invasive species in a “polishing” wetland, or in a sludge drying bed.

H.2. Variances from these Design Standards for Replacement Systems

A variance from these standards for replacement wastewater treatment systems may be granted with all of the following conditions:

- The replacement system may be granted minimum variances from the Standards when full compliance cannot be achieved.
- The replacement system must provide equal or better environmental and human health protection than the previous system, and meet all SPDES permit requirements.

- Variances will not be granted when the replacement system is designed for an increase in flow.
- Granting a variance shall not relieve the applicant of the responsibility to comply with all other applicable State and local laws, rules or ordinances.

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I. Surface Water Discharges: Disinfection And Reoxygenation

I.1. Introduction

Disinfection design guidance is limited to chlorination, dechlorination, and ultraviolet disinfection. Other methods may be considered through the process described in Section H.

Effluent reoxygenation, also described in the Water Environment Federation Manual of Practice Number 8 (MOP #8), “Design of Municipal Wastewater Treatment Plants,” includes diffused/mechanical and cascade aeration, outfall specifications, and controlled release requirements.

I.2. Surface Discharge Prohibitions

It should be noted that DEC does not allow surface water discharges from individual domestic-type sewage discharges from family residences with a total flow of less than 1,000 gallons per day. For existing homes demonstrating hardship, an individual SPDES permit is required for surface discharges per the NYSDEC Technical Operation and Guidance Series (TOGS) 1.2.4 [Individual Sewage Treatment System Discharges to Surface Waters](#), October 1990.

I.3. Disinfection and Dechlorination

I.3.a. Disinfection

The purpose of disinfection is to continuously reduce or eliminate harmful bacteria, viruses and pathogens from the treated effluent for public health and water quality protection. The most common disinfectants for on-site treatment systems are chlorine and ultraviolet light (UV). The use of UV is preferred as no residual products will be introduced into the wastewater. Advantages and disadvantages must be weighed when choosing a method of disinfection. Although the use of UV and chlorination are the only disinfection methods discussed herein, other methods including ozonation will be considered under the process described in Section H of these standards.

Where New York State is a member of an interstate compact, the State will conform to the interstate commissioner's rules on disinfection if they are more restrictive than state requirements.

I.3.b. Chlorination

Chlorine disinfection is usually accomplished with liquid (sodium hypochlorite), solid (calcium hypochlorite), or gaseous chlorine dioxide. The chlorination method selected should be based on wastewater flow rates, application and demand rates, pH of wastewater, cost of equipment, chemical availability, required maintenance and safety concerns. The form of chlorine most often used is solid chlorine tablets and liquid sodium hypochlorite. Chlorine in a gaseous form is usually reserved for larger municipal wastewater treatment systems and will not be discussed in this document.

Chlorination feed equipment should be adequately sized to produce a concentration of chlorine to dependably and consistently reduce the coliform concentration to that specified in the SPDES permit for the installation.

Chlorine feed systems can consist of stack-feed devices for chlorine tablets or solution feeders for sodium hypochlorite systems. Tablet feed systems may be preferable for small discharge systems (flow rate under 30,000 gpd). Liquid hypochlorite storage tanks must be constructed of a compatible material and should provide ample volume for several weeks of operation. Positive displacement pumps, that are capable of feeding a measured volume of hypochlorite solution during a specific time period, must be used in solution feeders. Liquid feed lines should be protected from freezing. Chlorination feed equipment typically is fixed to the inlet end of the chlorine contact tank.

Chlorine should be applied to the wastewater in an area that can provide adequate mixing. After mixing, a minimum contact period of 15 minutes at peak hourly flow or maximum pumping rate shall be provided in a chlorine contact tank.

Normally a 0.5 mg/l total chlorine residual present after 15 minutes of contact time will be sufficient to reduce the coliform concentration to the acceptable level. Effluent monitoring will indicate if higher concentration levels should be maintained.

The chlorine contact tank should be constructed with baffles so as to prevent short-circuiting of flow and to prevent any floating material from leaving the tank. Over-and-under or end-around baffling shall be provided for this purpose. Provision for draining the contact tank for cleaning purposes shall be included. The drain should be valved, and the tank bottom should taper to the drain or a sump to facilitate cleaning and draining.

Periodic analysis of both the chlorine residual after contact time and bacterial analysis prior to discharge are recommended and may be a permit requirement. The chlorine residual typically should be maintained in a range of 0.5 mg/l to 2.0 mg/l.

I.3.c. Dechlorination

Dechlorination of a chlorinated effluent discharge may be necessary to protect the receiving stream, and may be achieved by aeration, activated carbon, and chemical treatment. Sulfur dioxide has been the most common chemical used as a dechlorinating agent, but other chemicals such as sodium bisulfite and sodium metabisulfate may also be used. Dechlorination chemicals should be applied in an area where the effluent flow is turbulent and short-circuiting is minimal, just prior to discharge. A contact period of 1 to 5 minutes should be sufficient for reaction to occur.

If the design of a dechlorination system is necessary for the treatment process, the guidelines outlined in TR-16, *Guide for the Design of Wastewater Treatment Works* (Section 8.4) or Ten State Standards Section 103 must be followed.

I.3.d. Ultra-Violet Disinfection

Ultraviolet (UV) disinfection system utilizes UV radiation, generated by an electrical current through a mercury vapor lamp, to penetrate the genetic material of micro-organisms and interfere with their ability to reproduce. The optimum wavelength to effectively inactivate microorganisms is in the range of 250-270 nanometers (nm).

The effectiveness of a UV disinfection system depends on the characteristics of the wastewater, the intensity of the UV radiation, the amount of time the microorganisms are exposed to the radiation, and the reactor configuration. Therefore, the presence of excessive particular matter, turbidity, dissolved compounds that adsorb UV, short circuiting of flow through the reactor, and accumulation of substances on the lamps, can all reduce the effectiveness of UV systems.

The ultraviolet transmittance (UVT) or absorbance properties of water are critical for UV to be effective. UV transmittance (UVT) should be 65% or greater, and total suspended solids less than 30 mg/l. Pretreatment by intermittent sand filtration is recommended, but effective disinfection can be achieved following secondary treatment without filtration. Several manufacturers produce in-line UV disinfection systems that can be used for disinfection of treated effluent. Septic tank effluent or effluent with high TSS would be difficult to disinfect adequately with UV and would not be considered an acceptable

disinfection option by DEC.

When designing UV disinfecting systems, both the Ten State Standards and TR-16 should be followed. Section 104 of *Ten State Standards* advises a case-by-case review, gives design, safety and operational guidelines, and requires the UV disinfection system maintain a minimum UV dosage of 30,000 microwatts -seconds/cm² while operating and after accounting for typical energy absorption losses. Section 8.3.4 of *TR-16* includes additional specifications for lamp design and performance, configuration and specifications for open channel units, and recommendations for monitoring and cleaning systems.

Additionally, specific design conditions will depend on the technology used and manufacturer's recommendations, and more importantly, the technology selected will depend on the specific disinfection objectives (target pathogens and concentrations [cfu/100 mL], etc.), water quality characteristics, the level and consistency of influent flow and treatment, and sampling requirements.

The main components of a UV disinfection system are mercury arc lamps, a reactor, and ballasts. In order to operate within SPDES permit limitations the UV disinfection system should consist of multiple banks of lamp modules that are capable of continuously disinfecting the peak flow with one bank out of service.

The UV unit must be protected from dust, excessive heat, and freezing temperatures. Adequate ventilation of heat-generating electrical components should be provided. A logbook of required maintenance should be kept. Smaller plants should maintain an inventory of spare lamps and module on-site. In small systems it may be just as effective to use a single bank of lamps (with spare lamps onsite) as to install a second bank of lamps as a backup unit. Once the operator is on site, replacing a lamp or module is just as effective and nearly as quick as switching power to the backup unit.

UV systems require power to operate, so the availability of standby power is a critical measure of their reliability. Some standby power source is necessary in nearly all cases.

System Validation

UV system technologies shall have independent third-party bioassay validation that covers the proposed range of design conditions in accordance with the USEPA Environmental Technology Verification (ETV) Program.

I.4. Effluent Reoxygenation

I.4.a. General

Effluent reoxygenation may be used when the level of dissolved oxygen in the effluent is required to be higher than is available from the process and/or when it is required that the chlorine level be reduced prior to discharge. *Reoxygenation* must be accomplished in its own unit and cannot be combined with the chlorination facilities. Effluent reoxygenation is similarly addressed in TR-16 Section 7.5 Effluent Reoxygenation, and briefly in the Ten State Standards Section 55 Plant Outfalls.

I.4.b. Diffused or Mechanical Aeration

A detention time of at least 30 minutes at peak flow shall be provided. Air shall be provided at a minimum of 20 SCFM for each 1000 gallon capacity in the unit.

Three types of aeration units may be used: floating aerator, fixed aerator, or diffused air. Mechanical aeration requires a shallow tank with a larger surface area. Diffused air allows the use of deeper rectangular tanks. When using diffused air, a side-roll is preferred over an end-roll effect.

The inlet must be raised sufficiently to allow for the expansion created by the addition of air. The outlet must be properly baffled to minimize the discharge of foam generated by aeration.

Cascade Aeration

Step or cascade aerators are especially useful when the needed dissolved oxygen increment is small or moderate. Cascades consist of a series of weirs or concrete or metal steps over which the effluent flows in a thin sheet. The edges of metal steps may have low weirs. The objective of a cascade is to maximize turbulence thus increasing oxygen transfer. As the number of descents increases for a given head loss, the quality of exposure may decrease because the tendency of droplets to break away from jets of falling water as soon as the jets strike air decreases.

The following equation may be used to obtain an estimate of aeration potential.

$$h = (r-1) / [0.11 a^b (1 + 0.046 T)]$$

Where,

h	=	Height through which water falls, feet
r	=	Deficit ration = $(C_s - C_o) / (C_s - C)$
C _s	=	DO saturation concentration of wastewater at temperature T, mg/l
C _o	=	DO concentration of influent to cascade, mg/l
C	=	Required DO level after aeration, mg/l
T	=	Water temperature, degrees Celsius
a	=	Water quality parameter, equal to 0.8 for wastewater treatment plant effluent
b	=	Weir geometry parameters:

The parameter b should be set equal to 1.0 for free weirs, 1.1 for concrete steps, and 1.3 for step weirs unless analysis of a particular design shows that individual characteristics justify use of a different value.

Head requirements generally vary from 3 to 10 feet, and effluent pumping may be necessary if the required head is not available. Normally less than 100 sq. feet of surface area is needed per MGD of capacity. Also refer to WEF's MOP 8: *Design of Municipal Wastewater Treatment Plants* (2010).

I.5. Outfalls

Plant outfalls shall be designed with a size and slope that will prevent surcharging and/or interference with preceding treatment processes when the receiving water is at its highest anticipated elevation, and should include diffusion facilities.

Diffusers should be:

1. Located in the streambed so as to be submerged at low flow,
2. Structurally protected against erosion, displacement, and sedimentation, and
3. Designed to mix effluent and receiving waters thoroughly.

Shoreline outfalls are not acceptable except in extreme situations. In such instances, dispersion of the discharged wastes must be established.

Access to a suitable effluent sampling point must be provided.

I.6. Controlled Release

General

Controlled release may be employed when the discharge is to a stream meeting the definition of an intermittent stream, and it is not economically feasible to treat to dry-stream standards. Prior to holding and discharge, a minimum of secondary treatment must be provided and the effluent must meet secondary limits when discharged. Also see TR-16 Section 6.5.2.1.4 Controlled Discharge.

The amount discharged, or rate of discharge, shall be governed by the stream –flow –to- waste-flow ratio. A controlled release program must always include the use of stream-flow gauging equipment.

Design Parameters

The holding lagoon must be constructed in accordance with standard lagoon criteria.

Minimum holding capacity is 200 days, with a recommended capacity of one year.

Proper baffling is required to prevent the discharge of floating algae and duckweed. It is recommended that the draw off be from the lower half of the liquid depth.

The valves used to allow discharge must be durable, protected from freezing, accessible, and capable of being secured to prevent discharge by unauthorized personnel.

The discharge pipe must be secured by a headwall with splash plate. Energy dissipation may be required to prevent erosion.

Stream Gauging

Discharge will be allowed only when the stream flow is in excess of 1 cfs. Mechanical or non-mechanical methods of stream gauging may be employed as long as their reliability is demonstrated in the engineering report.

J. Operation, Maintenance, and Control

J.1. Introduction

Operation and maintenance of treatment facilities is as important as proper design and construction. Therefore, where possible, when selecting among alternatives, the system requiring the least operation and maintenance should be given priority. Some operation and maintenance guidance/requirements are given in previous subsections of these Design Standards. O & M for the treatment system overall is given here.

Designs should include ample clearance to remove and replace units and parts. To insure proper operation and maintenance, an operating manual shall be provided with all treatment plants.

Contents of Operating Manual:

1. Approved Design Report
2. Hydraulic Profile
3. Basic Electrical Schematic
4. Basic Plant Piping Schematic
5. Unit operating Theories and Procedures
6. Recommended Operating Ranges
7. Maintenance Check List
8. Program for Residuals Dispersal
9. Parts List
10. Copies of All Warranties and Guarantees
11. Trouble-shooting Guide
12. Permit Requirements
13. Laboratory Requirements and Testing Schedule
14. Operator Requirements and Hours
15. Emergency or Breakdown Procedures

J.2. Systems Requiring a Certified Operator

A sewage treatment plant must be under the responsible supervision of an approved operator at all times. According to 6 NYCRR Part 650.1 (c), only the following systems found in these Design Standards are

not subject to this requirement: (1) septic tanks followed by subsurface leaching facilities with eventual discharge to groundwater, and (2) septic tanks followed by open or covered intermittent sand filters, with a design capacity of less than 50,000 gallons per day. Grades of operators needed at various types of plants are listed in Table J-1.

Table J-1. Necessary Operator Grades

Type	Plant Score	Operator Grade
All variations of activated sludge	30 points or less	1A
	31 – 55 points	2A
	56 – 75 points	3A
	76 points or greater	4A
All other treatment processes		
Sand Filtration		
Trickling Filter		
Rotating Biological Contactor		
Lagoons		
	30 points or less	1
	31 – 55 points	2
	56 – 75 points	3
	76 points or greater	4

6 NYCRR Part 650.3 contains the NYSDEC Wastewater Treatment Plant Facility Score Sheet Form. For a copy of the form and other information on the NYSDEC Wastewater Treatment Plant Operator Certification program see <http://www.dec.ny.gov/chemical/8707.html>.

J.3. Wastewater Treatment System Operation and Effluent Quality Control

6 NYCRR Part 750-2.8 Dispersal System Operation and Quality Control states the requirements of the SPDES permittee in regard to preventive and corrective maintenance, written O & M procedures, maintaining effluent quality, bypasses, and upsets.

J.4. Emergency Repair and Rehabilitation

6 NYCRR Part 750-2.7 Incident Reporting gives the requirements for “anticipated non-compliance”.

6 NYCRR Part 750-2.9, Additional Considerations Applicable to POTWs, gives the requirements for POTWs that exceed 95% of the design flow based on the required annual compliance certification report to be submitted to the department with the permittee’s February Discharge Monitoring Report [see subsections 2.9 (c)(1) and (c)(4)].

6 NYCRR Part 750-2.6 Special Reporting for Dischargers that are not POTWs gives the requirements for notifying the department regarding anticipated activities that may result in non-compliance, or for facility expansion.

J.5. Remote Telemetry, Instrumentation and Alarms

For cluster-developments, multi-home housing units, commercial and institutional, and small municipal facilities, design engineers should consider remote telemetry to better monitor treatment system alarm situations. Wireless, computer-based options are available to owners, operators and service providers to monitor the performance of pumps, timers, and event counters associated with mechanized treatment systems. EPA⁸ and the National Onsite Wastewater Recycling Association (NOWRA / <http://nowra.org>) websites and publications provide more information.

Alarms (onsite and remote) should be considered to alert homeowners and service providers that system malfunction might be occurring. In addition to simple float alarms, several manufacturers have developed custom-built control systems that can program and schedule treatment process events, remotely monitor system operation, and notify technicians by pager or the Internet of possible problems.

J.6. System Additives

EPA’s 2002 Onsite Wastewater Treatment Systems Manual⁹ includes several fact sheets that apply to single and multi-family homes, and package plants or proprietary treatment units applicable for cluster developments and small commercial or institutional establishments. The “Septic Tank Additives” Fact

⁸ 2002 EPA Onsite Wastewater Treatment System Manual, (EPA/625/R-00/008, February 2002)

⁹ Ibid, “Special Issues Fact Sheet 1”.

Sheet describes the relevant aspects of septic tank operation, warns of potential harm from indiscriminant use of additives, and details some of the additive options available for sale.

The three general types of commonly marketed septic system additives are:

Inorganic compounds, usually strong acids or alkalis, are promoted for their ability to open clogged drains. Product ingredients (e.g., sulfuric acid, lye) are similar to those used in popular commercial drain cleaners. These products can adversely affect biological decomposition processes in the treatment system and cause structural damage to pipes, septic tanks, and other treatment system components. Hydrogen peroxide, once promoted as an infiltration field reconditioner, has been found to actually degrade soil structure and compromise long-term viability of soil treatment potential. Its use to unclog failed infiltration fields is no longer recommended.

Organic solvents, often chlorinated hydrocarbons (e.g., methylene chloride, trichloroethylene) commonly used as degreasers and marketed for their ability to break down oils and grease. Organic solvents represent significant risks to ground water and wastewater treatment processes. These products can destroy resident populations of decomposer and other helpful microorganisms in the treatment system. Use of products containing organic solvents in onsite treatment systems is banned in many states. Introduction of organic solvents into onsite systems located in states that ban the use of these products may trigger liability issues if ground water becomes contaminated.

Biological additives, like bacteria and extracellular enzymes mixed with surfactants or nutrient solutions, which mirror but do not appear to significantly enhance normal biological decomposition processes in the septic tank. Some biological additives have been found to degrade or dissipate septic tank scum and sludge. However, whether this relatively minor benefit is derived without compromising long-term viability of the soil infiltration system has not been demonstrated conclusively. Some studies suggest that material degraded by additives in the tank contributes to increased loadings of BOD, TSS, and other contaminants in the otherwise clarified septic tank effluent.

Other products containing formaldehyde, paraformaldehyde, quaternary ammonia, and zinc sulfate are advertised to control septic odors by killing bacteria. This objective, however, runs counter to the purpose and function of septic tanks (promoting anaerobic bacterial growth). If odor is a problem, the source should be investigated because sewage may be surfacing, a line might have ruptured, or another system problem might be present.

Another variety of consumer products is marketed for their ability to remove phosphorus from wastewater. These products are targeted at watershed residents who are experiencing eutrophication problems in nearby lakes and streams. Phosphorus is an essential nutrient for aquatic plant growth and limiting its input to inland surface waters can help curtail nuisance algae blooms. Aluminum (as alum, sodium aluminate, aluminum chloride, and activated alumina), ferric iron (as ferric chloride and ferric sulfate), ferrous iron (as ferrous sulfate and ferrous chloride), and calcium (as lime) have been proven to be effective in stripping phosphorus from effluent and settling it to the bottom of the tank. An important side effect of this form of treatment, however, can be the destruction of the microbial population in the septic tank due to loss of buffering capacity and a subsequent drop in pH. Treatment processes can be severely compromised under this scenario.

Finally, baking soda and other flocculants are marketed as products that lower the concentration of suspended solids in septic tank effluent. Theoretically, flocculation and settling of suspended solids would result in cleaner effluent discharges to the subsurface wastewater infiltration system. However, research has not conclusively demonstrated significant success in this regard.

J.7. Residuals Hauling and Disposal (6 NYCRR Parts 364 and 360)

Residual solids from wastewater treatment facilities may contain pathogenic organisms, nutrients, and oxygen-demanding materials. Proper handling and dispersal is necessary to protect public health and prevent degradation of groundwater and surface water quality.

Part 750-2.8(d) requires that septic tanks be inspected annually and septage (solids and scum) pumped out when the combined sludge and scum layers equal 25% of the tank volume. For STEP or STEG systems, residential septic tanks may have to be pumped out every 2 to 5 years. The bottom of the scum layer should always be three inches or more above the bottom of the outlet device.

The septage from community septic tanks, aerobic biological treatment, or physical-chemical treatment should be removed periodically by a professional hauler. All haulers of sewage sludge must have a valid Part 364 permit. At aerobic or physical-chemical treatment facilities storage should be provided for twice the volume of sludge to be generated between anticipated removal dates. Storage may be under either aerobic or anaerobic conditions, and provision should be included to control any possible odors. The sludge can be hauled to a larger sewage treatment plant, or disposed of in an approved manner such as land spreading, land filling, or composting.

If the sewage sludge is to be disposed of in a sanitary landfill the sludge must be dewatered to at least 20 percent solids, and the sanitary landfill must be approved by the NYSDEC. A Part 360 permit is necessary for land spreading or composting of the sewage sludge. Prior to 2003 there was an exemption for treatment plants smaller than 0.1 mgd; the exemption was eliminated and all land spreading or composting of sewage sludge now requires a permit. The web link <http://www.dec.ny.gov/chemical/8797.html> provides more information on land application of organic waste. For biosolids recycling, the same NYSDEC web page has fact sheets containing more information, definitions of terms, and references.

Information as to qualification and certification, along with the necessary application form, can be secured from any Department of Environmental Conservation regional office. The current list of Part 360 Permitted Land Application Facilities can be found at <http://www.dec.ny.gov/chemical/55420.html>.

For open sand filter treatment facilities, the scum or solids mat should be raked off the filter at least every six months. Dispersal of this sludge must also be in an approved manner as described above.

Appendix A Wastewater Treatment System Regulatory Framework in New York State

This table is meant as general statewide guidance, questions regarding applicability of DEC, local health department (LHD) or DOH District Office policy should be referred to the appropriate DEC, LHD or DOH District Office.

System Type	Flow Range (gpd)	Jurisdiction	Technical Standard ¹	SPDES Permit Required?
Residential with subsurface discharge.	≤ 1,000	local health department (LHD), DOH District Office, or local code enforcement official (CEO) ¹	❖ Appendix 75-A ❖ Local sanitary codes	No
Residential with subsurface discharge.	> 1,000 & < 10,000	DEC	❖ Intermediate Design Standards	Yes, may qualify for SPDES General Permit GP 0-05-001.

Existing Residential applying for a <u>surface discharge</u> .	Any	Consult DEC Regional Office (refer to TOGS 1.2.4 for flows less than 1,000 gpd).	❖ Appendix 75-A ❖ Intermediate Design Standards	Yes, may qualify for an individual SPDES Permit.
Private, Commercial and Institutional sanitary sewage (non-industrial) with subsurface or surface discharge under DOH permit. ²	< 10,000	DOH/LHD DEC (SPDES) ²	❖ Appendix 75-A ❖ Intermediate Design Standards and ❖ 10 State Standards (surface discharges only)	Yes, <u>if</u> subsurface systems are > 1000 gpd; may qualify for GP 0-05-001. All surface discharges require an individual SPDES permit.
Private, Commercial and Institutional sanitary sewage (non-industrial) with surface or subsurface discharge NOT under DOH permit.	< 10,000	If < 1,000 gpd; Design Professional, LHD, or CEO¹; If $\geq$ 1,000 and < 10,000 gpd: DEC	❖ Appendix 75-A ❖ Local sanitary codes ❖ Intermediate Design Standards	Not if subsurface discharge is < 1000 gpd. Otherwise, yes, <u>and</u> all surface discharges require an individual SPDES permit.

Private, Commercial and Institutional sanitary sewage (non-industrial) <i>with surface or subsurface discharge.</i>	$\geq 10,000$	DEC	❖ Intermediate Design Standards ❖ 10 States Standards (surface discharges only)	Yes
Any System discharging sanitary sewage with admixture of industrial wastes or other wastes, to <i>surface or subsurface waters.</i>	Any	DEC	❖ Multiple references identified in 6NYCRR Part 750 Section 1.24 ❖ 10 State Standards (surface discharges only)	Yes

¹ Under New York State Education Law, all wastewater treatment systems are to be designed by or under the supervision of a design professional. Rules and regulations of local watersheds, the New York City Department of Environmental Protection, or the Adirondack Park Agency may require additional design requirements beyond the minimum state standards (Appendix 75-A or Intermediate Design Standards). Also see DOH Fact Sheet “Need for Licensed Design Professionals – Residential Onsite Wastewater Treatment Systems” available from local health departments (LHDs) or DOH District Offices. Specific office location information can be seen at http://www.health.state.ny.us/environmental/water/drinking/doh_pub_contacts_map.htm.

² Pursuant to the 1984 NYSDEC - NYSDOH MOU, DOH shall be responsible for approval and regulatory activities regarding all new on-site [subsurface discharge] sewage treatment and dispersal systems with a design flow of 1,000 gpd or less from a residential dwelling which does not have the admixture of industrial wastes or other wastes as defined in Section 17-0701 of the Environmental Conservation Law of the State of New York. This also includes responsibility for plan approval and regulatory actions except SPDES permit issuance activities for all surface and sub-surface discharges of 10,000 gpd or less, and designated by DEC as non-significant at facilities permitted by DOH under parts 6 (Public Pools and Beaches), 7 (Temporary Residences), 14 (Food Service), 15 (Migrant Labor Camps) and 17 (Mobile Home Parks) of the State Sanitary Code except where written agreements are entered into by DEC with a Federal, State or local governmental agency to perform this responsibility.

Appendix B**Conversion Factors**

MULTIPLY	BY	TO OBTAIN
Acres	43560	Square Feet
Atmospheres	33.9	Feet of Water
Centimeters	0.3937	Inches
Cubic Feet	7.48052	Gallons
Cubic Feet	28.32	Liters
Cubic Feet/Second	449	Gallons/Minute
Cubic Meters	35.31	Cubic Feet
Cubic Meters	264.2	Gallons
Cubic Meters	10 ³	Liters
Cubic Yards	27	Cubic Feet
Cubic Yards	202	Gallons
Feet	30.48	Centimeters
Feet	0.3048	Meters
Feet of Water	62.43	Pounds/Square Foot
Feet of Water	0.434	Pounds/Square Inch
Gallons	3785	Cubic Centimeters
Gallons	0.1337	Cubic Feet
Gallons	3.785	Liters
Gallons water	8.3453	Pounds of Water
Gallons/Minute	2.228 x 10 ⁻³	Cubic feet/Second
Gallons/Minute	1440	Gallons/Day
Gallons/Minute	0.06308	Liters/Second
Gallons/Day	6.944 x 10 ⁻⁴	Gallons/Minute
Gallons/Day/Square Foot	1.604	Inches/Day
Grams	2.205 x 10 ⁻³	Pounds
Grams/Liter	1000	Parts/Million
Hectares	2.471	Acres
Horsepower	33,000	Foot-pounds/Minute
Horsepower	0.7457	Kilowatts
Inches	2.54	Centimeters
Inches/Day	0.6234	Gallons/Day/SquareFoot
Kilograms	2.205	Pounds
Kilowatts	1.341	Horsepower
Kilowatts-hours	2.655 x 10 ⁶	Foot-pounds
Liters	103	Cubic Centimeters
Liters	0.03531	Cubic Feet
Liters	0.2642	Gallons
Meters	3.281	Feet
Milligrams/Liters	1	Parts/Million
Million Gallons/Day	1.54723	Cubic Feet/Second
Parts/Million	8.345	Pounds/Million Gallons
Pounds	453.5024	Grams
Pounds of Water	0.1198	Gallons
Pounds/Square Inch	2.31	Feet of Water
Pounds/Square Inch	2.036	Inches of Mercury
Temperature (°C) + 17.78	1.8	Temperature (°F)
Temp (°F) - 32	5/9	Temp. (°C)

Appendix C Sewer And Manhole Leakage Tests

Low-Pressure Air And Vacuum Testing

The proper procedure for air testing of sanitary sewers is described in ASTM C 828 for clay pipe, ASTM C 924 for concrete pipe, and ASTM F 1417 for plastic pipe. The document title specific to vitrified clay pipe, sizes 4 to 12 inches, is a general procedure and may be used for any other sanitary sewer pipe material and is not limited to a maximum diameter of 12 inches. The parameter to be measured is the rate of air loss based on an average test pressure of 3.0 psig above any backpressure due to any groundwater that may be over the pipe.

It is extremely important that the various test plugs be properly installed and braced to prevent blowouts. It is also important to maintain adequate pressure relief valves to prevent over-pressurizing the system. A maximum relief pressure of 10 psi is suggested in most literature.

Although line testing may be done at any time during the construction phase, there are two time periods when testing is of special value - (1) prior to placement of paving materials, in order to avoid unnecessary expense in locating and repairing leaks, and (2) at as late a date as possible, after work has been completed and some settlement has had a chance to occur. This latter period is the appropriate time for the final line acceptance test, since significant damage can occur after backfill from subsequent settling.

All portions of a new sewerage system should be tested, including any building sewers that may be constructed in conjunction with the main lines.

Air testing for concrete sewer manholes, shall conform to either the test procedures described in ASTM C 1244 - "Standard Test Method for Concrete Sewer Manholes by the Negative Air Pressure (Vacuum) Test Prior to Backfill," or the vacuum testing specifications given in TR-16 Section 2.4.9. Manholes which cannot be properly air (vacuum) tested by the ASTM or TR-16 procedure should be visually inspected and leakage tested using internal or external hydrostatic pressure.

Hydrostatic Testing

All conventional gravity sewers, manholes, and cleanouts shall be tested by any standard method after being flushed and before being used. One procedure for hydrostatic testing of sanitary sewers is described in "AWWA C-600, Section 4, Hydrostatic Testing". Depending upon the groundwater table elevation, either an infiltration or exfiltration method may be used. The maximum rate of infiltration/exfiltration

shall not exceed 100 gallons per inch diameter per mile per day, under a minimum positive head of 2 feet as given in Section 33.94 of the Ten State Standards.

Manholes constructed to be water-tight according to Section 34.6 of the Ten State Standards shall also be tested for damage and water-tightness as detailed in TR-16 Section 2.4.9.

For STEP systems, service line testing can be accomplished with an air-compressor to bring the line to its test pressure; the test is a success if the pressure holds for 60 seconds or more. See AWWA for allowable leakage rate. When the service line can be filled with water from the tank test, particularly if the service line is short and doesn't require a large volume to fill it, a small hand pump with pressure gauge can be employed for the pressure test.

Introduction

There are many reasons to ensure that all septic tanks are watertight. Leakage from the tank releases minimally treated sewage into subsurface soils and/or groundwater. Sewage injected deeply in the soil profile is much less likely to be adequately treated as it moves down through the soil. In areas of relatively shallow water tables or where tanks are located in low areas, groundwater or surface water can leak into the tank. Inflow of groundwater can disrupt settling, treatment, and storage of solids (i.e., the important functions of the tank) as well as the function of downstream components of the wastewater treatment system. Possible locations on a septic tank where leakage can occur include:

- Weep holes at the base of the tank. (Weep holes are used in some pre-cast concrete tanks to release forms from tanks and to prevent collection of rainwater during storage prior to installation. If used, these should be sealed appropriately prior to installation.)
- Mid-seam joint.
- Inlet/outlet pipe penetrations.
- Top-seam joint.
- Tank top/access riser joint.
- Access riser/lid joint.
- Any damaged, improperly formed location or area where material is too thin.

New tanks can be tested for watertightness by filling with water (hydrostatic testing) or by vacuum testing. In both cases, the tank should be tested in the ready-to-use state. Inlets and outlets should be plumbed with the appropriate pipes, which can then be plugged for the test.

Hydrostatic Testing

Be careful when performing hydrostatic tests on plastic and fiberglass tanks as they gather much of their strength from the soil support. For all mid-seam tanks, keep the backfill near the mid-seam, but leave the seam itself exposed to monitor the test.

The following is a suggested water testing procedure for tanks. Note that this test does not evaluate the tank's ability to withstand external pressures; that issue must be ensured through adequate engineering design.

1. Plug the inlet and outlet pipes with a watertight plug, pipe and cap, or other seal. Seal the pipes away from the tank to test any pipe connections that may be of concern.
2. If testing a mid-seam tank, ensure that the seam is exposed for the water test.
3. Fill the tank to the top.
4. If the tank has a riser, add water into the riser to a maximum of 2 inches above the tank/riser seam. Care must be taken not to overfill as the top section of a two-piece tank may become buoyant.
5. Measure and record the water level.
6. Wait 24 hours. Any obvious leakage during this time should be evaluated and remedied by the application of a suitable sealing compound.
7. If the test reveals leaks that cannot be repaired, the tank is considered unacceptable.
8. Refill concrete tanks to original level after 24 hours as they will absorb some water.
9. Check again after 24 hours. If less than 1 gallon is lost in a concrete tank, the leak test is considered acceptable.

When performing hydrostatic testing in cold climates, there are a few important points to consider. First, water is its densest at about 4 degrees C (just above freezing); water put into a tank at 10 to 20 degrees C (typical of groundwater) and left in the tank overnight at freezing temperatures will drop the level in the tank a substantial amount (about 2 percent or 3 gallons in a 1500-gallon tank). A “loss” of 3 gallons in the risers will look like a leak. Additionally, water used in the test will freeze and expand by approximately 9 percent. If the site is not occupied quickly, the tank may crack as a result of the test itself, assuming the water is left in the tank following the test.

Vacuum Testing

Vacuum testing requires less time than hydrostatic testing and can be performed without having water available on the site. Testing should be done on the tank in its ready-to-use state (*i.e.*, pipes in the inlet and outlet, risers with lids.) In the test, all pipe penetrations, manholes, and risers are sealed airtight, and a special insert is sealed on one of the tank manholes. Using a pump, air is evacuated through this insert to a standard vacuum level, and the reading on a vacuum gage is recorded. Local codes, ASTM Standard C-1227, or the National Precast Concrete Association (NPCA) standard can be used to determine the target vacuum for the size, and type of tank (*i.e.*, concrete, plastic and fiberglass have different compressive strengths). Be careful not to exceed the recommended vacuum level for a tank as recommended by the manufacturer. It is possible to damage or implode a tank.

Specifically, as of August 2003, the NPCA standard states: “The recommended [vacuum test] procedure is to introduce a vacuum of 4 inches of mercury. Hold this pressure for 5 minutes. During this initial 5 minutes, there is an allowable pressure equalization loss of up to half-inch of mercury. If the pressure drops, it must be brought back to 4 inches and held for a further 5 minutes with no pressure drop.”

If a tank will not hold the vacuum, leaks must be located and repaired. The test can then be repeated. If the tank cannot be repaired and rendered watertight, it should be replaced. Note that vacuum testing of concrete tanks draws seams together for a positive mastic seal, assuming there are no other problems. With any tank, collapse, deflection, deformation, or cracking indicate a poor quality tank. It is important to test the entire system: tank, pipe sleeves, risers, inspection ports, and lids.

Testing Existing Tanks

It is more difficult to check watertightness in an existing septic tank. Adequate testing requires a period of several hours to a day or more without inflow to the tank and sealing off inlet and outlet pipes. Seal the line at the distribution box (or other appropriate place in the case of secondary treatment units) and at the clean-out between the building and the tank. Apply vacuum or water as desired. If there are no leaks, the entire system passes in one step. If there are leaks, successive tests will locate the source or sources. Although actual testing of existing tanks may be impractical, much can be discerned by a thorough inspection of a tank both before and after it has been pumped out. Most tanks built using older methods of construction (such as built-in-place block or brick tanks) are typically not watertight, nor structurally sound, and probably cannot be reasonably repaired. In some cases it may be possible to do more to check existing tanks. If the soil around the tank is saturated, the tank contents can be pumped down and observations made over the next few hours to detect leakage into the tank around pipe penetrations, seams, or through breaks in the tank. Caution should be exercised, however, as high groundwater may cause empty tanks to become buoyant and float out of the ground. Alternatively, excessive soil pressure may collapse a tank. In some cases, it may be necessary to excavate completely around the tank to make a visual inspection for leaks. If there is any doubt about the integrity of the existing tank, it should be replaced.

CIDWT. 2008. Installation of Wastewater Treatment Systems. Appendix G. Septic Tank Watertightness Testing. Consortium of Institutes for Decentralized Wastewater Treatment. National Review Version1. June 20, 2008. www.onsiteconsortium.org.

Appendix E Recommended Guidance for the Design of Wastewater Drip Dispersal Systems

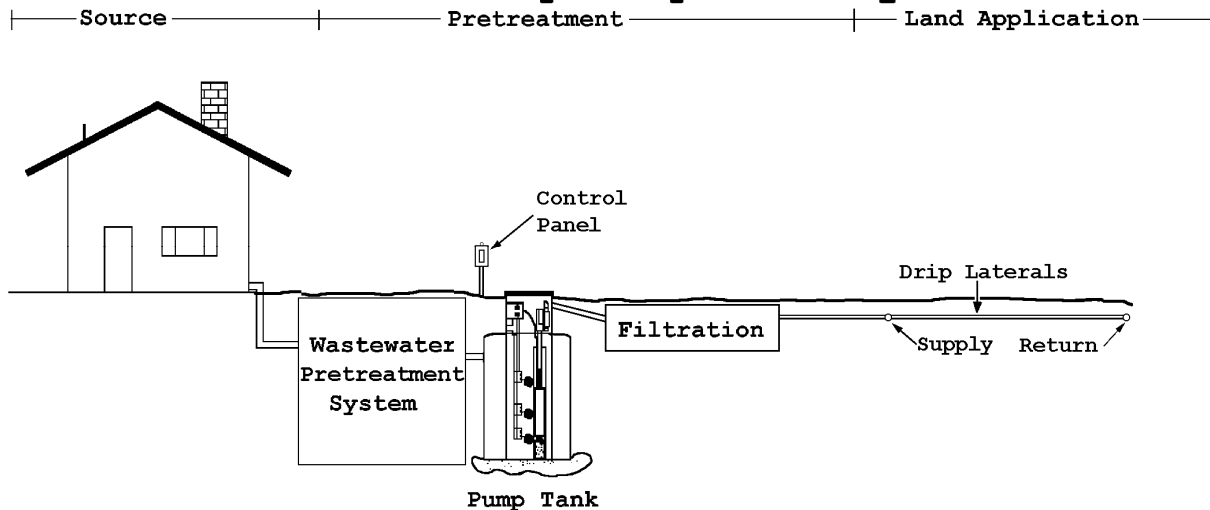
Appendix E.1. Scope

1.1 Drip dispersal is a method used to distribute wastewater that has received at least primary treatment over an area of land for final polishing, reuse, or recharge of groundwater. This method of dispersal is capable of uniformly distributing the wastewater effluent over large areas. It has been used in the U.S. for dispersal of preconditioned wastewater onto soil infiltrative surfaces since the late 1980s.

Drip dispersal is frequently, but inappropriately, referred to as drip irrigation. Drip dispersal is seldom designed to meet the agronomic water requirements of a crop. Instead, it is usually designed to maximize infiltration of water into the soil throughout the year. Some of the dispersed water will evaporate, or be transpired by vegetation during the growing season, but most will percolate into the soil and recharge the underlying groundwater. However, plant irrigation or other water reuse applications can be incorporated into the design.

1.2 For purposes of these standards, drip dispersal consists of a dripper-line, or low-profile conduit, installed for the purpose of treating and uniformly distributing preconditioned wastewater into the shallow soil horizon. These systems rely on electronic controllers to manage the frequency of the wastewater doses, preventing the hydraulic overloading of soil and also providing for subsequent re-aeration of the soil adjacent to the conduit. The typical dripper line or low-profile conduit is $\frac{3}{4}$ to 2 inches high and designed to fully infiltrate the water into the soil before the next dose. Drip dispersal systems do not provide for internal storage of wastewater; consequently, it is utilized in conjunction with other flow equalization devices.

Subsurface Drip Dispersal System



Drip-Dispersal Diagram (from www.onsiteconsortium.org/graphics)

1.3 This guidance describes the appropriate design, installation, operation, monitoring, and maintenance practices that are necessary to ensure the long-term performance of drip, or low-profile, dispersal systems.

1.4 Site-specific engineered designs must be used. The owner may choose to specify a “pre-engineered” package that is appropriate for the site requirements; however, using a pre-engineered package does not preclude the need for proper site-specific design.

Appendix E.2. Reference Documents

2.1 USEPA. Onsite Wastewater Treatment Systems Design Manual, EPA/625/R-00/008. Office of Water, Office of Research and Development. Washington, D.C. February 2002.

2.2 National Onsite Wastewater Recycling Association (NOWRA). Subsurface Drip Dispersal Systems Workshop Manual. National Onsite Wastewater Recycling Association, Edgewater, Maryland. 2001.

2.3 NOWRA-Recommended Guidance for the Design of Wastewater Drip Dispersal Systems. Approved and adopted by the NOWRA Board of Directors, March 23, 2006.

2.4 CIDWT. Residential Onsite Wastewater Treatment Systems: An Operation and Maintenance Service Provider Program. Consortium of Institutes for Decentralized Wastewater Treatment (CIDWT), www.onsiteconsortium.org, 2005.

2.5 American Society for Testing and Materials (ASTM), West Conshohocken, Pennsylvania.

2.5.1 ASTM D5925-96. Standard Practice for Preliminary Sizing and Delineation of Soil Adsorption Field for On-Site Septic Systems. 1996.

2.5.2 ASTM D5879-95. Standard Practice for Surface Characterization for On-Site Systems. 1996.

2.5.3 ASTM D5921-96. Standard Practice for Subsurface Site Characterization of Test Pits for On-Site Septic Systems. 1996.

2.6 Texas Cooperative Extension, The Texas A&M University System. Drip Dispersal Systems for Land Application of Effluents.

2.7 EPRI & TVA, Wastewater Subsurface Drip Distribution, Peer Reviewed Guidelines for Design, Operation, and Maintenance. Report #1007406, Revised Report, September 2004, Electric Power Research Institute and Tennessee Valley Authority, co-sponsors. Electric Power Research Institute (EPRI), Concord, California.

Appendix.E.3. Design

3.1 A demand analysis of water use at the building(s) to be served should be conducted to estimate the average daily flow, expected daily peak flows and diurnal and weekly variations. Local codes usually will dictate unit values to estimate the design flows for wastewater systems; determination of occupancy load in NYS is per the Uniform codes. The design flow estimates obtained from using these unit values typically will represent maximum peak flows. While dispersal systems must be designed to distribute the maximum expected peak flows, drip dispersal systems are usually designed to distribute the average daily flow with peak flows controlled by flow equalization.

3.2 The dose or pump tank should provide sufficient storage for equalization of peak flows. The storage volume is calculated to hold any peak flows that are expected to routinely occur over a given period, typically a day or week depending on the expected flow variations. The pump tank should provide at least one quarter of a day's storage above the alarm-level, and more storage is better. Equalization storage from one-half to one day between the pump-enable water level and the alarm water level is necessary for small flow systems. Local regulations may require a specific storage capacity. The design may increase or

decrease this storage based on available redundancy of facilities.

3.3 Drip distribution may be used with a wide range of preconditioning processes and water quality. Preconditioning, water quality and quantity should be evaluated when selecting soil loading rates and mechanical equipment.

3.4 The layout of the dispersal system piping network must provide reasonably uniform distribution over the proposed soil treatment area. The hydraulic design should achieve discharge rates and volumes that vary no more than 10% between all the emitters within a zone during a complete dosing event. Consideration should be given to any unequal distribution during flow pressurizing and flow depressurizing periods. The designer must be able to mathematically support the design for equal distribution and demonstrate it upon installation. The design of the soil treatment area (sizing, depth, geometry, and orientation), must be based on the vertical hydraulic application rates given in Table E-1 in Section E.2 of the NYS DEC Design Standards for Intermediate-sized Wastewater Treatment Systems.

3.4.1 Manufacturers must rate all valves, pressure regulators, fittings, and piping for wastewater application. The system designer must evaluate the compatibility of these appurtenances for the specific application.

3.4.2 Drip field piping layout must provide a sufficient number and density of emitters, or other flow distribution devices, to achieve reasonably uniform distribution and application of the preconditioned wastewater over the entire soil treatment area. The number of emitters must be sufficient to maintain an instantaneous loading rate (gallons per dose) that will maximize use of the hydraulic and treatment capacities of the soil and prevent breakout of wastewater on the treatment area surface during dosing.

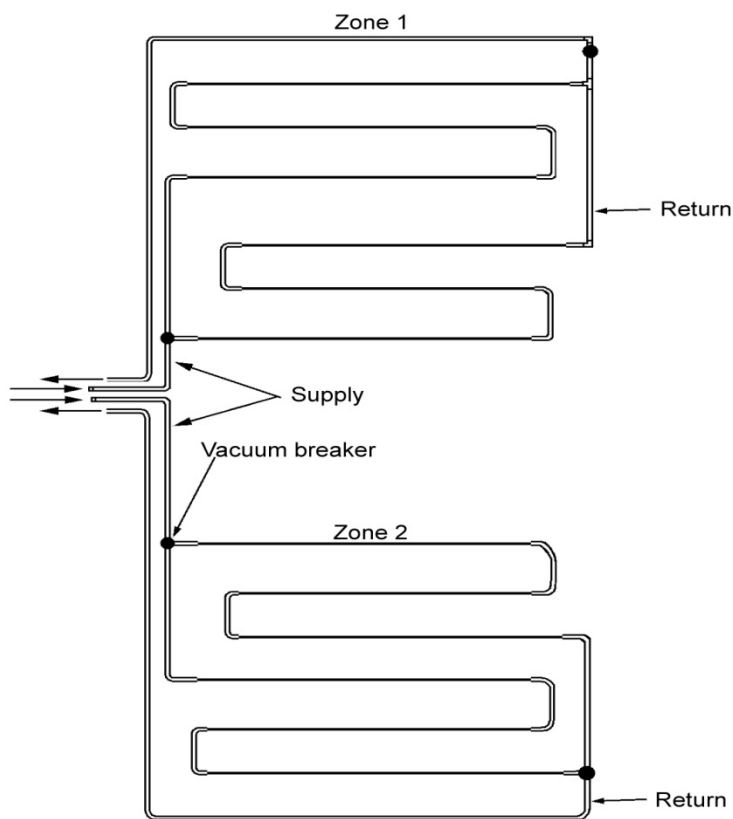
3.4.3 Emitter and dripper-line spacing should be based on the permeability of the soil. Horizontal movement of water in coarse-textured soils with high permeability is much less than it is in fine-textured soils, which can draw water several feet, so the horizontal spacing of the dripper-line should be adjusted accordingly to avoid exceeding the instantaneous hydraulic capacity of the infiltrative surface. A typical range of spacing from 1 to 4 feet between emitters is sufficient.

3.4.4 Minimum and maximum installation depths of the dripper-line may be established by local rules based on soil characteristics and separation distances. Installation depths typically range from 6 to 12 inches. Recommendations of the manufacturer of up to 18 inches should be

considered depending on site-specific issues. Shallow drip dispersal systems must be protected from possible physical damage.

3.4.5 Drain-down, which occurs after each dose as the dripper-line depressurizes, must be managed to prevent localized overloading through the lower laterals of the network. Dripper-line should be placed on contour and laid out to drain itself through the emitters as evenly as possible so as not to cause localized overloading.

3.4.6 Dispersal systems are often divided into zones that can be loaded independently. This is done to better adapt the dispersal of wastewater to the capacity of the receiving environment and to meet the hydraulic requirements for equal distribution, field flushing of the dripper-line, reduce localized overloading from the drain-down prior to and after pressurization of dripper-line. Multiple zones also can provide standby capacity for equipment servicing and system repairs.



Drip-Dispersal Diagram (from www.onsiteconsortium.org/graphics)

3.4.7 Lateral lengths within a zone should be close to equal to achieve efficient flushing of each of the laterals. To determine the suitable flushing flow and pressure requirements at the proximal end necessary to achieve the flushing velocity at the distal end, the designer should obtain dripper-line head-loss information relating dripper-line diameter, emitter spacing, and emitter and flushing flow rates to lateral lengths. Computer programs are available to aid in evaluating the hydraulic design of the dispersal system.

3.4.8 Drip dispersal systems should be designed to operate in the manufacturer's specific pressure range for emitter operation. The dripper-line should be placed within appropriate elevation tolerance limits in each zone to maintain equal distribution within the preferred range. It may be necessary to control the inlet pressure with a pressure-regulating valve in order to control emitter flow rate. Hydraulic analyses should be performed to ensure appropriate pressure and flow is achieved for both dosing and flushing conditions.

3.4.9 Air/vacuum release valves must be installed at the high points in each zone to provide a vacuum break as the dripper-line drains after a dose event. Recommendations of the manufacturer for other vacuum-breaking devices or methods should be considered depending on the proprietary design. Breaking the vacuum is critical to prevent aspiration of soil particulates back into the dripper-line through the emitter.

3.5 The piping layout is typically flushed. Flushing velocities should meet or exceed the recommendations of the manufacturer of the dripper-line used. When a dedicated scour cycle is present, flushed materials should be returned to the head-works of the wastewater treatment system.

3.6 The pump must be designed to handle preconditioned wastewater and to manage all hydraulic operations required for the system. The dosing capacity must be sufficient to apply a full dose at the design rate for the largest zone in the system and meet the flushing rate requirements. If automatic particle separators are used, the pump must also be capable of achieving the back-flushing or washing rate and pressure requirements of the manufacturer of the separator.

3.7 Particle separation is required to reduce the size of suspended particles in the wastewater effluent to prevent emitter plugging. Separators should follow the manufacturer's recommendations and be suitable for wastewater applications. They should be accessible for maintenance and designed to match the maintenance frequency of the system.

3.8 Monitoring Devices: A method for measuring the volume of wastewater dispersed against elapsed time should be provided. Also, means to measure flow rates and operating pressures are beneficial to diagnose hydraulic problems. Continuous data recording should be considered.

3.9 Controls: An integrated controller is necessary to manage the multifunction processes of drip dispersal systems. The control panel shall be located in an accessible location where an operator can monitor and perform diagnostics on the system. Manual override switches for all automated mechanical functions should be provided. H-O-A (Hand-Off-Auto) switches for manual operation of pump and valves should be provided for an operational interface. Visual indications of specific operations are recommended.

3.9.1 The controller must manage the dosing, aeration and resting cycles to the drip field as designed. The run times and the rest times should be adjustable to manage the instantaneous loading rates to regulate the demand with the field capacity.

3.9.2 Each major component should be located to perform properly and to be accessible for operation and maintenance.

3.10 All components of the drip arrangement must work together for the successful, long-term, reliable operation of a drip dispersal system. Each function of the system design, regarding flow rates and pressures, should be appropriately integrated and designed to meet requirements. All components in a drip dispersal system should be rated to withstand contact with wastewater and recommended for this application by the manufacturer or supplier. Additional components may be used as deemed appropriate by the manufacturer or designer to treat and evenly disperse the wastewater to prevent emitter- or soil-clogging, prevent physical damage, monitor operation, or otherwise enhance system performance.

Appendix E.4. Installation

4.1 Only trained and otherwise qualified contractors shall install drip dispersal systems.

4.2 The installer must pay particular attention to site protection and protection of the dripper-line (e.g.: freezing). Installation practices should provide site protection for shallow soil installations.

4.3 The installation of the dispersal system should reasonably follow the designer's plans.

4.4 During installation, the dripper-line should be protected against entry of construction debris and soil

materials by taping or otherwise tightly covering the ends until their connections to the manifolds are made.

4.5 All aspects of the design objectives should be tested, proven, and recorded at startup to confirm that site-specific design objectives are met.

Appendix.E.5. Operation, Monitoring, And Maintenance

5.1 Drip dispersal systems should be designed such that system operation can be monitored for proper usage and performance. The monitoring frequency should be based on the most limiting process in the system.

5.2 Monitoring of flow rates and pressures is necessary to diagnose possible overuse and ensuing system damage.

5.3 All aspects of the design objectives should be monitored, proven, and recorded at regular intervals to confirm that site-specific design objectives are met.

5.4 Operational monitoring should determine if wastewater has been or is surfacing as a result of the operation of the drip system and that the system is in good repair.

5.5 Only trained and otherwise qualified operators or installers should operate and service drip dispersal systems.

5.6 For more complete guidance, see reference documents 2.2, 2.3, 2.4 and 2.7 above.

Glossary:

Blackwater: Wastewater from toilets (i.e., flush water carrying human excreta), urinals, or kitchen sinks that include integral garbage disposal units.

Certified Tank: A certified tank is a tank (of any material) that is supplied with a permanent identification and tracking method, stating total compliance with cited specifications (structural & water-tightness) and documenting that the testing of tanks (i.e. factory vacuum testing), and tank design have both passed a minimum annual inspection by a licensed professional engineer; documentation is required. (Note: the project engineer must also assure that the OWTS system, from the building sewer to the disposal point – including the septic tank and grease interceptor- is water-tight prior to backfilling.)

Conventional Septic System: A septic tank followed by gravity flow to a subsurface soil-based treatment system.

Decentralized Wastewater System: A wastewater treatment system used for collection, treatment, dispersal or reuse of wastewater from individual homes, clusters of homes, isolated communities, industries, or institutional facilities, at or near the point of waste generation (CIDWT Glossary, 2007).

Design Engineer: a person or firm licensed to practice professional engineering in NYS.

Enhanced Treatment Unit: A treatment unit that provides secondary or tertiary treatment of wastewater.

Flood-prone areas: those areas identified as the one percent annual chance floodplain on FEMA's Flood Insurance Rate Map.

FOG: Animal fats, vegetable oils and petroleum greases.

Gravity Grease Interceptor (Type II): An exterior vessel (pre-cast concrete, HDPE plastic, or fiberglass-reinforced polyester [FRP]) used to separate by gravity, retain for disposal, and minimize short-circuiting and turbulence to facilitate FOG separation.

Graywater: Wastewater discharged from clothes washers, bathtubs, showers, dishwashers, and sinks

(including kitchen sinks without garbage disposal units), but excluding “blackwater,” fats, oils and greases (FOG), excessive solids (i.e. no garbage grinders) and industrial wastewater containing toxic or hazardous materials .

Hydro-mechanical Grease Interceptor (Type I): (“Grease Trap” is a discontinued term as of 2007 national codes, and 2010 NYS codes.) An interior, under-sink, hydro-mechanical device or vessel used to help facilitate grease separation, collection and disposal. The brown grease collected from these devices cannot be recycled or reused for human or animal consumption or contact.

Responsible Management Entity (RME): A legal entity that has the managerial, financial, and technical capacity to ensure the long-term cost-effective operation of onsite or cluster wastewater treatment systems in accordance with applicable regulations and performance requirements (e.g.: a wastewater utility or wastewater management district).

Reviewing Engineer: an employee of NYSDEC, NYSDOH, a County or City Health Department, and licensed to practice professional engineering in NYS.

Significant Delivery Period: A number of hours when a facility produces wastewater several times the rate of the average daily flow. As the delivery period for the daily design flow gets shorter, the receiving septic tank volume shall increase proportionately to give a 24-hour detention time for the significant delivery period based on the higher flow rate.

Soil-based Treatment Area (STA): The area devoted to the active subsurface wastewater infiltration system, or alternating and active system zones, plus any required tapered fill areas, slopes and setbacks.

Soil-based Treatment System (STS): The excavation and the piping components used to distribute and infiltrate the wastewater into the subsurface soil.

Subsurface Wastewater Infiltration System (SWIS): Same as STS; the term is used in other states and the 2002 EPA OWTS Manual.