CONNECTICUT DEPARTMENT OF ENVIRONMENTAL PROTECTION

BUREAU OF MATERIALS MANAGEMENT AND COMPLIANCE ASSURANCE

GUIDANCE FOR DESIGN OF LARGE-SCALE ON-SITE WASTEWATER RENOVATION SYSTEMS

February 2006

Funded in part by the CT DEP through a US EPA Nonpoint Source Grant under § 319 Clean Water Act

Jacobson

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DEDICATION

Randall "Randy" May, Principal Sanitary Engineer, Connecticut Department of Environmental Protection, co-author of "Seepage and Pollutant Renovation Analysis for Land Treatment, Sewage Disposal Systems" (Healy and May, 1982), was a strong advocate for the preparation of this document.

Randy passed away in 2000, while still in the prime years of his life. He made numerous friends throughout the state, nation and beyond. He was not only very active in the field of on-site wastewater systems but also participated in development of many Department policies and procedures on other environmental matters as a senior staff member of the Department's Planning and Standards section. Randy also wrote and presented many technical papers and was a featured speaker at professional seminars and symposiums concerned with on-site wastewater treatment. His advice was sought and appreciated by many persons and he was always ready with a humorous anecdote, many times using one to illustrate a point he was making.

Randy was a mentor to many of us who practice in the field of environmental engineering and his advice and good-natured critical commentary are sorely missed. Those of us who were fortunate to know Randy and have him as a friend are much richer for having had that opportunity. He lives on in our memories.

All of us who participated in the preparation of this document are saddened that Randy did not live to critique it and see to its completion. It is to Randy that our efforts, as represented in this document, are dedicated.

ACKNOWLEDGEMENTS

Nathan L. Jacobson, P.E. was the principal researcher and writer of this document. Other members of the consulting engineering firm of Nathan L. Jacobson & Associates, Inc. who assisted in the development of this document were Brian C. Curtis, P.E., Vice President, Wade M. Thomas, M.S., Associate, Hydrogeologist, David P. Campbell, Environmental Analyst; Ms. Tracy Bloch, Technician (CAD); and Ms. Mary Carroll, Administrative Assistant.

Dr. Kent A. Healy, P.E., made a major contribution in preparing Sections II and III of this document and providing valuable guidance on several other sections.

At the Connecticut Department of Environmental Protection, a very significant contribution was made by Warren Herzig, Supervising Sanitary Engineer of the Connecticut Department of Environmental Protection Bureau of Water Management, Land Disposal Section, who administered the preparation of this document and provided a valuable critique of each section. Members of his staff, including Joseph Wettemann, Jennifer Perry Zmijewski, Sarah Overton, William Coleman, Joseph Faryniarz, Robert Lorentson, and Antoanela Daha also made significant contributions by reviewing most sections of this document and participating in a number of meetings at which the various sections were critically reviewed. Jennifer Perry Zmijewski also provided valuable assistance in coordinating the final revisions based on review comments by Department staff.

The assistance of Dr. Gary Robbins of the University of Connecticut Department of Geology & Geophysics and the UCONN Environmental Engineering Program with the analysis of plume development in the ground water down-gradient of a subsurface wastewater renovation system was very helpful and is greatly appreciated.

PREFACE

Twenty four years ago, the Connecticut Department of Environmental Protection issued a landmark publication, “Seepage and Pollutant Renovation Analysis for Land Treatment, Sewage Disposal Systems” (Healy and May, 1982), that provided guidance for meeting the Department’s permitting requirements for on-site discharge of domestic wastewater to the subsurface. That document was, at that time, quite different from those issued by many other state and local regulatory agencies for on-site systems. It stressed an approach to design of such systems that utilized the basic physical, chemical and biological principles that govern the flow and renovation of wastewater that is discharged to the ground water, rather than using prescribed “standards”.

The following document is both an update and expansion of the 1982 publication. In the years that followed the issuance of that document, much additional information has become available regarding the subject matter contained in that document and the Department also gained considerable experience utilizing the concepts set forth in that publication in administering the permitting requirements for on-site wastewater renovation systems. Accordingly, the Department in issuing this new guidance document is building upon the success of the previous publication. This document contains much of the information included in the 1982 publication, as it is timeless. In some cases, however, newer information has replaced older information and in other cases older information has been updated. It also includes subject matter not included in the 1982 publication.

The authors of “Seepage and Pollutant Renovation Analysis for Land Treatment, Sewage Disposal Systems” were Dr. Kent A. Healy, Professor of Civil Engineering, University of Connecticut and Randall “Randy” May, Principal Sanitary Engineer, Connecticut Department of Environmental Protection. In writing that publication, the authors made use of research conducted by Dr. Healy and his associates at the University of Connecticut (UCONN) and research conducted by staff of the Connecticut Agricultural Experimental Station, as well as research conducted by others and published in various professional and scientific publications. The 1982 publication benefited greatly from the keen insight of Kent Healy and Randy May regarding the physical, chemical and biological processes that govern wastewater renovation in the subsurface soil-ground water regime. Likewise, their desire to encourage the use of basic scientific and engineering principles and sound engineering judgment in developing solutions for on-site wastewater renovation contributed greatly to the enlightenment of those persons in the private or public sector who are responsible for designing or permitting of on-site wastewater renovation systems.

Dr. Healy was a well-respected member of the UCONN faculty and is still fondly remembered by many of his former students. He has since retired from UCONN and taken up residence on Martha’s Vineyard, MA, where he is in active practice as a professional engineer. He has actively participated in the writing and critiquing of portions of this new guidance document and his efforts are greatly appreciated. Randy May passed away in 2000.

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TABLE OF CONTENTS[1]

[1] Detailed Tables of Contents are given at the beginning of each section.

SECTION I INTRODUCTION

TABLE OF CONTENTS

GUIDANCE FOR DESIGN OF LARGE-SCALE
ON-SITE WASTEWATER RENOVATION SYSTEMS

SECTION I INTRODUCTION

A. General

The intent in issuing this new document remains the same as when "Seepage and Pollutant Renovation Analysis for Land Treatment, Sewage Disposal Systems" (Healy and May-1982) was published; that is, to emphasize the use of basic scientific and engineering principles and sound engineering judgement in designing on-site wastewater renovation systems (OWRS). That is not to say that the Department does not require design standards. There is a need to establish certain minimum standards in order to protect the public health and the environment. Such standards are presented in a separate document issued by the Department entitled "Standards for Design of Large-Scale On-Site Wastewater Renovation Systems" (hereinafter the "Design Standards") and are based on reasoned evaluation of findings obtained from a review of relevant literature and the performance data obtained from monitoring operations of existing on-site wastewater treatment systems that had been designed based on the Healy and May-1982 publication.

The reader will note that the words "wastewater (sewage) disposal" are not a part of the vocabulary used in this document. To quote David Venhuizen, an engineer specializing in the design and management of on-site systems[1] and fervent advocate of decentralized wastewater management systems: "There is no such thing as disposal of [waste] water - it doesn't go away and it doesn't stay where we put it. We are merely sending it on its journey through the hydrologic cycle."

Domestic wastewater contains pollutants, including certain microorganisms and chemicals, which can be harmful to humans and/or the environment. The microbial pollutants include pathogenic bacteria, protozoa and viruses. Nitrogen, Phosphorus and synthetic organic chemicals are the chemical constituents of domestic wastewater that are of major concern with respect to contamination of ground water. Heavy metals may also be present in the wastewater, but in very low concentrations that are usually removed by electrochemical binding to soil particles. Additional discussion of the pollutants in domestic wastewater is given in following sections of this document.

An on-site wastewater renovation system (OWRS) consists of wastewater pretreatment facilities followed by a subsurface wastewater absorption system (SWAS). While domestic wastewater receives some pretreatment, either in a septic tank or other pretreatment facilities, the effluent from these facilities still contains pollutants that can adversely affect human health or the environment.

[1] Venhuizen, D. 2002. Decentralized Digest 366, 5/11/02. [A national list server for discussion of onsite/decentralized wastewater treatment system management issues.]

When the pretreated wastewater is discharged to the subsurface via a properly designed SWAS it is further renovated as it travels through the subsurface soils and eventually reaches and commingles with the ground water. The ground water in turn is eventually extracted via wells for various water supply purposes, including drinking water, or discharges to surface waters that are used for many purposes.

Therefore, the chief objective for design, construction, operation and maintenance of a SWAS and the associated pretreatment facilities must be to renovate the wastewater so as to protect the public health and the environment. Most soils have substantial but finite capacities to accomplish the renovation of pretreated domestic wastewater by providing an environment that causes the death or inactivation of pathogens and removal or attenuation of chemical pollutants. It is axiomatic that the pretreated wastewater must remain in the soil for a suitable time to permit such renovation to take place. A corollary objective is to ensure that the wastewater makes intimate contact with the soil particles under suitable environmental conditions so as to effect such renovation. This requires that the soils in which a SWAS are installed have ample hydraulic and renovative capacities and that there are adequate vertical and horizontal separating distances between the SWAS and any point of concern to provide the necessary time and adequate soil contact for the renovation to take place.

While this document is directed toward design, construction, operation and maintenance of large-scale OWRS having design flows greater than 5,000 gpd, including associated wastewater collection systems, the underlying principles involved apply to all on-site systems, regardless of size. The size of a system is a function of the daily rate of wastewater discharge and its bio-chemical characteristics, and the physical characteristics of the site, including: area, shape, topography, depth to the controlling ground water table and soil characteristics. The two basic criteria for judging the adequacy of an OWRS are: “Will the discharge cause pollution?” and “Will the system work?”

The basic concerns that must be addressed to judge the adequacy of an OWRS are:

- Does the proposed site of a subsurface wastewater absorption system (SWAS) have sufficient land area to accept the size of the system necessary to meet the requirements of the Department?
- Does the soil-ground water regime in which the SWAS is proposed to be located have sufficient renovative capacity to bring the pretreated wastewater into compliance with the required ground water quality standards of the Department before it reaches a point of concern, such as: a potable water supply well, wetland, surface water body or the applicant’s property boundary?
- Does the soil-ground water regime in which the SWAS is proposed to be located have sufficient hydraulic capacity to accept and transport the pretreated wastewater for an adequate distance without surfacing or breakout?
- Is or will there be a responsible entity, with adequate and continual authority and assured financial means, to properly construct, operate and maintain the OWRS to the satisfaction of the Department?

To address these concerns, many factors must be thoroughly and methodically evaluated, using the best engineering practice in applying fundamental scientific and engineering principles, and the best information currently available or reasonably obtainable.

The basic purpose of this document is to present information and methodologies that can be used in evaluating these factors and addressing these concerns.

B. Units Of Measure

The U.S. ("English") system of measurement units is utilized in this document, with few exceptions (e.g. mg/L, meters). For those persons who need, or prefer, to work in metric units, a table of U.S. to Metric conversion factors is provided in Appendix D.

C. Terminology

As previously stated, this document stresses the renovation of domestic wastewater. However, at the time this document was written, the governing State Statutes and the Department's Water Quality Standards, Water Discharge Regulations, Rules of Practice and Environmental Permit Application Package all refer to "Sewage", "Domestic Sewage", "Sewage Disposal", "Subsurface Sewage Disposal System", "Leaching System" and like terms. In this document, the following words and terms are equivalent:

This Document	Existing Terminology
Wastewater	Sewage
Domestic Wastewater	Domestic Sewage
Wastewater Renovation	Sewage Disposal
Subsurface Wastewater Absorption System (SWAS)	Subsurface Sewage Disposal System, or Leaching System
On-site Wastewater Renovation System (OWRS)	Land Treatment System, Subsurface Sewage Disposal System

D. Disclaimer

Throughout this document, proprietary commercial products and processes have been mentioned by trade name in order to illustrate a point or to provide a general indication as to what products or processes may be available for use in on-site wastewater renovation facilities. Mention of trade names, proprietary commercial products and processes does not constitute endorsement or recommendation for use by the Department.

E. Departments' Jurisdiction over OWRS

The Department has jurisdiction over the design, construction and operation of: OWRS facilities having a design capacity in excess of 5,000 gallons per day that discharge to any one property, regardless of the number of systems; systems including advanced pretreatment regardless of capacity; and Community Sewerage Systems (those serving more than one residential structure). Under t 22a-430 of the Connecticut General Statutes (CGS), the Department is responsible for issuing State Discharge Permits for operation and monitoring of such systems.

F. Recommended Procedure for Applicant to Follow

Step 1. Applicant and Applicant's Engineer meets with Department staff to discuss the proposed project, obtain Department staff input on the information that will need to be developed to accompany a Discharge Permit Application and schedule site testing. (It should be noted that at this initial meeting, the Department staff, and in many cases the Applicant's Engineer also, are not fully aware of the limitations of a proposed site for an OWRS. Therefore, the information initially requested by the Department may have to be supplemented with additional information after the initial site characterization has been completed.)

Step 2. Applicant submits Discharge Permit Application, conceptual design and supporting documentation to the Department for review. Applicant also issues a public notice of the permit application in a newspaper having circulation in the project area and provides a copy of the notice to the Chief Elected Official of the community in which the project is located, as required by t 22a-6g of the CGS.

Step 3. Review of submitted documentation by the Department's Bureau of Water Management. Upon review of this documentation, the Department staff may request additional information.

Step 4. When the Department is satisfied with the conceptual design of the OWRS, the Department issues a public notice in a newspaper having circulation in the project area stating that on the basis of preliminary review of the permit application and supporting documentation, the Department has made the tentative determination that the proposed OWRS will protect the waters of the State from pollution, and the Department's Commissioner proposes to require the Applicant to submit plans and specifications for the proposed system and such additional information as the Commissioner deems necessary to ensure the protection of the waters of the state from pollution.

The notice also states that if the Commissioner approves such plans and specifications, and the proposed system is constructed in full compliance with the approved plans and specifications, the Commissioner proposes to issue a permit for the discharge. Prior to making a final decision to approve or deny any application, the Commissioner must consider comments from interested persons that are received within 30 days of the Public Notice date. If the Commissioner decides that the public interest will best be served thereby, or upon receipt of a petition signed by at least 25 persons, or intervention by an interested party under the Connecticut Environmental Policy Act, a Public Hearing will be held on the application. Notice of any public hearing must be published at least 30 days prior to the hearing in a newspaper having circulation in the project area.

Step 5a. If the Public Notice period expires without receipt by the Department of adverse comments, request for a public hearing or intervention by an interested party, and the Commissioner does not deem a public hearing to be necessary, the Commissioner makes a final determination and the Department advises the Applicant to submit final construction contract documents for the OWRS.

Step 5b. If a Public Hearing is held, a Department Hearing Officer will prepare findings and recommendations for the Commissioner regarding approval or denial of the application.

Step 5c. If a Public Hearing is held and the Hearing Officer's recommendation is affirmative and the Commissioner accepts the recommendation of the Hearing Officer, the Commissioner makes a final determination and the Department advises the Applicant to submit final construction contract documents for the OWRS. This action may be appealed by anyone that is a party to the public hearing proceedings.

Step 6. Final construction contract documents are submitted to the Department Bureau of Water Management for review and approval.

Step 7. When the Department is satisfied with the construction contract documents, it issues an approval to the applicant to proceed with construction of the OWRS in full compliance with the approved final construction contract documents. The Department approval will require that the applicant retain a licensed professional engineer to provide construction services to verify that construction of the OWRS is done in conformance with the approved construction contract documents. Pre-construction meetings with Department staff may be required on certain projects.

Step 8. Upon completion of construction, if the Department finds that the construction is in compliance with the approved construction contract documents, and upon receipt of record drawings with supporting information and the design engineer's verification that the construction is in conformance with approved contract documents, the Department issues a State Discharge Permit to initiate a discharge.

The Department may include certain conditions as part of the permit approval process. Examples of types of conditions include:

a. Preparation of an operation and maintenance manual for the proposed OWRS.

b. Operation of the completed facilities under the direction of a State licensed wastewater treatment plant operator for facilities involving advanced pretreatment of the wastewater.

G. Summary of Basic Design Requirements

The applicant must demonstrate that sufficient land area with suitable soil conditions is available to install an OWRS conforming to the following criteria:

1. The OWRS facilities must be sized on the basis of approved conservative wastewater design flows and wastewater characteristics.

2. The soils in which the proposed subsurface wastewater absorption system (SWAS) will be installed must have sufficient hydraulic capacity, including an appropriate hydraulic reserve capacity as established in the Design Standards, to transmit the pretreated wastewater for a sufficient distance to permit renovation of the wastewater to drinking water quality before it reaches the closest point of concern.

3. To provide for attenuation of organic and inorganic pollutants and pathogens remaining in the pretreated wastewater, the soils in which the proposed SWAS will be installed must have sufficient hydraulic capacity to provide a minimum depth of unsaturated soil beneath the bottom of the SWAS leaching facilities, during mounded seasonal high ground water periods, as established in the Design Standards.

4. Any remaining pathogens that were not removed (by filtration, die-off, or inactivation) in the unsaturated zone below the SWAS must be removed by natural means in the saturated soils down gradient of the SWAS before the commingled effluent/groundwater reaches a point of concern. The required time of travel of the wastewater from the point of disposal to the closest point of concern must conform to the requirements in the Design Standards.

5. Application rates of pretreated wastewater should not exceed the soil capacity for pathogen attenuation.

6. The concentration of total nitrogen in the pretreated and renovated wastewater at the closest point of concern should not exceed the appropriate water quality values established in the Design Standards.

7. The phosphorus in the pretreated wastewater should be removed by the soil before the renovated wastewater reaches the closest point of concern, with no discharge of phosphorus of other than natural origin permitted to any surface water body.

Further information on relevant regulatory requirements and Design Standards for discharge of wastewater to the ground waters of the State can be found in the following publications, which may be obtained from the Connecticut Department of Environmental Protection:

1. Water Quality Standards and Criteria
2. Water Discharge Regulations §b22a-430 of CGS
3. Rules of Practice – § 22a-3a-2 through 6 of the Regulations of Connecticut State Agencies (RCSA)
4. Permit Application for Wastewater Discharges (DEP-PERD-APP-100 and all supporting Documents)
5. “Standards for Design of Large-Scale On-Site Wastewater Renovation Systems” (hereinafter the “Design Standards”)

Most of these documents may also be obtained via the Internet. The Department's web address site is: http://dep.state.ct.us. Care should be taken to assure that documents obtained are the most current ones available.

Applicants and their consultants are urged to obtain a copy of each of these documents and review them prior to initiating a permit application for a discharge of wastewater to the ground waters of the State. (In the case of OWRS, the permit discharge category is: Land Treatment Non-point Source Systems.)

Likewise, applicants and their consultants are urged to attend a preliminary meeting with a staff member of the Department's Land Disposal Section prior to beginning any extensive field work on a proposed OWRS, in order to avoid unnecessary expense and loss of time. However, a desktop review of available information, including anticipated wastewater flows and characteristics, topography, adjacent points of concern, soil, surficial geology and bedrock geology data, and preliminary field test pit investigations should be made to determine if there are any obvious impediments to the use of the proposed site for on-site wastewater renovation.

H. Role of The Principals

The proper siting, design, construction, operation and maintenance of an OWRS are all crucial to the satisfactory performance of these systems, and thus, adequate protection of potable water supplies, ground water, surface water and the public health. Several distinct parties are involved in the development and use of an OWRS. These include: I) Owner/Developer, II) Designer, III) Regulator, IV) System installer and V) Owner/operator.

1. Owner/Developer

This can be the present landowner, potential buyer or the builder. Typical responsibilities of the owner consist of:

a. Responsible party for the project
b. Hires the designer(s) and installer
c. Obtains permit and is responsible for compliance with it
d. Transfers information to subsequent owner

2. Professional Consultant(s)

The design of on-site systems is an interdisciplinary project that may be performed by one or more individuals with specialized skills in several phases of on-site system design. Each consultant should be an individual with appropriate training and licensure or certification in Civil/Sanitary Engineering, Soil Science, Public Health, Hydrogeology or Environmental Science and have experience designing on-site systems. Typical responsibilities of the Consultant(s) include, but are not limited to, the following:

a. Site Evaluation

    1. Assessment of the Site's Suitability

        - Soils
        - Groundwater
        - Bedrock

- Topography
- Hydrogeology
- Isolation from water supplies and other features

2. Site Plan

   - Measurement of Essential Features
   - Topography
   - Soil Logs
   - Groundwater Contours
   - Wetlands
   - Locations of groundwater reaching grade
   - Water Supply Location
   - Detailed and Scaled Map/Plan
   - Subsurface wastewater absorption system (SWAS) location and orientation

3. Site Report

   - Site Hydraulic and Renovative Capacity
   - Reasons for SWAS Location and Orientation
   - Site Limitations

b. System Design

1. Basis of Design

   - Wastewater Source
   - Wastewater Flows and Characteristics
   - Design Loads
   - Hydraulic Loading Cycles
   - Design Parameters for any Enhanced Pretreatment Facilities
   - System Operation Concept
   - Overall Site Development Plan

2. SWAS Design

   - Loading Rates (Hydraulic, Organic, Nitrogen, Phosphorus)
   - Type of Distribution System
   - SWAS Sizing

3. Site Layout

   - Site Modification
   - System Component Placement

4. Final Plan
    - Site Plan
    - Specifications
    - All materials, including earthen storage locations
    - Component Plan and Hydraulic Profiles
    - SWAS Plan and Profiles
    - Details
    - Construction Instructions
    - Restrictions on type and use of equipment
    - General sequence of construction

5. Operating Plan

c. Construction Oversight

1. Site Layout
    - Bench Marks
    - Stake Out System

2. Pre-construction Conference with Owner, Installer & Regulator
    - Explanation of plans, specifications & construction sequence

3. Construction Administration
    - Site Visits
    - Inspection
    - Component Testing

4. Record Drawings
    - Prepare record drawings including location & depths of components
    - Delivery to owner & regulator

d. Operation and Maintenance Instructions

1. Provision to Owner

3. Regulator/Reviewer

This is the governmental official responsible for regulatory compliance. This individual should be employed by the state, have appropriate training in Civil/Sanitary Engineering, Soil Science, Public Health, Hydrogeology, or Environmental Science and have experience with on-site systems. The regulator/reviewer's responsibilities may include:

a. Witnessing a representative portion of the site testing to:

1. Confirm site conditions
2. Review Designer(s) findings
3. Determine if any additional information is necessary for a thorough evaluation

b. Application Review

   1. Assures application is complete
   2. Assures all other regulatory requirements are being addressed

c. Technical Design Review

   1. Checks plans for regulatory compliance and good design practices
   2. Assures that design considers all site and design factors
   3. Confirms review and issues in writing

d. Approval/Permit Issuance

   1. Produces formal approval document
   2. Writes necessary construction, operation, maintenance and reporting conditions
   3. Assures all parties advised of decisions

e. Installation Inspection

   1. Conducts inspection during construction
   2. Inspection prior to backfill
   3. Reviews all change orders
   4. Reviews designer's construction certification report
   5. Review and filing of record drawings

f. Operation & Maintenance Monitoring - May not be the same entity as approving authority

   1. Review of approval/permit conditions
   2. Review of operation and monitoring submittals
   3. Provides owner with maintenance and operation information
   4. Maintains all files including record drawing, maintenance and repair records for at least 20 years
   5. System re-inspections

g. Enforcement - The regulatory community also maintains an enforcement program

4. System Installer

This is the individual or firm that builds the system. The installer must be state certified as required by the Public Health Code, have received training, and be knowledgeable in the installation of on-site systems. The installer's responsibilities typically include:

a. Pre-construction

   1. Site Inspection
   2. Plan Review
   3. Construction conference with Owner & Designer
   4. System Stake Out - confirm with designer

b. Construction

   1. Build the system as designed
   2. Clear all field adjustments (change orders) with owner, designer & regulator and seek approval
   3. Follow the construction plan
   4. Utilize good construction procedures
      - Avoid construction of SWAS during high soil moisture
      - Use correct equipment
      - Use specified materials

c. Final Inspection

   1. Notation of Change Orders
   2. Location of Components
   3. Coordinate with Designer to insure the record drawings are correct
   4. Coordinate final inspection with Regulator

d. Turnover to Owner

   1. System demonstration and component locations
   2. Provide Record Drawings to Owner

5. System Owner (Operation and Maintenance)

This is the individual who will actually own and maintain the system. The Owner's responsibilities include:

a. Obtain Record Drawings and Permit Documents and Requirements of local Water Pollution Control Authority

   1. From seller
   2. Contact designer/regulator for missing information
   3. Contact installer for any special construction information

b. Locate system components and benchmark

c. Set-up maintenance schedule

d. Contact regulatory bodies for compliance information and maintenance/operation information

e. Assure permit condition compliance

f. Routine Maintenance - determined by permit requirements and may include:

   1. Regular inspection of all permitted facilities
   2. Septic tank pumping on regular schedule based on loading and inspection
   3. Regular walkover for obvious problems
   4. Record inspection/maintenance/repair

g. When facilities served are sold, transfer of permit and all records to Buyer (new owner)

I. Keeping Informed on Continuing and New Research

In keeping with the intent of issuing this document, as discussed on page 1 of this section, it is crucial for persons involved with design and review of OWRS to keep up-to-date and informed on the continuing and new research that is being and will be conducted in this field. This document is by no means an exhaustive compendium of all the knowledge required for design of a fully satisfactory OWRS; rather it is a "living document" that will have to be periodically brought up to date as new information becomes available.

J. References

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SECTION II RENOVATION OF WASTEWATER IN SUBSURFACE SOILS

TABLE OF CONTENTS

TABLES

SECTION II RENOVATION OF WASTEWATER IN SUBSURFACE SOILS

A. Introduction

This section discusses: 1.) pollutants usually present in domestic wastewater that are of most concern with respect to adverse effects on human health and the environment; 2.) properties of soils that effect wastewater renovation; and 3.) the ability of the subsurface soil environment to remove or attenuate pollutants in the pretreated wastewater discharged to a subsurface wastewater absorption system (SWAS). More specific information on each of the topics discussed herein is given in some of the following sections of this document.

Pollutants contained in domestic wastewater include suspended and dissolved biodegradable natural organic chemicals, a wide range of synthetic organic chemicals (usually in the dissolved form and varying in their susceptibility to biodegradation), inorganic chemicals (including heavy metals) and pathogenic microorganisms that can be harmful to humans and/or the environment. The wide range in the types and concentrations of such pollutants speaks to the need to exercise caution when selecting pollutant concentration values for designing on-site wastewater renovation systems (OWRS).

While domestic wastewater receives some pretreatment, either in a septic tank and/or other pretreatment facilities, prior to discharge to the subsurface via a SWAS, the effluent from these pretreatment facilities still contains pollutants that can adversely effect human health or the environment. It has been amply demonstrated by numerous investigators, in the U.S. and elsewhere, that most soils have substantial but finite capacities to accomplish the renovation of pretreated domestic wastewater by providing an environment that causes removal or attenuation of chemical pollutants and the death or inactivation of pathogens.

With respect to the soil's capacity to accept and renovate a discharge of pretreated wastewater, two separate capacity factors must be considered. The first factor is the soil's hydraulic capacity to accept, contain and transport the pretreated wastewater in such a manner that renovation will occur before the pretreated wastewater reaches a point of public health or environmental concern. This capacity depends upon the soil's hydraulic conductivity (a measure of how easily water flows through the soil), the driving force (hydraulic gradient or pressure head) available for inducing flow through the soil, and the cross-sectional area of the soil through which the pretreated wastewater must flow.

The second factor is the soil's renovative capacity. For a particular soil, this is a volumetric capacity. It depends upon volume of soil through which the pretreated wastewater flows, and the renovative capacity per unit volume of soil for each pollutant.

It has been found that significant renovation is achieved as the wastewater flows through a biomat that forms at the infiltrative surface of the SWAS and then into an unsaturated, aerobic soil zone between the bottom of the SWAS and the water table. To obtain this renovation, there must be an adequate depth of unsaturated soil, and the pretreated wastewater must be applied in such a manner and at such a rate as to not overwhelm the renovative capacity of the unsaturated soils.

Additional renovation of the wastewater can and often does continue to take place after the pretreated wastewater has percolated through the unsaturated zone and reached the ground water. This renovation is a function of the renovative capacity of the soil in the saturated (ground water) zone and the time it takes for the commingled percolate and ground water to reach a point of concern, such as a drinking water supply well, property line or surface water body. This travel time is generally a function of the horizontal separating distance between the SWAS and the closest point of concern and the velocity of a pollutant as it travels with or through the ground water. The horizontal separating distance is important in that it provides time for the pollutants remaining in the commingled waters to be exposed to environmental conditions in the saturated soils that results in their additional decay or attenuation.

B. Soil Features and Wastewater Renovation

In the following discussion on soil features as they effect wastewater renovation, the terminology used is that of soil scientists, rather than that normally used in geotechnical engineering. It is the terminology used by the U.S. Department of Agriculture, Natural Resources Conservation Service (NRCS) in their soil survey reports and other soils publications. There are several NRCS references listed in the bibliography at the end of this section that provide a great deal of information on soils. The reader is urged to refer to them, or references of similar content, to gain a fuller understanding of soil features that are only discussed in a very rudimentary manner herein.

The NRCS has published a soil survey for each of the eight counties in Connecticut. However, the new digital "Soil Survey of the State of Connecticut, dated July 15, 2005", is the official soil survey for the state. The digital survey can be found on the website: http://www.ct.nrcs.usda.gov.

The soil maps included in the soil survey show the kinds of soils in an area and their location, distribution and extent. These maps provide useful information for site screening and planning purposes, but are inadequate for obtaining the detailed soils information required for design of a SWAS. The boundaries of the soil mapping units are not a perfect representation of the soil patterns on the ground. This is because soils grade from one to another and are not always present in bodies large enough to delineate at the map scale used for the maps included in the various soil surveys of Connecticut counties.

For example, the minimum size delineation at a map scale of 1:15840 (typical of the original county soil maps) is 2.5 acres. The new soils maps, to be published at a scale of 1:12,000, will have a minimum size delineation of 1.43 acres (Sautter - 1990). Soil differences often occur within short distances. Within most mapping units are small areas of soils (inclusions) that differ significantly from the named soils. They are typically too small to be delineated separately (NRCS-1991). Thus, while a soil survey report is highly useful in gaining broad understanding of landscapes, having such a report in hand does not remove the need for onsite investigation in determining suitability of soils for SWAS (Brown, R.B.-1992)[1].

The soils in Connecticut were formed from parent materials deposited during at least two separate occasions by glaciers that covered the entire region with ice sheets ranging up to thousands of feet in depth. These soils have a wide range of complexities in their characteristics. They have been classified into 94 major soil series (the basic units of soil classification) consisting of soils that are essentially alike in all major profile characteristics. Each of these soil series has a definite relationship to the landscape, local geology, and parent materials (Sautter - 1990). The local geological and hydrologic conditions associated with these soils greatly modify the capacity of the soils to accept and renovate domestic wastewater. This association is complex and provides an infinite spectrum of soil capabilities for accepting and renovating wastewater and eliminates the use of generalizations for the use of soil as a medium for wastewater renovation (Miller and Wolfe-1982, Reneau, et al.-1996).

The soil series, consisting of a characteristic profile with a unique arrangement of horizons, occur in a variety of landscape positions. Each soil profile depicted is a vertical cross section of the undisturbed soil showing the characteristic soil horizons (horizontal layers), that have formed as a result of the combined effects of parent material, topography, climate, biological activity, and time.

The soil horizons consist of more or less distinct layers of soil running approximately parallel to the soil surface and they have distinct characteristics produced by pedogenic (soil-forming) processes. They are normally designated alphabetically, but not necessarily in alphabetic order, proceeding vertically through the soil profile from the soil surface downward. An upper case letter represents the major horizons; numbers or lower case letters that follow represent subdivisions of the major horizons. The thickness of each horizon can vary considerably in relatively short horizontal distances, and not all horizon designations may be present at any one location.

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[1] Other sources of information on subsurface conditions can be found in the various publications available from the CT DEP Natural Resources Center. These include such publications as the State Geological and Natural History Surveys of Connecticut (bedrock geology, surficial geology), the U.S. Geological Survey Water Resources Inventories of Connecticut and Surficial Geology Quadrangle Maps, and other similar publications covering various locales in the State.

The master horizons include:

O-horizon	An organic layer of fresh and decaying plant residue (litter and humus) at the surface of a mineral soil.
A-horizon	The mineral soil at or near the surface in which an accumulation of decayed organic matter (humus) is mixed with the mineral material, which predominates.
E-horizon	Consists of mineral soil characterized by the loss of iron, aluminum, calcium, clay or organic matter to the underlying B-horizon due to leaching.
B-horizon	The mineral soil below an A-horizon or E-horizon. It is in part a transitional layer from the overlying soil horizons to the C-horizon. It contains accumulations of clay and metals (e.g. iron, aluminum, calcium) and humus.
C-horizon	The mineral layer, excluding bedrock, which is little effected by soil forming processes and does not have the properties typical of the overlying horizons.
R-horizon	Bedrock.

These horizons can also be grouped into surface soils, subsurface soil, subsoil, and substratum. A very basic description of these groupings, taken from R.B. Brown (1992), is given below:

- Surface Soil. The surface soil, or "topsoil", is enriched in organic matter -- and usually darkened somewhat as a result -- from decomposing organisms.

- Subsurface soil. The subsurface horizon, if present, is a leached zone, beneath the surface soil, from which mobile soil constituents (clay particles, organic matter, iron and aluminum oxides, carbonates, and/or other constituents) have been removed in solution or suspension by downward-percolating rainwater and/or by a fluctuating water table.

- Subsoil. Underlying the surface soil and/or subsurface soil may be"subsoil". The subsoil, where present, is a zone that has been influenced in some significant manner by soil-forming processes. These processes may be subtle, consisting of simple alteration of chemicals and minerals, or very distinct, consisting not only of alteration but enrichment of subsoil horizons by materials that have been leached from the overlying topsoil and subsurface soil and deposited in the subsoil. Where such enrichment has occurred, a relatively clayey horizon, a horizon enriched with organic materials and iron and aluminum oxides, or other horizons marked by some sort of accumulation may result.

- Substratum. Beneath the zone of soil formation is the "substratum", which is non-soil material, ranging from bedrock to unconsolidated sediment. Substrata are largely unaffected by soil-forming processes other than deep weathering." This is equivalent to the master C-horizon.

In addition to the soil-forming factors of alteration, leaching, and enrichment, the stratification of soil or soil parent materials can have enormous impact on the behavior of water in soils. Stratification is the result of deposition of relatively fine-textured layers over and/or under coarser-textured layers. Behavior of water, including septic tank effluent, is a function of such layering, of the permeability of the various soil horizons, position in the landscape, weather and other factors (R.B. Brown -ibid.).

Inspection of the soil survey reports for the Connecticut counties reveals that the B-horizon rarely extends more than 2.5 ft below the A-horizon. Below this depth lies the C-horizon, which is usually lacking in the soil characteristics discussed below which contribute the most to wastewater renovation. This indicates that the portion of an SWAS located below the B-horizon will not make use of the most effective characteristics of the soil profile.

Those soil features which effect the removal or attenuation of pollutants remaining in the percolate discharged to the soil beneath a SWAS (e.g. natural organics, suspended solids, nitrates, phosphates, human pathogens and toxic organic chemicals), include:

- texture and structure,
- silt, clay, organic and reactive mineral content,
- microbial population,
- soil gas oxygen content,
- cation exchange capacity,
- pH,
- alkalinity,
- temperature; and
- moisture content.

Soil texture refers to the visual or tactile surface characteristics of soil, and to the distribution, on a percent by weight basis, of sand, silt and clay. Soil structure is the arrangement or grouping of individual soil particles into aggregates or clusters (Peds). The principal forms of soil structure are blocky (angular or sub-angular), columnar (prisms with rounded tops), granular, platy (laminated) and prismatic (vertical axis of aggregates longer than horizontal). Structure-less soils are either single grained (each grain by itself) or massive (the particles adhering without any regular cleavage, as in many dense till soils).

Texture and structure are important factors governing the movement of water, pollutants, and microbial populations in porous media. Clay particles are particularly dominant in determining the physical and chemical properties of soils. For example, clays, which are often composed of aluminum silicates, add both surface area and charge to a soil. The surface area of a clay particle can be five orders of magnitude larger than the surface area of a 2 mm sand particle (Maier and Pepper - 2000). These characteristics (surface area and electrical charge) of clays play an important part in the sorption of pollutants. Structure is important in that a few large macropores (voids left by decayed roots, cracks, animal and worm burrows, etc.) in the soil can have a very large influence on the soil's ability to transmit wastewater.

Under saturated soil conditions, wastewater fills all the soil pores; however under unsaturated conditions it flows predominately through the small pores or is retained as a film around soil particles. When all soil pores are filled with wastewater, the macropores in structured soils (and the larger pores in coarse-grained unstructured soils) transmit most of the wastewater at rapid rates and do not allow the pollutants to come into close contact with the soil particles. Wastewater flowing through the small pores in unsaturated soils travels at a slower rate, and through a more tortuous path, giving more time for close contact of the pollutants with the soil particles. Such close contact is of paramount importance for sorption of pollutants onto the soil particles, which is in many cases the necessary first phase in the attenuation or removal of pollutants from wastewater.

Two other soil characteristics are important to the design and operation of a SWAS. These are hydraulic conductivity (a measure of how easily water flows through soil), and porosity (the ratio of the volume of pores in a soil to the bulk volume of the soil). These characteristics are a function of a soil's texture, structure and bulk density. Hydraulic conductivity is also a function of the temperature of the liquid, although this is typically ignored as being insignificant in design of a SWAS, and the soil moisture content. (Note: In the past, the term "permeability" was often used in some texts and technical papers, rather than "hydraulic conductivity", to define the soil characteristic used to determine the hydraulic capacity of the soil. However, "permeability" has been replaced by "hydraulic conductivity" in the current literature, since these terms refer to somewhat different soil characteristics, and it is the hydraulic conductivity of a soil that is of interest in designing a SWAS. Thus, unless otherwise stated, wherever the term "permeability" is encountered in this document, it shall be taken to mean "hydraulic conductivity".)

Dilution of pollutants, such as nitrates and toxic organic chemicals in the SWAS percolate, by precipitation infiltrating to the ground water beneath and downgradient of a SWAS, and by dispersion in the ground water, has been found to be generally much less than assumed in previous decades. Therefore, consideration needs to be given to optimize the conditions that permit the unsaturated, aerobic soil located above the water table to remove or attenuate pollutants. Understanding and identifying those soil features that are vital to the soil's ability to renovate wastewater is required prior to designing an SWAS in order to make efficient use of the renovative capacity of the soil. Only then can informed decisions be made with respect to SWAS design parameters such as system siting, layout, hydraulic and pollutant loading rates, etc.

C. Pollutant Removal Processes

Pollutants are removed from wastewater in and below a SWAS by several processes, including:

- Physical processes, such as filtration of suspended solids and some microorganisms;
- Biodegradation processes that transform organic and inorganic pollutants into harmless or less harmful substances;
- Sorption (absorption, adsorption, chemisorption and chemical precipitation) processes that remove chemicals and microorganisms from the wastewater; and,
- Biological and physiochemical processes that result in the death or decay of pathogenic microorganisms.

When wastewater receiving pretreatment in a septic tank is discharged to a SWAS, a slimy biomat begins to form at the infiltrative surface of the soil, and eventually reaches a thickness of one or more centimeters, depending upon the amount of organic matter and suspended solids in the wastewater. This biomat consists of an accumulation of suspended solids filtered from the wastewater, indigenous soil microorganisms (both their active cells and cell remnants), the slimes (e.g. polysaccharides) produced by these microorganisms and various minerals. The biomat acts as a filter and as a medium for biochemical degradation, absorption and adsorption of pollutants; it plays an important part in the pollutant removal process, as a significant amount of pollutant removal occurs as the wastewater contacts and passes through the biomat.

The biomat also serves to restrict the rate of flow of wastewater through the adjacent soils. This results in the wastewater flowing through the smaller soil pores, bringing the wastewater into close contact with the soil particles while allowing the larger soil pores to remain open to the diffusion of atmospheric oxygen into the subsurface soils. The slower flow rate through the smaller pores results in increased contact time between the pollutants and the soil particles. It is the increased contact time, the close contact of the wastewater with the soil particles, and the existence of an aerobic environment that results in the efficient removal or attenuation of the pollutants.

Various types and species of microorganisms exist on and within the biomat. Of major importance are the heterotrophic aerobic, anaerobic and facultative bacteria, which require organic carbon as a food source, and the autotrophic bacteria, which obtain their carbon from inorganic compounds (e.g. carbon dioxide). All bacteria require water and a food source to exist. Aerobic bacteria also require the presence of free dissolved oxygen for their metabolic processes, whereas anaerobic bacteria obtain their oxygen by breaking down chemical compounds, and cannot function in an aerobic environment. Facultative bacteria have the unique ability to function in either an aerobic or anaerobic environment, although they function much more efficiently under aerobic conditions. Autotrophic bacteria are strict aerobes.

It is important to note that the removal or attenuation of most pollutants in domestic wastewater is best accomplished in an aerobic soil environment.

As the pretreated wastewater percolates through the biomat and reaches an aerobic, unsaturated soil zone beneath the SWAS, biochemical degradation continues to occur, and in addition, certain pollutants become sorbed to the soil particles. This sorption process can be irreversible or reversible, depending upon the nature of the attractive forces between the pollutants and the soil particles, the pH of the soil solution and the soil moisture content. Pathogens may be adsorbed for a sufficient time to allow their degradation by indigenous antagonistic microorganisms, or by their natural death or decay (inactivation) processes. Heavy metals may also be sorbed and immobilized. Soils with the highest percentage of clay and iron oxides, and greatest cation exchange capacity are most effective in immobilizing heavy metals (McHugh - 1990). However, all soils have a finite sorption capacity and, if that capacity is exceeded, pollutants in percolating wastewater can enter and travel with the ground water.

It is also possible that after reaching the ground water, pollutants may continue to be filtered, sorbed or biochemically degraded. However, in the latter case the ground water is generally anoxic or anaerobic, and thus the biochemical processes are much slower than the aerobic processes that take place in the unsaturated zone. In most cases the saturated soil is limited or lacking in organic material and this also hinders the microbes that require such material for their metabolic processes. Where there is insufficient travel time between the point of entry of the pollutants into the ground water and a point of concern, the pollutants may not be removed before they reach the point of concern.

D. Suspended Solids

Domestic wastewater contains both organic and inorganic suspended solids (TSS, or total suspended solids). These suspended solids, if not largely removed in the septic tank, can clog the pore spaces between soil grains at the soil interface with the infiltration system and the biomat that forms at that interface.

While some of the organic suspended solids may be biochemically degraded in the biomat, and thus cause a biochemical oxygen demand, some are highly resistant (refractory TSS) and may take a long time to degrade. It is therefore important to minimize the carryover of TSS from the septic tank or other pretreatment facilities.

E. Organic Chemicals

When wastewater containing biodegradable organic chemicals is discharged to the subsurface, these organics are used as a source of food and energy in the metabolic processes of various microorganisms indigenous to the soil and ground water. As a result, these organics are largely removed from wastewater provided the environmental conditions in the subsurface are suitable for the microorganisms. Many of these indigenous microorganisms are aerobic, requiring free (molecular) oxygen for their metabolic processes.

The principal sources of free oxygen are the gaseous molecular oxygen that has diffused from the atmosphere into the unsaturated pores of the soil and the free molecular oxygen that is dissolved in the ground water. If sufficient free molecular oxygen is available, the aerobic microorganisms will rapidly degrade the organic chemicals to carbon dioxide, water and new microbial cellular material.

However, if an excess amount of organic chemicals is discharged to the subsurface, the oxygen present will be depleted as the numbers of aerobic microorganisms increase in an attempt to utilize this “food”, resulting in anoxic (very low free oxygen concentrations) or anaerobic conditions (no free oxygen present) prevailing. When these conditions occur, anaerobic or facultative microorganisms that are able to exist and function when free molecular oxygen is absent or present in very low concentrations will utilize chemically bound oxygen in their metabolic processes to degrade the organic chemicals.

However, under anaerobic conditions, the degradation of the biodegradable organic chemicals is slower and less efficient, and the degradation by-products can cause undesirable taste, odor and color in the water, making it unpalatable. Therefore, it is critical that aerobic conditions are maintained in the subsurface into which wastewater is introduced.

Many synthetic organic chemicals, both volatile and non-volatile, have been found to be toxic to humans and other life forms in low concentrations in water. Synthetic organic chemicals are found in numerous household products, including cleaning agents, solvents, “septic system cleaners” pesticides, pharmaceuticals and personal care products (drugs, including antibiotics; food supplements, fragrances, sunscreen agents, etc.).

Some investigators have indicated that where the concentrations of these chemicals in wastewater are low, many are substantially degraded by microbial action in the biomat and unsaturated soils before they reach the ground water (Bicki and Lang - 1991; Robertson, et al. - 1991; Sherman and Anderson - 1991; Siegrist, et al. - 2000). However, other investigators have determined that in some cases, synthetic organic chemicals can and do reach the ground water. Samples of untreated ground water from 1,926 rural, self-supplied domestic wells were tested for volatile organic compounds (VOCs) during the period 1986-1999. Water samples were collected at the wellhead prior to treatment or storage. In most samples, analyses were conducted for 55 target VOCs, and occurrence and status information generally was computed at an assessment level of 0.2 µg/L. The seven most frequently detected VOCs have many uses in a variety of household products and have been found in septic tank systems. For example, tetrachloroethene is predominantly used as a solvent, but also is used in products such as household cleaners and products used for personal hygiene. At least one VOC was detected in 12 percent of samples at the assessment level, with detection of at least one VOC found in samples collected from wells located in 31 of 39 states (including Connecticut, where 6 out of 12 wells sampled contained one or more VOCs.) (Moran et al. - 2002).

Despite the concerns, samples of ground water from rural, self-supplied domestic wells often did not contain VOCs, and where VOCs were detected, the occurrence and concentration levels were low. Solvents were the most frequently detected VOC group, and were found in 4.6% of samples at an assessment level of 0.2µg/L. However, only 1.4% of samples had one or more VOC concentrations that exceeded a federally established drinking water standard, a health criterion, or both, and only 0.1% of samples had VOC concentrations that exceeded a taste/odor threshold (Moran, et al. - ibid.) In general, user education on minimizing the discharge of VOCs' to an OWRS is recommended.

It should be anticipated that when domestic wastewater is discharged to a large-scale SWAS, not all synthetic organic chemicals may be removed by microbial action, and some may reach and mix with the ground water. In such cases, it is possible that the resulting concentrations may reach levels that will have an adverse impact on human health and/or the environment. This may be particularly true for wastewaters containing synthetic organic chemicals on a more or less regular basis. Such wastewaters may include, for example, those discharged from food preparation and serving facilities (e.g.: restaurants, multiple residential dwelling facilities with common dining facilities); multiple residential dwelling facilities that include beauty parlors and hobby shops; and convalescent care facilities. In such cases, methods for enhanced pretreatment of the wastewater to remove toxic synthetic organic chemicals may be necessary before the wastewater is discharged to a SWAS.

It is vitally important to limit the infiltrative surface loading rate of pollutants that exert an oxygen demand to that which has been found by experience to enable the soil to sustain, on a long-term basis, the aerobic environment needed to degrade such pollutants. Failure to do this will result in overwhelming the soil's capacity to renovate the wastewater and cause contamination of the ground water. A discussion of surface loading rates is given in Section X of this document.

F. Nutrients

1. General

The accumulation of nutrients in lakes or other bodies of water results in a process called eutrophication. It can result from natural or anthropogenic processes. The critical nutrients are nitrogen and phosphorus (P). Eutrophication in fresh waters is typically caused by relatively low phosphate concentrations and results in depressed oxygen levels and excessive growth of plants and algae. Phosphorus is usually the limiting nutrient in eutrophication of freshwater bodies. Depressed oxygen levels may result in fish kills of desirable fish stock and their replacement with less desirable fish. Excessive growth of plants and algae can result in development of unsightly scum on the water surface and limit recreational uses. Decaying mats of dead plants and algae can produce foul tastes and odors. Phosphorus, like nitrogen, is critical for key life processes but, unlike nitrate-nitrogen, phosphates are not known to cause adverse health effects (U.S. EPA- 1975; Tyler, E.J., et. al.- 1977). Phosphates have been determined to be generally safe by the U.S. Food and Drug Administration.

Nitrogen is essential to the growth and reproduction of phytoplankton. In saline bays and estuaries, nitrogen is the limiting nutrient. In the presence of an over abundance of nitrogen, organisms such as algae (phytoplankton) and floating, submerged or emergent aquatic vegetation (macrophytes) can proliferate in these water bodies. This can accelerate the natural processes of eutrophication.

The death and decay of excessive algae results in oxygen depletion, a condition that is inimical to fish and other aquatic life that require oxygen to survive. The decay of organic sediment under anoxic conditions can also result in the release of ammonia, which can have a toxic effect on aquatic life, as discussed elsewhere herein. The adverse ecological effects of high nitrogen loads to The Long Island Sound, which stimulate phytoplankton blooms, leading to hypoxia (dissolved oxygen (DO) concentration of 3 mg/l or less), have been well documented (LISS–1990, CT DEP–1998).

Health problems can occur when water that contains nitrates in excess of 10 mg/l, (expressed as nitrate-nitrogen, or NO_3-N), is consumed by infants, either by direct ingestion, as a result of its use in preparing baby formulas, or to a fetus in a pregnant woman. Nitrate is reduced in the baby's body to nitrite. Nitrite is able to oxidize ferrous iron in hemoglobin to ferric iron and convert hemoglobin (the blood pigment that carries oxygen from the lungs to tissue) to methemoglobin that is incapable of carrying molecular oxygen to tissue. This condition, known as methemoglobinemia (infant cyanosis, or "blue baby disease"), can result in suffocation and is particularly toxic to infants less than three months old.

Methemoglobinemia can also occur in older children and adults if sufficient nitrate is ingested (Bitton and Gerba, 1994; Ammann-1995). Nitrite is also reputed to induce human gastric cancer (Lee, et al.- 1995). The U.S. EPA has established a maximum pollutant level (MCL) of 10 mg/l of nitrate, expressed as NO_3 –N, for drinking water supplies.

2. Nitrogen

Nitrogen is one of two most prominent nutrients in pretreated wastewater discharged to the ground water (the other being phosphorus), and its fate and transport in the soil/ground water regime is of considerable concern when designing OWRS. Concentrations of total nitrogen (TN) in septic tank effluent (STE) typically range from 40-80 mg/l or more, depending upon the source of the wastewater. Sources of wastewater containing higher percentages of toilet/urinal wastes (blackwater) than typical residential septic tank effluent can have much higher TN concentrations.

Most of the nitrogen in wastewater receiving pretreatment in a septic tank is in the form of the ammonium ion (NH_4), with some organic nitrogen, and sometimes trace-to-small amounts of nitrite (NO_2) and nitrate (NO_3) also present. In a conventional OWRS that has an aerobic soil zone beneath the SWAS, ammonium and organic nitrogen are rapidly converted to nitrate. Organic nitrogen must first be mineralized (converted to the inorganic form) by microbial action to ammonium, which takes place in the septic tank and in the biomat, before it is oxidized to nitrate by autotrophic bacteria in the aerobic unsaturated zone.

Where conditions are favorable, various processes remove some of the nitrate. The most significant of these processes is biological denitrification, the reduction of nitrates to nitrogen gas by the metabolic processes of facultative microbes under anoxic conditions. The gaseous nitrogen is then released into the atmosphere via the unsaturated soil pores in the unsaturated zone. However, where conditions are not favorable for the denitrification process (the usual case), nitrate, being very soluble and chemically inactive, may easily percolate down to, mingle, and move with the ground water to points of concern such as drinking water wells and surface waters. The presence of nitrates in both ground water and surface water in concentrations significantly greater than natural background levels can lead to environmental problems.

Up to 20 percent or more of the total nitrogen in raw domestic wastewater can be removed in the septic tank by sedimentation and microbial assimilation (Hardesty, 1974; Laak-1986; Pell and Nyberg-1989, Long-1995). However, where most of the nitrogen enters the septic tank in dissolved forms, as may be the case for wastewaters from schools and domestic wastewater from commercial establishments and industrial facilities (where urine may be the main contribution), the amount removed will be much lower, as sedimentation will not be a significant factor. The remaining nitrogen is discharged with the septic tank effluent to the SWAS.

The fate of the remaining nitrogen depends upon a number of mechanisms and processes, including mineralization, adsorption, plant uptake, volatilization, fixation, immobilization, nitrification and denitrification. These processes in turn depend upon such factors as; soil pH, temperature, moisture, oxidation-reduction (redox) potential, oxygen present in the soil gases, presence and type of organic matter in the soil, soil cation exchange capacity, and microbial populations. In a properly functioning SWAS, 15 to 25 percent of the nitrogen remaining in the septic tank effluent may be removed (Laak – 1986; Long, 1995; Wilhelm et al. – 1996; Crites & Tchobanoglous - 1998).

Mineralization of nitrogen, the conversion of organic nitrogen to inorganic nitrogen (predominantly ammonium) by biological action occurs both in the septic tank and at the biomat in the SWAS, and very little organic nitrogen is found in the wastewater after it flows through the biomat.

Adsorption of ammonium via soil cation exchange may play a role in nitrogen removal, but nitrogen so adsorbed is subject to subsequent desorption and leaching. In addition, eventually a state of equilibrium may become established as all of the cation exchange sites are occupied. When this occurs, desorbed ammonium is replaced with new ammonium cations, and no net removal of ammonium occurs (Magdoff et. al.-1974; Sikora and Corey-1976; Brown, et al.-1978; Brown et al.-1984). Thus, ammonia-nitrogen may remain in ground water that discharges to surface water bodies.

Ammonia is reported to be toxic to aquatic life at very low concentrations of less than 1 mg/L (Laak-1986; U.S.EPA-1993). The EPA criteria for ambient water quality, as well as modified-state criteria, give both maximum total and unionized (free) ammonia levels as a function of pH and temperature. The maximum one-hour average in-stream concentrations of un-ionized ammonia-nitrogen (NH_3) permissible in a three-year period are all under 1 mg/L. The maximum four-day average concentrations for the same are all under 0.1 mg/L (USEPA-1985; USEPA-1993).

The acute toxicity of NH_3 has been shown to increase as pH and temperature decrease. Thus, if nitrification does not occur due to the existence of anaerobic conditions beneath the SWAS, and small amounts of free ammonia persist in the ground water, an adverse effect on aquatic life could result where the ground water discharges to nearby surface waters.

Plant uptake of some of the nitrogen may occur, provided the [SWAS] is within the root zone of the plants, but the amount of nitrogen discharged to a [SWAS] greatly exceeds that which can normally be utilized by nearby plants (Sikora-1976). Plant uptake is usually visually evident from the distinctively greener grass that grows above a SWAS where the effluent can rise into the root zone. This situation may occur when a SWAS malfunctions and floods the surface or near surface, or when a normally operating SWAS has been constructed at a shallow depth below ground surface. However, most of the pretreated wastewater is discharged below the root zone of local vegetation; also, such uptake essentially ceases during the dormant season. Further, unless the vegetation is harvested, it is likely that N will be recycled to the soil as the vegetation decays during the dormant season.

Volatilization of ammonium is only significant at high pH values (≥ 9.5), which seldom exist in and beneath a SWAS. Fixation occurs when ammonium ions become trapped between intercellular layers of clay. Volatilization and fixation are not thought to be significant nitrogen removal processes (Lance-1972).

Immobilization occurs as the microorganisms engaged in removing organic matter incorporate nitrogen in their cells during synthesis reactions. This may account for five to ten percent, or less, of nitrogen removal (Lance –1972). Research has shown that nitrogen incorporated into microbes is held in a rather stable form (Laak–1986).

In a properly functioning SWAS, underlain by an ample depth of unsaturated aerobic soil, almost complete oxidation of ammonium to nitrate usually occurs within 30 - 60 cm (1-2 ft.) of unsaturated soil below the bottom of the leaching system due to the metabolic action of nitrifying bacteria. This usually occurs within a few hours of the exposure of ammonium to an aerobic soil environment (Anderson, et al.–1994; Duncan, et al.-1994; Long -1995). Ammonium is first oxidized to nitrites and the nitrites are subsequently oxidized rapidly to nitrates. If dissolved oxygen is present in the effluent when it reaches the water table, or if the background ground water contains appreciable dissolved oxygen, aerobic oxidation of ammonium may continue in the saturated zone (Wilhelm, et al.-1994).

It should be noted that the necessity for aerobic, unsaturated soil conditions requires that the SWAS not be installed too deeply into the soil, since the oxygen present in the unsaturated soil voids decreases rapidly with depth below ground surface. Below about 40 cm (16 in.) from the surface, the rate of oxygen diffusion decreases exponentially (Otis-1997). Likewise, the placement of a dense layer of soil or pavement above the [SSAS] will severely restrict the transfer of oxygen into the soil (Long-1995).

If conditions are favorable for denitrification (presence of a suitable carbon source, facultative heterotrophic bacteria, and anoxic or anaerobic conditions), some of the nitrate may be denitrified. Very little denitrification will take place in clean sands because of the lack of organic carbon. Some small amount of denitrification may take place in saturated micro-sites between the soil grains (Sextone et al. 1985; Long-1995) where the traces of dissolved organic carbon in the ground water may be sufficient to support the denitrification process. Crites and Tchobanoglous (1998) indicated that about 15% of the nitrate is denitrified in sandy, well-drained soils and 25% in heavier soils.

Nitrate removal from wastewater by denitrification is considered to be rare in aquifers below SWAS (Wilhelm et al.–1994) and most investigators have presumed that dilution by ground water is the predominant mechanism that significantly lowers the nitrate-nitrogen concentration in the ground water. Recent studies have indicated that, in some cases, dilution of nitrates (and other constituents of wastewater) below a SWAS may be much less than posited in previous decades (ibid.) Most investigators have indicated that, generally, any remaining nitrate in the percolate from a SWAS that has not been denitrified before it reaches the ground water will remain unaltered in chemical composition or concentration other than by dilution. In general, nitrate is found to be more mobile in soils with greater moisture content, greater hydraulic conductivity, coarser texture and greater structure.

On the other hand, there is evidence that substantial denitrification may take place where nitrate laden ground water flows through saturated soils with significant readily assimilable (labile) organic carbon content, such as those that exist in wetlands and beneath some water bodies (Robertson, et al. 1991; Korom-1992; Long-1995). Denitrification can also be caused by the action of certain autotrophic bacteria using reduced iron and sulfide as electron donors in the absence of organic carbon (Korom, ibid). However, current capabilities to predict an aquifer's denitrification characteristics are site specific at best (Korom-ibid.).

Nitrogen also reaches the ground water from other sources such as decomposing plants and animals, animal wastes, application of fertilizers for lawn care and agricultural purposes, bacterial action in soil, and direct deposition from the atmosphere.

3. Phosphorus

Phosphorus (P) is the other prominent nutrient in wastewater discharged to the ground water. Phosphorus occurs in natural waters and in wastewaters almost solely as phosphates (PO_4-P) (Standard Methods-1995). The principal sources of PO_4-P in domestic wastewater are human waste, food wastes, toothpaste, pharmaceuticals, detergents (particularly dishwashing detergents), and food-treating compounds. Phosphates in wastewater may include orthophosphates, condensed phosphates (polyphosphates) and organically bound phosphates.

Since in most cases ground water will eventually reach a surface water body, it is important that the phosphorus concentration in the percolate from a SWAS be reduced to background levels in the ground water prior to the ground water reaching a point of concern. Laak (1986) reported that natural ground waters contain 0.01-0.06 mg/l of PO_4-P, while Reneau, et al (1989) reported that PO_4-P concentrations in shallow groundwater are normally low, with values ranging from 0.005 mg/l to 0.1 mg/l. The PO_4-P concentration in ground water samples in a forested site in eastern Connecticut were found to range from 0.02 to 0.04 mg PO_4-P /L (Pietras-1981), while another study conducted in the Eastern Highlands and Central Lowlands of Connecticut found the mean concentration of PO_4-P in control wells installed up-gradient of on-site systems ranged from 0.02 to 0.06 mg/L (Luce and Welling-1983). U.S. EPA (1986) recommended that to control algal growth, total phosphorus (as P) should not exceed 0.05 mg/L if streams discharge directly to lakes or reservoirs, 0.025 mg/L within a lake or reservoir, and 0.1 mg/L in streams or flowing waters not discharging into lakes or reservoirs. Generally, surface waters maintained at 0.01 to 0.03 mg/L of total P tend to remain uncontaminated by algal blooms or excessive macrophytes.

Septic tank effluent (STE) contains total PO_4-P concentrations ranging between 3–20 mg/l depending upon the source of the wastewater. (The Soap and Detergent Association -1991; Wilhelm et al.-1994; Robertson, W. D. et al.-1998; Crites and Tchobanoglous -1998). Currently, raw sewage total PO_4-P concentrations are usually lower. This decrease has been due to declines in the PO_4-P content of powdered detergents, (in some cases due to regulation of the P content), a significant increase in consumer use of liquid laundry detergents, which do not contain P (Soap and Detergent Association- ibid.) and in some cases due to the regulation of the P content.

Because the phosphorus content of detergents is regulated in Connecticut, it can be expected that the total phosphorus concentration in Connecticut domestic wastewater would be in the mid-to-low end of the range given above. (In Connecticut, Sec. 22a-462 of the Connecticut General Statutes limits the phosphorus content in detergents. However, detergents used for medical, scientific or special engineering purposes or for use in machine dishwashers, dairy equipment, beverage equipment, food processing equipment and industrial cleaning equipment are exempt from this limitation.) Therefore, the estimated P content in the wastewater from such sources should be based on actual sampling and testing of the existing facility or, for new facilities, from facilities similar to that which is being proposed to be served.

Within a septic tank, most organic phosphorus and polyphosphates are converted to inorganic soluble orthophosphates during the anaerobic decomposition process that occurs in the tank, with some organic phosphorus also being present in particulate form (Sikora and Corey -1976). Some investigators indicated that approximately 20-30% or more of the total phosphorus in raw wastewater is removed via sludge that accumulates in the bottom of the tank (Bicki, et al.-1984; Pell and Nyberg -1989). On the other hand, other investigators have indicated a much lower percentage removal of P in the septic tank.

Orthophosphate (PO_4^{3-}) is the most stable configuration of phosphorus in the soil (USEPA-1977). Organic phosphates are much more mobile than inorganic phosphates and can move rapidly through soil systems. To remove organic phosphates from solution using a soil system, the organic phosphate must be physically filtered and held for sufficient time to allow decomposition of the phosphate compound to an inorganic form (U.S. EPA -1978). The conversion of organic P remaining in the septic tank effluent continues to a lesser degree in the biomat, where additional conversion of organic P to soluble orthophosphates takes place (Wilhelm, et al.-1994).

In addition to the contribution resulting from wastewater discharged into the ground, P reaches the ground water from other sources. Such sources include background P resulting from the natural erosion of rock containing P compounds, animal wastes, decay of vegetation, and application of fertilizers for lawn care and agricultural purposes. P in fertilizers is usually taken up in the upper layers of soil, and generally does not reach the ground water unless excess amounts are used on a long-term basis. Except in active livestock farming operations, animal wastes are a minor source of P in the ground water.

The soil's ability to remove phosphorus from the wastewater percolating from the SWAS is related to its sorptive capacity. The term "sorption" here refers to any process - physical, chemical, or biological-which causes phosphorus to be lost from the soil solution. It includes such processes as adsorption, absorption, chemisorption and precipitation by chemical reaction.

Some biological uptake of phosphorus by soil microorganisms and plants can also occur. However, the phosphorus uptake by microorganisms is small compared to the amount of phosphorus in domestic wastewater. The phosphorus uptake by plants can be significant during the growing season provided the SWAS is installed within the root zones of the plants; however, most systems are installed below that zone. Further, if the plant growth is not harvested and removed from the site, it is likely that the phosphorus will be recycled to the soil as the plant growth decays during the dormant season. Therefore, biological uptake of phosphorus-laden wastewater discharged to the subsurface is not likely to be a significant method of phosphorus immobilization.

The fate and transport of P in wastewaters applied to soils has been extensively studied. The ability of a soil to remove wastewater P from solutions passing through the soil matrix is primarily related to the formation of relatively insoluble phosphate (PO_4) compounds of iron, aluminum and calcium (U.S. EPA -1975).

Most P in orthophosphate form is readily adsorbed to soils that contain reactive iron (Fe) or aluminum (Al) or calcium carbonates (CaCO3). Reactive iron and aluminum surfaces can occur at the broken edges of crystalline clay minerals, as surface coatings of oxides and hydroxides and of amorphous silicates. Aluminum in the form of positively charged hydroxide polymers, and as an exchangeable ion in acid soils can also adsorb phosphorus. Reactive calcium surfaces are mainly found on solid calcium carbonates and calcium-magnesium carbonates. Organic or mineral soils with minor amounts of Fe and Al show minimal P sorption capacity. In basic soils, calcium phosphates control the P in solution. Calcium compounds predominate above pH 6 to 7, and iron and aluminum compounds predominate below pH 6 to 7. P in orthophosphate form can also combine with silicate materials (compounds containing silicon, oxygen, and one or more metals).

Therefore, in most soils in which Fe and Al are present in a reactive form, or $CaCO_3$ is present, and flow rates are minimal, phosphorus (P) movement is minimal and concerns of pollution to ground or surface waters from P applied via an SWAS are unfounded. However, pollution of ground water with P can occur in cases where: a.) water tables are shallow; b) soils are coarse textured; c.) flow rates are increased due to strong soil structure; d.) loading rates are high; e.) soils have a low P sorption capacity; f.) the capacity of a soil to immobilize P has been met (VA Dept. of Health -1990).

A major factor contributing to P movement in soil is the flow rate. Increased flow rates are generally associated with soils of coarse texture, strong structure or macro voids associated with biological activity (which provide preferential flow pathways), or high water tables. Thus there is less opportunity for P to be adsorbed to the soil particles and then precipitate with the Fe, Al, or $CaCO_3$ present in the soil.

Phosphorus that has previously been adsorbed can be desorbed under certain conditions. Desorption of P is always much slower than adsorption but the effect can be considerable when anaerobic conditions exist, organic ions are present, or when the concentration of P in the soil solution is very dilute (VA Dept. of Health - 1990).

Following adsorption, P may precipitate to a separate mineral form. Precipitation reactions occur with soluble iron, aluminum and calcium. Particles of phosphate compounds can also form by separation of adsorbed phosphorus along with iron, aluminum or calcium from solid surfaces (U.S. EPA - 1977).

While precipitation reactions are usually considered permanent, they are reversible, particularly if significant changes should occur in pH, oxidation-reduction (redox) potential, soil solution composition of aluminum, iron or calcium carbonate, or ionic strength (Gold and Sims -2002) or when saturated conditions are present.

Phosphorus removals in the subsurface soils can range from 70 to 99%, depending on the physical and chemical characteristics of the soil. Therefore, the mass and characteristics of soil in contact with the wastewater limit the long-term capacity for phosphorus removal. Removals are related also to the residence time of the wastewater, the travel distance, climatic and other environmental conditions (U.S. EPA-1981).

In the past, it had been assumed that while it was possible that P could move through the soil and reach the ground water, this was not a major concern since P could be easily retained in the soils underlying the SWAS due to chemical changes and adsorption. Many studies that monitored P removal from wastewater discharged to a SWAS documented high P removal in close distance from the SWAS. However, concern had been expressed that this is not necessarily true with respect to coarse-grained soils with low clay and reactive metal content, high pH, and with continually saturated anaerobic soils (Hill and Sawhney-1981). In recent times, the expectations for some environmental impact by P in the ground water have become much stronger. Fairly recent studies have confirmed that P is in fact mobile in such soils and, under such conditions, can travel significant distances (tens of meters) over a period of one or more decades (Wilhelm, et al- 1996; Robertson, et al.-1998; Robertson and Harman-1999).

In numerous studies, P derived from an SWAS has been detected above background levels in ground water adjacent to the SWAS under conditions of saturated flow, due to high water tables or high hydraulic loading rates. However, P concentration in ground water is found to decrease with distance from a SWAS, because P is also capable of undergoing sorption and precipitation in the ground water zone. One study found that passage of effluent through approximately 26-ft (8-m) of somewhat poor to poorly drained soil under water-unsaturated conditions resulted in a 99% reduction in the P concentration of the ground water. However, under conditions of saturated flow, passage of effluent through 78-ft (30-m) of soil was required before comparable reductions were noted. Thus, P removal in saturated soil is much slower than in unsaturated soil (Reneau-1979).

There are a number of factors that can effect the fate and transport of P in a soil. Some of these factors are beyond the control of a designer of an SWAS, while others can be controlled in order to optimize the immobilization of P in the soil.

Factors that effect the fate and transport of phosphorus in a soil include:

- Physical characteristics of the soil
- Chemical characteristics of the soil
- Temperature of the soil
- Chemical properties of the wastewater
- **Rate of application of P to the soil**
- **Contact time of P solution with the soil**
- **Thickness of unsaturated zone**
- **Degree of oxidation of wastewater reaching the soil**
- **Oxygen levels in the subsurface**
- **Travel distance in the saturated soil**
- **Repeated wetting and drying of the soil.**

For a given site, those factors that can be controlled are printed in bold lettering in the listing given above. Also, where fill systems are used, the designer also has some limited control over the physical and chemical characteristics of the soil. Examples of methods to control such factors are given on the following page.

Limiting the rate at which P is applied to the soil involves adjusting the infiltrative surface P loading rate to that of the soil's long-term P sorption rate. Limiting the infiltrative surface hydraulic loading rate can control the contact time in the unsaturated zone. Constructing an SWAS at a sufficient height above the mounded seasonally high ground water table can control the thickness of the unsaturated zone. The degree of oxidation of the effluent reaching the soil can be controlled by limiting the infiltrative surface organic loading rate and, if necessary, by providing enhanced pretreatment of the wastewater.

Installing the SWAS at a shallow depth below the finished ground surface, avoiding placement of an impervious surface over the SWAS, and limiting the hydraulic loading to a small fraction of the soil's hydraulic conductivity will permit atmospheric oxygen to reach the subsurface via the unsaturated soil pores. Providing sufficient horizontal separating distance between the SWAS and a point of concern can control the travel distance in the saturated soil and thus increase the contact time of P with the reactive soil particles. Where fill systems are used, fill material can be chosen with respect to both its physical (texture) and its chemical characteristics (P sorption capacity) as well as its hydraulic capacity. [Note that many of the factors that effect the fate and transport of P also effect the fate and transport of other pollutants contained in the wastewater discharged to a SWAS, such as organics, nitrogen, pathogens and toxic metals.]

In order to complete the design of a SWAS, it is necessary to know the amount of phosphorus that can be sorbed to the soil particles, and the volume of soil available for sorption. The phosphorus sorption capacity of a soil is typically determined by the use of batch equilibrium experiments. Samples of the soil are added to solutions containing known concentrations of phosphorus. After the soil is mixed into the solution and allowed to come into equilibrium for a period of time (up to several days), the solution is filtered and the filtrate is tested for phosphorus. The difference between the initial and final solution concentrations is the amount adsorbed for a given time. The data obtained from these experiments is used to generate adsorption isotherms. These isotherms are plots of the amount of P adsorbed by a soil vs. the P concentration in each solution. The phosphorus sorption capacity thus determined is usually expressed as mg P removed/100 g of soil.

For example, Sawhney and Hill (1975) determined the P sorption capacity of six major soils in Connecticut from sorption isotherms based on short term (36-48 hours) laboratory P sorption experiments on soil samples obtained from the B2(see appendix for definition) horizons. The results were as follows:

Soil Name	Parent Material	Sorption Capacity, mg/100g Soil
Merrimac	Sandy, gravelly terraces	9.0
Stockbridge	Firm limestone till	14.5
Buxton	Lacustrine silts and clays	20.0
Charlton	Loose till of granite and gneiss	21.8
Cheshire	Loose till of Triassic sandstone and shale	27.5
Paxton	Compact till of gneiss and schist	29.0

Based on analysis of samples from operating drain fields, Sawhney and Hill (ibid.) concluded that most P in wastewater is sorbed within a shorter distance from the point of application than simple laboratory sorption experiments would indicate. They opined that P sorption sites regenerated with time. Regeneration was confirmed in further laboratory experiments. Soils that had been successively treated with P solution showed reduced P sorption capacity but regained the capacity to sorb P after drying and wetting cycles. Thus they stated that phosphate sorption capacities of soils are greater than simple laboratory experiments indicate.

[N.B. The soils used in the tests were those from the B-horizons, which usually contain the most reactive soil particles and minerals. Where a SWAS is located below the B-horizon, it is probable that the P sorption capacity will be considerably less than if it had been located in the B-horizon.]

Important considerations in phosphorus removal are slow reactions between phosphorus and cations present in the soil that may “free up” previously used adsorption sites for additional phosphorus retention. However, these slow reactions are not modeled by isotherm experiments. The slow reactions involve the formation of precipitates of limited solubility and the regeneration of adsorptive surfaces that are then available to adsorb additional phosphate (U.S. EPA-1977). Therefore, the use of adsorption isotherms as a measure of the phosphorus immobilization capacity of a soil significantly underestimates that capacity. Actual phosphorus retention may be from 2 to 5 times the value obtained from analysis of the adsorption isotherms (Tofflemire and Chen-1977). Thus, the use of the adsorption capacity determined from isotherm experiments will be conservative. However, the use of the isotherm adsorption capacity will provide a margin of safety that is warranted, given the heterogeneity encountered in most Connecticut soil deposits and the recent concerns with respect to P mobility by various investigators. (Hill and Sawhney-1981; Wilhelm, et al- 1996; Robertson, et al.-1998; Robertson and Harman-1999).

G. Pathogens

Pathogens contained in domestic wastewater include various bacteria, parasitic protozoa, parasitic worms (helminths) and viruses. These pathogens can cause infection and disease in humans, are excreted from the human body, and are contained in domestic wastewater discharged to the subsurface via a septic tank and SWAS. Virtually all of these pathogens, with the exception of some helminths, are microscopic in size and cannot be seen without the aid of either an optical microscope or an electron microscope. Accordingly, pathogens are commonly referred to as pathogenic microorganisms.

The U.S. EPA indicates that over one-half of the waterborne disease outbreaks in the United States are due to the consumption of contaminated ground water. While some of these outbreaks are caused by chemical contamination, most are caused by consumption of groundwater that has been contaminated due to the presence of pathogenic microorganisms in domestic wastewater that has been discharged onto or into the soil.

Reported occurrences of drinking-water related waterborne disease outbreaks in the U.S. during 1999 and 2000 more than doubled over the previous two-year reporting period, mostly among untreated private wells, according to the Centers for Disease Control and Prevention (CDC-2002). The CDC report shows that 25 states reported 39 outbreaks that sickened 2068 people and killed two. All but two of the outbreaks were linked to microbial pathogens. Eleven were associated with community water systems, 11 with non-community systems, and 17 with private supplies. Of the community systems, five were attributed to treatment problems, five to the distribution system, and one to untreated ground water. While no outbreaks were reported in Connecticut, the potential for such outbreaks cannot be disregarded.

All pathogenic microorganisms, except for the viruses, are living organisms. They can metabolize substances for food and energy, require water to exist, and can reproduce. As is the case for all living organisms, they eventually die. However, viruses are different. Outside of a host cell, they are inert, have no metabolism, cannot reproduce or move about on their own. When adverse conditions occur which result in destruction of the viruses, they do not “die” but become inactivated, and thus are no longer viable and able to infect a living organism. Therefore, the terms inactivate, and inactivation, refer to the destruction or decay of viruses, whereas the terms die, die-off, or death ordinarily refer to the living pathogens.

Viruses in ground water are of major concern to public health and environmental agencies. This is because of their ability to survive for long periods of time in the subsurface and still remain infectious, and the very small number (as little as one virulent particle, in some cases) thought to cause infection and disease.

Inactivation of viruses contained in domestic wastewater occurs primarily in the biomat that forms at the SWAS-soil infiltrative surfaces, and in the unsaturated soil zone beneath the SWAS. There are a number of factors believed to cause such inactivation. These include climate, the considerable heterogeneity of the soil physicochemical characteristics, the natural processes that tend to remove or degrade pathogens as they travel through the subsurface, and the nature of the pathogen, including its physical, chemical and biological characteristics. However, once viruses reach the ground water, the only factor that has been found to significantly correlate with virus inactivation is temperature. The higher the ground water temperature, the greater the rate of inactivation. Under cold temperature conditions, viruses are known to survive long periods of time (many months and in some cases, years). This is not to say that temperature is the only factor that effects virus survival. Many of the same factors that cause inactivation of viruses in the unsaturated zone are also thought to cause inactivation of viruses after they reach the ground water. However, it has proven difficult to obtain a significant statistical correlation for such factors. Therefore, the travel times set forth in the Design Standards were selected based on the prevailing ground water temperatures in Connecticut, with no allowance for other factors.

Factors that control the adsorption and deactivation of viruses are listed in Table 1, with those factors over which some control can be exercised in design of a SWAS are shown in bold lettering.

In addition to those factors shown in bold typeface in Table 1, factors such as soil texture, soil structure and soil organic content can be controlled to some degree by prohibiting or placing severe restrictions on the installation of a SWAS in certain soils (i.e.: extremely coarse soils, poorly drained to very poorly drained soils, massively structured soils, etc.).

The techniques shown in Table 2 appear to be the most viable means of influencing the adsorption and inactivation of viruses and the die-off of other pathogenic microorganisms. All of the techniques listed in Table 2 interact in attaining the desired result, attenuation of pathogens contained in septic tank effluent before they reach a point of concern. Failure to consider any one technique may have an adverse effect on the desired result regardless of how well the other techniques are applied.

TABLE 1

FACTORS THAT INFLUENCE ADSORPTION AND INACTIVATION OF VIRUSES

Factors Influencing Adsorption	Factors Influencing Inactivation
Climate (Temperature, Precipitation)	Climate (Temperature, Precipitation)
pH	pH
Soil Texture and Structure	Soil Characteristics
% Clay Content and Clay Mineral Species	Antagonism from Soil Microflora
Infiltrative Surface Loading Rate	**Adsorption/Desorption**
Soil Moisture (Saturated vs. Unsaturated Flow)	**Moisture** (Dessication)
Soil Gas Oxygen Content	**Soil Gas Oxygen Content**
Depth to seasonal high ground water table	**Time** (Travel Time)
Species and Strain of Virus	Species and Strain of Virus
Soil Organic Content	Other natural processes
Cation Exchange Capacity	
Soil Minerals (multivalent cations)	
Ionic Composition of Soil Solution	
Attractive and repulsive forces between soil particles and viruses	
Soil Macropores	

Sources: U.S. EPA-1978; Sobsey et al. -1980; U.S. EPA -1981; -Gerba and Bitton-1984, Newby, et al.-2000.

TABLE 2

TECHNIQUES FOR CONTROLLING FACTORS THAT INFLUENCE THE FATE AND TRANSPORT OF PATHOGENS IN THE UNSATURATED ZONE.

Technique	Factors
1) Provide Additional Pretreatment	Soil gas oxygen content, aerobic soil, pH, microflora antagonism, virus inactivation.
2) Minimize Depth of SWAS below Ground	Soil gas oxygen content, aerobic soil, microflora antagonism, adsorption.
3) Provide Uniform Flow Distribution	Soil moisture, adsorption.
4) Select proper Infiltrative Surface Hydraulic and Organic Loading Rates	Adsorption, aerobic unsaturated soil, soil moisture.
5) Select proper Linear Hydraulic Loading Rate	Depth of aerobic unsaturated zone, adsorption.
6) Allow for effects of seasonal high ground water table and ground water mounding	Depth of aerobic unsaturated zone, adsorption.
7) Provide adequate Vertical Separating Distance	Depth of aerobic unsaturated zone, adsorption.
8) Provide adequate Horizontal Separating Distance	Travel Time.
9) Provide proper construction methodology	All of the above.

A brief discussion of these factors follows. Detailed discussion on addressing these factors when designing a SWAS will be given in later sections of this document.

Additional Pretreatment

Additional (enhanced) pretreatment, other than by a septic tank, can provide some additional attenuation of pathogens above that provided by a septic tank. Additional pretreatment is often used for reduction of the organic and solids concentrations in the pretreated effluent discharged to a SWAS. Investigators have shown that additional pretreatment beyond that provided by a septic tank will substantially increase the rate of infiltration into the soil surrounding the SWAS by reducing the clogging effect of the biomat. It can also help to maintain aerobic conditions in the unsaturated zone by reducing the oxygen demand caused by the metabolic processes of soil microorganisms that utilize the organic matter as a source of food and energy. This will result in fostering the growth of aerobic soil microflora that will have antagonistic effects on viruses. If disinfection is provided following the additional pretreatment, pathogens can be greatly attenuated.

Depth Below Ground Surface

The depth below ground surface at which a SWAS is installed is important, as it will effect the ability of the unsaturated soil to remain aerobic. (Hurst, et. al.-1980; Lance-1982; Bicki, et al.-1984; Cogger-1989; Otis, et al.-1993; Toze-1997; Cardona-1998)

The transport of oxygen from the atmosphere through the soil and eventually to the onsite system occurs as a result of molecular diffusion, which is a response to the concentration gradient formed by the oxygen rich atmosphere and the oxygen-poor wastewater infiltration site. The oxygen concentration gradient and the distance between the ground surface and the onsite wastewater infiltration site effect the flow rate of oxygen into the system. As the distance between the ground surface and the infiltration site increases, the concentration gradient decreases. The greater the distance, the longer the path the oxygen must travel, causing a decrease in gradient and thus a decrease in the mass flux of oxygen (Erickson and Tyler-2001).

It should also be noted that while oxygen is diffusing downward through the open (non-water filled) soil pores there will be a concurrent diffusion of other gases (e.g. carbon dioxide), released during the metabolism processes of soil microorganisms, in the opposite direction toward the atmosphere. If any denitrification is occurring beneath the SWAS, nitrogen gases are also diffusing upward toward the atmosphere. If anaerobic microsites exist in an otherwise aerobic zone, methane gas (CH_4) as well as carbon dioxide (CO_2) may also be released. (CO_2 is also released during the aerobic oxidation of organic matter, but some of the CO_2 will combine with water to form carbonic acid.) Therefore, it is vitally important that a sufficient amount of open pores remain available so as not to hinder the diffusion processes.

Uniform Flow distribution

The basic objective of flow distribution is to uniformly distribute the septic tank effluent to the infiltrative surfaces of the leaching system so as to maximize the volumetric renovative capacity of the soil. However, there is considerable debate as to whether the distribution should be by means of gravity flow to the various units of the leaching system or by means of a pressure distribution system (PDS). In the latter case, this would require the use of septic tank effluent pumping stations or dosing siphons. The arguments on both sides of this issue appear persuasive. The use of pressure distribution for individual residential subsurface soil absorption systems is arguable because of problems resulting from probable lack of maintenance by individual residence property owners. However, for large systems where the SWAS is extensive and system maintenance is required as part of the permit issued for such systems, pressure distribution may be warranted.

Infiltrative Surface Organic and Hydraulic Loading Rates

- Using an infiltrative surface organic loading rate that will result in a pollutant load applied to the soil in excess of its volumetric capacity to remove or attenuate the pollutants can result in unfavorable conditions developing in and below the SWAS. These unfavorable conditions include depletion of the oxygen in the unsaturated soil beneath the SWAS, and severe clogging of the infiltrative surface. Depletion of the soil oxygen can have a significant adverse effect on the ability of the soil's defense mechanisms to remove or attenuate chemical pollutants and remove or inactivate pathogens. Severe clogging of the infiltrative surface can result in overt failure of the system manifested by wastewater backing up into the facility sanitary drain piping or surfacing of septic tank effluent. A high infiltrative surface hydraulic loading rate can also result in the percolate from the SWAS short-circuiting through large micropores or macropores in the soil, thus reducing the contact time and contact with the soil surface area. This will result in severely reducing the soil's capability for removing and inactivating pollutants. The adverse effects resulting from excessive organic and hydraulic loading rates can be largely avoided by limiting the loading rate to a very small fraction of the soil's saturated hydraulic conductivity. (Siegrist, et al –2000)

Linear loading rates

A high linear loading rate can result in significant mounding of the ground water beneath and immediately down-gradient of a SWAS and may result in the seasonally high ground water table reducing the depth of an unsaturated zone otherwise assumed to be adequate.

Determination of seasonally high ground water elevations

Determination of the seasonal high ground water table (SHWT) is of paramount importance in the design of a SWAS, as it is the basis for establishing an adequate vertical separating distance. Underestimating the height of the SHWT can render an otherwise conservatively selected vertical separating distance ineffective for removal or attenuation of pollutants.

Vertical separating distances

When pretreated domestic wastewater is discharged to the subsurface via a SWAS, it tends to cause a local rise (mounding) of the ground water. This mounding may vary from a very few inches to several feet or more, depending upon the rate of effluent discharge, the configuration of the SWAS, and the characteristics of the subsurface soils. If this mounding is not taken into account, the vertical separating distance may be significantly overestimated. This is particularly important with respect to the large SWAS, and those soils that have restricted hydraulic capabilities. If insufficient vertical separating distance occurs because of failure to take ground water mounding into effect, the result will be a decrease in the volume of unsaturated, aerobic soil available to renovate the wastewater.

Horizontal separating distances

Adequate horizontal separating distance between a SWAS and a point of concern enables any residual pollutants reaching the ground water to be removed or attenuated to the level where the risk to the public health or the environment from the use of the ground water and associated surface waters is acceptable.

Construction methodology

Even the most conservative and meticulous design of a SWAS may not serve to protect the system from failure, or the ground water from contamination, if the implementation of that design during construction is faulty. Proper construction methodology begins before the contractor arrives on the site to install the system by initiation of quality control procedures. This involves such efforts as a review of the contractor's qualifications, a pre-construction meeting to discuss the contractor's duties and obligations, the role of the system designer's field representative, scheduling of the work, and submission and review of shop drawings. Quality control for the project continues until final inspection and approval by the designer and the regulatory agency having jurisdiction.

H. Summary

Natural and synthetic organic chemicals (both dissolved and suspended), nitrogen, and phosphorus are the chemical constituents of domestic wastewater that are of primary concern with respect to contamination of ground water. Heavy metals may also be present, but in very low concentrations that are usually immobilized in the soil. Domestic wastewater contains pathogenic microorganisms, including bacteria, parasitic protozoa, parasitic worms, and viruses. While it is important to consider all of the pathogens, the most important are the viruses that can cause infection and disease in humans. Experience has shown that viruses can exist for long periods of time in the subsurface, particularly under low temperature conditions, and still remain viable long after bacteria and parasites have died off.

When pretreated domestic wastewater (e.g. septic tank effluent) is discharged to the subsurface via a properly designed SWAS, it is renovated as it travels through the biomat that develops at the soil interface with the SWAS. It is further renovated as it travels through aerobic unsaturated soils beneath the SWAS and eventually reaches and commingles with the ground water, and travels to a point of concern, such as a drinking water supply well. The ground water in turn is eventually extracted via wells for various water supply purposes, including drinking water, or discharges to surface waters that are used for many purposes. In this manner, renovated wastewater may be recycled for use many times. Therefore, the chief objective for design, construction, operation and maintenance of a SWAS and its associated pretreatment facilities must be to renovate the wastewater so as to protect the public health and the environment. It is axiomatic that the pretreated wastewater must remain in the soil for a suitable time to permit such renovation to take place.

For the most efficient use of the soil as a wastewater treatment medium, site evaluators and designers need to learn to identify features of soil properties such as those discussed and take them into account when designing soil treatment systems. Soil horizons need to be understood from the perspective of their ability to remove undesirable constituents in wastewater and then system depth, loading rate and method of distribution designed to take maximum advantage of the soil properties that will contribute to achieving the desired level of treatment (Mokma, et al-2001). For example where nutrient removal, particularly P, is of concern, distribution of effluent in the shallow portion of the profile will result in its passage through the most reactive portion of the profile for P removal.

In summary, renovation of wastewater by the soil depends upon the soil's renovative capacity to:

- Allow for organic matter (both natural and synthetic) to be decomposed or mineralized into relatively innocuous substances by indigenous soil microflora.

- Immobilize toxic metals by sorption onto reactive soil particles.

- Immobilize phosphorus by adsorption onto reactive minerals in the soil and precipitation with metal oxides and calcium compounds.

- Immobilize pathogenic microorganisms by filtering and/or by adsorption to soil particles until they ultimately die or become inactivated by antagonistic soil microflora, dessication, and other adverse environmental effects.

- Oxidize ammonia-nitrogen compounds to nitrates in aerobic soil zones and remove some of the nitrates by denitrification in anaerobic microsites and/or anaerobic soils located beneath and downgradient of the aerobic zones. (Dilution of nitrates in the ground water beneath and downgradient of an SWAS is also a factor, but may not always be sufficient to reduce the nitrate concentration to acceptable values.)

To ensure proper functioning of an OWRS:

1. The hydraulic, organic and nutrient loadings on the SWAS should be such that the finite, long term, capacity of the soil to accomplish wastewater renovation is not exceeded.

2. There must be sufficient vertical separation between the bottom of a SWAS and the water table so that an adequate depth of unsaturated aerobic soil will be maintained at all times, including during seasonal high ground water conditions.

   a. Unsaturated flow beneath a SWAS is important in ensuring slow travel and thus long residence time for pathogenic microorganisms in the unsaturated zone, good aeration, increased opportunity for contact between effluent and soil particles, opportunity for adsorption of pathogenic microorganisms to soil particles, and eventually die-off or inactivation of these pathogens.

   b. The same conditions resulting from unsaturated flow and the existence of an aerobic environment beneath a SWAS discussed in a. above are also very important for removal of organic compounds, immobilization of phosphorus and heavy metals. Such conditions are also required for conversion (oxidation) of ammonium-nitrogen to nitrates.

The facts discussed time and time again in the literature regarding removal of the most significant pollutants from domestic wastewater discharged to the subsurface are:

- The subsoil horizons that are the most active with respect to wastewater renovation are those that are aerobic, unsaturated, and have been enriched by materials (organic material, clay, iron and aluminum oxides, and calcium carbonates) that have been leached (eluviated) from the overlying topsoil and subsurface soil.

- Most of the wastewater renovation processes occur at the biomat that forms at the SWAS interface with the surrounding soil and in the unsaturated aerobic soil zone beyond the biomat.

- The pollutant loading must not exceed the finite capacity of the soils to effect pollutant attenuation and/or removal. The capacity of the soil should be estimated for each pollutant of interest, and the design of a SWAS should be based on the limiting capacity. (e. g.: Using an infiltrative surface organic loading rate that will result in a pollutant load applied to the soil in excess of the soil's volumetric capacity to remove or attenuate the pollutants can result in unfavorable conditions developing in and below the SWAS. These unfavorable conditions include depletion of the oxygen in the unsaturated soil beneath the SWAS, and severe clogging of the infiltrative surface.)

- The hydraulic loading must not exceed the soil-ground water hydraulic capacity for assimilating the wastewater discharged to the SWAS without surfacing before renovation is complete.

- It is vitally important to maintain an aerobic, unsaturated soil zone of sufficient depth below the SWAS by providing sufficient vertical separating distance between the bottom of the SWAS and the seasonal high ground water table.

- The infiltrative surface hydraulic loading rate must be controlled so as to provide for a slow rate of travel of wastewater through the unsaturated soil zone that will assure an intimate contact of wastewater with soil particles and mitigate against rapid flow through preferential pathways. This is accomplished by limiting the loading rate to a very small fraction of the soil's saturated hydraulic conductivity.

- Adequate horizontal separating distance between the SWAS and a point of concern must be provided to enable any residual pollutants reaching the ground water to be removed or attenuated to the level where the risk to the public health or the environment from the use of the ground water and associated surface waters is acceptable.

These facts should be kept firmly in mind when reviewing the information in the following sections of this document that discuss design, operation, and maintenance of on-site wastewater renovation systems.

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SECTION III ESTIMATING WASTEWATER FLOWS

TABLE OF CONTENTS

TABLES

TABLE OF CONTENTS

TABLES, Continued

SECTION III DATA FOR ESTIMATING WASTEWATER FLOWS

A. Introduction

One of the first tasks that need to be completed before beginning actual design of an OWRS is predicting the wastewater flows and characteristics (physical and biochemical) that will be generated by the facilities to be served by the system. This section addresses wastewater flows, and Section IV addresses wastewater characteristics.

The unit flow allowances given in the Design Standards will govern the prediction of wastewater flows. Where the Design Standards do not prescribe unit flow allowances for a particular type of wastewater generating facility, the data given in this section may be found useful in assisting the development of such allowances.

This section is based on published information on wastewater flows that has become available since publication of Healy and May (1982). Also included herein is information on wastewater flows gleaned from the engineering reports and Discharge Monitoring Reports (DMRs) in the files of the Department for large-scale on-site wastewater renovation systems in Connecticut. In addition, information on wastewater flows contained in Healy and May (sic) is included in Appendix A and similar information in the CT DPH Technical Standards for Subsurface Sewage Disposal Systems is included in Appendix G.

The historic data given herein, that predates the most current data, will provide a perspective on changes in water use brought about by water saving fixtures and may be helpful in developing unit flow allowances where an existing OWRS is to be remediated and the existing water using fixtures will not be upgraded to the newer low flow fixtures.

B. Water Use vs. Wastewater Flows

In many cases the wastewater flows determined for individual buildings are normally based on metered water use, rather than on measured wastewater flows, because of the difficulty in accurately metering small wastewater flow rates. It is normally assumed that almost all of the metered water use inside the building is converted to wastewater discharged from the building because very little of the water used is consumed. That assumption is supported by the following publications.

Linaweaver and Wolfe (1963) stated that: “ In the absence of more accurate data it is suggested that approximately 6 per cent of the water supplied for indoor use is not returned into the domestic sanitary sewer system. Table 2 in Chapter 11 of the National Handbook of Recommended Methods for Water Data Acquisition (USGS-19) indicates that domestic consumptive use amounted to about 2-3 percent of total indoor average annual use. Thus the water supplied for indoor use that is discharged as wastewater ranges from 94-98%.

Thus, it is not overly conservative to consider indoor domestic water use as equivalent to domestic wastewater discharge, absent any significant use of water for cooling or other purposes where the water is not discharged to the building sanitary drains.

C. Effect of Efficient Water Using Fixtures on Wastewater Flows

Perhaps the single most significant effect on water use and wastewater flows has been that of the relatively recent adoption of water conservation regulations by the Federal government and many of the states. Prior to about 1980, water use rates for the various fixtures were as follows (Fagan, D. April 1998): lavatories, 3.0 gal. per minute; sinks, 4.5 gal per minute; showers, 5.0 gal per minute; and water consumption for water closets, 4 to 7 gallons per flush.

In the late 70s, toilet manufacturers began introducing "water-saver" designs using 3.5 to 4 gal. per flush. By the early 1980s, most American plumbing codes had been revised to accept and ultimately require the installation of "water saver" toilets in new residential construction. The 1984 Building Officials and Code Administrators International (BOCA) Plumbing Code required that showers, lavatories, and sinks flow at not more than 3.0 gal per minute but no mention was made of a limit to water closet or urinal consumption (Osann, E.R. and J.E.Young 1998). However, the 1987 edition of that code specified that water closet and urinal water use rates should not exceed 4.0 gal and 1.5 gal per flush respectively.

One comparison of domestic water use before and after introduction of water saving fixtures is given in Table 2 of Chapter 11-Water Use, in the U.S. Geological Survey, National Handbook of Recommended Methods for Water Data Acquisition. Information abstracted from that table is given in the following Table R-1. This table shows that a 20% reduction in domestic water use below the pre-1980 use occurred where buildings were equipped with post 1980 fixtures.

TABLE R-1

PRE-1980 AND POST-1980 AVERAGE ANNUAL RESIDENTIAL INDOOR WATER USE

Activity	Pre-1980 Fixtures			Post 1980 Fixtures		
	Water Use, gpcd	Consumptive Use, gpcd	Consumptive Use, Percent	Water Use, gpcd	Consumptive Use, gpcd	Consumptive Use, Percent
Flushing	20	0	0	14	0	0
Bathing	28	0.5	2	19	0.4	0
Clothes Washing	14	1.0	7	14	1.0	7
Dish Washing	3	0	0	3	0	0
Other(Cooking& Cleaning)	10	0.5	5	8	0.4	5
Leaks	8	0	0	8	0	0
TOTAL gpcd	83	2	2	66	1.8	3

In 1990, the following minimum efficiency standards for plumbing fixtures and other water-saving devices were established in the Connecticut General Statutes (CGS Sec. 21a-86a and Sec. 21a-86b):

After October 1, 1990:

- Showerheads 2.5 gpm
- urinals 1.0 gal/flush
- bathroom sinks, lavatory and kitchen faucets and replacement aerators 2.5 gpm
- * lavatories in restrooms of public facilities shall be equipped with outlet devices which limit the flow to a maximum of 0.5 gpm

<u>After January 1, 1992:</u>

tank type toilets, flushometer-valve toilets, flushometer-tank toilets and electromechanical hydraulic toilets -1.6 gal per flush .

In 1992, Congress passed and the President signed the Energy Policy and Conservation Act, in part to "promote the conservation and the efficient use of energy and water." The Act established the following national water conservation standards for:

Showerheads -	2.5 gallons per minute
Toilets -	1.6 gal per flush
Faucets -	2.5 gal per minute
Urinals -	1.0 gal per flush

The Energy Policy and Conservation Act required that, effective January 1, 1994, all new toilets produced for home use must operate on 1.6 gallons per flush or less (Shepard, 1993). Toilets that operate on 3.5 gallons per flush were allowed to continue being manufactured, but their use would only be allowed for certain commercial applications through January l, 1997.

In 1993, the BOCA Plumbing Code maximum water consumption requirements were modified as follows:

Water Closet	1.6 gpf cycle (except as noted below)
Urinal	1.0 gpf cycle (except as noted below)
Shower head	2.5 gpm at 80 psi
Lavatory, Private	2.2 gpm at 80 psi
Lavatory, Public	0.5 gpm at 60 psi
Lavatory, public, metering or self closing	0.25 gal per metering cycle
Sink faucet	2.2 gpm at 60 psi

*Gpm = gal per minute; gpf = gal per flush.

A maximum water consumption of 4 gal per flushing cycle for toilets using Blowout Design fixtures, and 1.5 gal per flushing cycle for urinals was permitted under the BOCA code in the following cases:

- Fixtures for public use in theaters, night-clubs, restaurants, halls, museums,
- Fixtures provided for patients and residents in hospitals, nursing homes, sanitariums and similar occupancies.
- Fixtures provided for inmates and residents in prisons, asylums, reformatories and similar occupancies."

Thus, all new housing units now required to be equipped with water-saving fixtures including 1.6 gpf toilets, water efficient showerheads, faucets, and urinals. Existing housing equipped with higher gal/flush toilets are not required to switch to 1.6 gpf toilets, or the other water saving fixtures, although some municipalities have offered financial incentives to homeowners to make the switch voluntarily. Homeowners who may be renovating an older home are not required to switch to a 1.6 gpf toilet or the other water saving fixtures. If the existing fixtures are still functional, a homeowner may remove them, renovate the building, and re-install the same fixtures.

As of May 1, 1999 CGS Sec. 29-252-1c, the State Building Code-Connecticut Supplement, amended the Connecticut State Building Code. That amendment repealed the previous State Building Code. The BOCA National Building Code/1996, the 1997 International Plumbing Code, and the 1995 CABO One and Two Family Dwelling Code, except as amended, altered or deleted by the Connecticut Supplement, were adopted by reference as the State Building Code. (Note: there were no amendments, alteration or deletions in the Connecticut Supplement that effect the maximum water consumption requirements for water using fixtures.)

Section 604.4 of the 1997 International Plumbing Code established the following maximum water consumption requirements for plumbing fixtures and fixture fittings:

Lavatory, Private	2.5 gpm at 80 psi
Lavatory, Public	0.5 gpm at 60 psi
Lavatory, public, metering or self closing	0.25 gal per metering cycle
Shower head	2.5 gpm at 80 psi
Sink faucet	2.5 gpm at 80 psi
Urinal	1.0 gal per flushing cycle
Water Closet	1.6 gal per flushing cycle

The 1997 International Plumbing Code also provides for the same exceptions for toilets and urinals that were included in the 1993 BOCA National Plumbing Code.

Thus, other than for the exceptions noted above, all new buildings are required to install the new low-flow water fixtures. The use of these fixtures can be expected to reduce the wastewater flows generated in new or retrofitted buildings below the flows experienced before the adoption of water conservation regulations by the State and the Federal governments.

D. Published Studies on Residential Water Use

A study of residential water use had been conducted by the Johns Hopkins University in the mid-1960s for the U.S. Department of Housing and Urban Development prior to the advent of widespread use of water conservation fixtures. The final report of the study (Linaweaver, Geyer and Wolff-1967) provided information obtained in cooperation with sixteen water utilities located in various metropolitan areas of the U.S.

The study did not incorporate individual water use recorders, as this was not considered feasible for the number of study areas and homes involved. Instead, the flow data was determined by the installation of master meter-recorder systems to measure water supplied to small homogeneous residential areas and dividing the total water use during the non-sprinkling season by the number of homes in each area to obtain the domestic water use per residence. Domestic water use was defined as water used within the home for domestic purposes including drinking, cooking, bathing, washing, and carrying away wastes. The authors stated, "Practically all domestic water is discharged to the sewers or septic tank systems and thus is non-consumptive use". The characteristics of the study areas were as follows:

Statistic	Number of Dwelling Units	Persons per Dwelling Unit
Minimum	44	3.1
Mean	178	4.1
Maximum	307	4.9

The mean values of domestic (household indoor) water use in five eastern metropolitan areas[1] with metered public water supply and septic systems were as follows:

	Avg. Annual Day	Maximum Day	Peak Hour
	191 gal/du	247 gal/du	530 gal/du
Ratio to Avg.:	1.00	1.29	2.77

The mean values of domestic (household indoor) water use in 13 eastern metropolitan areas with metered public water supply and public sewers were as follows:

	Avg. Annual Day	Maximum Day	Peak Hour
	209 gal/du	271 gal/du	536 gal/du
Ratio to Avg.:	1.00	1.30	2.56

Thus, the average annual day and maximum average day domestic water use in areas served by septic systems was about 91% of that in areas served by public sewers. However, the peak hourly use to average annual daily use ratio in areas served by septic systems was somewhat higher.

[1] District of Columbia, Washington, D.C., Washington Suburban Sanitary District, Hyattsville, MD, City of Baltimore and Baltimore County, MD, City of Philadelphia, PA, and Philadelphia Suburban Water Company, Bryn Mawr, PA.

The most recent definitive study on the effects of low-flow fixtures on residential water use was underwritten by the American Water Works Association Research Foundation and reported in "Residential End Use of Water" (AWWARF, 1999). This report is based on data collected from 1,188 single- family homes in 12 North American locations (11 in the U.S., 1 in Ontario, Canada). This study involved the use of state-of-the art surveys, data loggers and trace analysis equipment.

In addition to providing overall water use data, the report (ibid.) provided a quantitative description of the various uses of water in the home and a comparison of such use with and without the use of low-flow water using fixtures. Some of the information included in this report is given in Tables No. R-2, R-3, and R-4.

TABLE R-2

CURRENT INDOOR WATER USE
(Adapted from AWWARF 1999 Report on Residential End Uses of Water)

		Mean Persons	Gallons per Capita per Day (gpcd)		
Study Site	Sample Size	per Household	Mean Daily	Median Daily	Std. Deviation
Waterloo/Cambridge, Ontario	95	3.1	70.6	59.5	44.6
Seattle, WA	99	2.8	57.1	54.0	28.6
Tampa, FL	99	2.4	65.8	59.0	33.5
Lompoc, CA	100	2.8	65.8	56.1	33.4
Eugene, OR	98	2.5	83.5	63.8	68.9
Boulder, CO	100	2.4	64.7	60.3	25.8
San Diego, CA	100	2.7	58.3	54.1	23.4
Denver, CO	99	2.7	69.3	64.9	35.0
Phoenix, AZ	100	2.9	77.6	66.9	44.8
Scottsdale/Tempe, AZ	99	2.3	81.4	63.4	67.6
Walnut Valley WD, CA	99	3.3	67.8	63.3	30.8
Las Virgenes MWD, CA	100	3.1	69.6	61.0	38.6
Total of Study Sites	1,188	2.8	69.3	60.5	39.6

TABLE R-3

MEAN DAILY PER CAPITA INDOOR WATER USE
(Adapted from AWWARF 1999 Report on Residential End Uses of Water)

Study Site	Sample Size	Toilets	Faucets	Showers	Baths	Dish washers	Clothes washers	Leaks	Other	Total
Waterloo/Cambridge, Ontario	95	20.3	11.4	8.3	1.9	0.8	13.7	8.2	6.0	70.6
Seattle, WA	99	17.1	8.7	11.4	1.1	1.0	12.0	5.9	0.0	57.1
Tampa, FL	99	16.7	12.0	10.2	1.1	0.6	14.2	10.8	0.3	65.8
Lompoc, CA	100	16.6	9.9	11.1	1.2	0.8	15.3	10.1	0.9	65.8
Eugene, OR	98	22.9	11.9	15.1	1.5	1.4	17.1	13.6	0.1	83.5
Boulder, CO	100	19.8	11.6	13.1	1.4	1.4	14.0	3.4	0.2	64.7
San Diego, CA	100	15.8	10.8	9.0	0.5	0.9	16.3	4.6	0.3	58.3
Denver, CO	99	21.1	10.5	12.9	1.6	1.2	15.6	5.8	0.5	69.3
Phoenix, AZ	100	19.6	9.6	12.5	1.2	0.8	16.9	14.8	2.2	77.6
Scottsdale/Tempe, AZ	99	18.4	11.2	12.6	0.9	1.1	14.5	17.6	5.0	81.4
Walnut Valley WD, CA	99	18.0	12.3	11.7	1.0	0.8	14.1	7.6	2.3	67.8
Las Virgenes MWD, CA	100	15.7	11.2	11.4	1.3	0.9	16.8	11.2	1.1	69.6
Total of Study Sites	1,188									
Mean Values gpcd		18.5	10.9	11.6	1.2	1.0	15.0	9.5	1.6	69.3

TABLE R-4

HOUSEHOLD END USE OF WATER

WITHOUT AND WITH CONSERVATION, POTENTIAL SAVINGS
(Adapted from the AWWARF Residential End Use of Water Study)

	Without Conservation**		With Conservation		Savings	
End Use	Share	gcd	Share	gcd	%	gcd
Toilets	27.7%	20.1	19.3%	9.6	52%	10.5
Clothes Washers	20.9%	15.1	21.4%	10.6	30%	4.5
Showers	17.3%	12.6	20.1%	10.0	21%	2.6
Faucets	15.3%	11.1	21.9%	10.8	2%	0.3
Leaks***	13.8%	10.0	10.1%	5.0	50%	5.0
Other Domestic	2.1%	1.5	3.1%	1.5	0%	0
Baths	1.6%	1.2	2.4%	1.2	0%	0
Dish Washers	1.3%	1.0	2.0%	1.0	0%	0
Inside Total	100%	72.5	100%	49.6	32%	22.9

** Based on the average inside uses measured in 1,188 homes in 14 North American cities including an additional 5% to account for estimated "in place" savings due to existing conservation.

*** The leakage rate shown is an average for the large population of homes monitored in the Residential End Use Study. Nearly 60% of leakage volume was found to be explained by less than 10% of the homes.

The authors of the AWWARF study indicated that creating national water use "averages" was not an objective of the study and that the pooled results were presented for summary and comparative purposes alone. Since all but two of the 12 locales in that study are located in the west, the relatively high per capita mean water use of 69.3 gpcd given in that report may be biased with respect to water use for residences in Connecticut served by on-site wastewater renovation systems because of differing climatic conditions and patterns of water use. It should be noted that Linaweaver, et al. (1967) indicated that the per capita inside water use in residences served by septic systems averaged 47 gpcd as compared to 51 gpcd for residences served by public sewers in the east and 67 gpcd for such residences in the west.

However, a significant conclusion of the report was the similarities between the twelve study sites in the amount and types of water fixtures and appliances used. The report states that this information had significant "transfer" value across North America. Thus, the projected savings in water use resulting from use of each type of low flow fixture and appliance, as shown in Table 3, should be applicable to any location in the United States.

If only the water savings resulting from use of toilets, showerheads and faucets conforming to the current code requirements are considered, the results given in Table 3 indicate the following per capita savings would result:

Fixture Measure	Use Rate	Savings in gpcd
Ultra-Low flush toilets	1.6 gpf	10.5
Low flow showerheads	2.5 gpm	2.6
Low Flow Faucets (installed on kitchen sink and bathroom faucets)	2.2 gpm	0.3
Total Savings:		13.4

It should be noted that the savings resulting from use of ultra-low flush (ULF) toilets shown in Table 3 include the effects of occasional double flushing. (This is also true of the amount of water use shown in Table 3 resulting from the non-conservation types of toilets.) Also, the results are relevant even though some of the 1.6gpf ULF toilets reputedly have a flushing volume in excess of 1.6 gallons (LADW&P- 2000), as the results are based on measured water use rather than on fixture rating. Thus, the results shown above are net savings.

Applying these savings to the per capita flow allowance of 75 gpd derived from the residential design flow of 150 gpd per bedroom in the CT Public Health Code (2004) results in reducing the allowance to about 62 gpcd, a reduction of 17 percent. If only the saving from an ultra-low flush toilet is considered, the 75-gpcd allowance would be reduced to about 65 gpcd, a reduction of 13 percent. A number of municipal water agencies have reported savings upwards of 15 percent of indoor water use after retrofitting of customer buildings with low flow and ultra-low flow water fixtures.

For example, Santa Monica, CA has observed water savings averaged 15% after completion of a toilet and showerhead replacement program. Houston, TX found water savings averaged 18% per household after distribution of water conservation kits.

New York City, in a survey of 67 apartment buildings retrofitted with water saving fixtures found an average reduction of 29% of water use. Seattle, WA, estimates that its Commercial (water) Conservation Program could cut commercial-sector water use by 20%. Tampa, FL's toilet replacement program resulted in a 15% reduction in water use.

The information presented herein regarding the reduction in water use resulting from use of water efficient fixtures and appliances is particularly helpful in addressing situations where on-site wastewater disposal systems have failed due to hydraulic overload. Replacing all existing faucets and shower heads with low flow fixtures; replacing existing dishwashers with the more energy efficient types, replacing existing clothes washers with the new energy efficient side loading designs that save from 30 to 40 percent of water used by older top loading washers, and replacing existing toilets with ultra-low-flow (ULF) toilets could effect a very substantial (30% or more) reduction in wastewater flows. Indications are that the combined cost of retrofitting plus the cost of a smaller replacement on-site wastewater renovation system may be less than the cost of constructing a replacement on-site wastewater renovation system sized on the basis of previous water use.

The replacement of clothes washers with new high-efficiency washers is problematic, because of the additional "first cost" involved. However, it is reputed that the high-efficiency front-loading washers use up to 50 percent less energy, can cut water usage by 30 percent or more and get clothes 25 percent cleaner than traditional top loading models (U.S. Water News Online-a). Thus, there is a payback resulting from the use of such washers in a lower electric bill, and where water is also purchased, an additional payback would result. In 1998, a consortium of 16 electric and gas utilities in New England (the Northeast Energy Efficiency Partnership) launched the TumbleWash program to promote awareness and use of high-efficiency, front-loading washing machines (U.S. Water News Online-a). Thus, there should be an increase in the high-efficiency washer penetration of the clothes washer market in years ahead, and further reduction in residential water use can be expected.

With respect to allowing a wastewater flow reduction credit for water efficient plumbing devices, Siegrist (1981) discussed two major considerations that have to be addressed. "Firstly, one must be relatively confident that the use of a given device or system will yield the predicted waste load (flow) reduction. A second major consideration is the necessity that the technique or device utilized be accepted by the present users as well as future users." He went on to state that "(1) the appropriate regulatory authority could allow only those devices whose characteristics and merits indicate the potential for long-term user acceptance; (2) the plumbing system could be installed in such a way as to discourage disconnection or replacement of a device; or (3) periodic inspection by a local inspector within the framework of a sanitary district or the like may serve to identify plumbing alterations." (In other words, consideration must be given to user circumvention or removal of water saving devices.)

In general, passive wastewater modification methods or devices not significantly affected by user habits tend to be more reliable than those that are subject to user habits and require a preconceived active role by the users. For example, a low-flow toilet is a passive device, while a flow-reducing showerhead is an active one. (National Small Flows Clearinghouse -1997)

It would seem that long-term user acceptance is the better approach to use with respect to granting any wastewater flow credit for water-efficient plumbing devices and appliances. In this respect, it would seem that the Ultra-Low Flush toilet is generally accepted by the public and improvements will continue to be made by the manufacturers to allay the public concern, such as it is, for such matters as double-flushing, bowl staining, and noise (for pressure operated types).

Other water savings devices such as the low flow faucets and showerheads are now required by the State Building Code and are generally accepted, although they can be altered relatively easily to provide an increased flow rate. Thus, there is some risk in granting a flow credit for such fixtures.

The use of high-efficiency, front-loading washing machines has not yet been codified, and they can be replaced, although the replacement would represent a significant monetary expenditure. Additional information will have to be developed regarding the acceptance of such machines by the public.

The AWWRF 1999 study indicates that the savings in per capita water use from use of more efficient dishwashers will be small, on the order of 1 gpcd or less, and thus is not a significant factor in considering any credit for water-efficient plumbing devices and appliances.

Based on the information presented above, a reduction of the residential per capita flow allowance inherent in the CT Public Health Code (currently 75 gpcd) to 65 gpcd might appear to be appropriate where ULFTs are proposed. Note that this reduction is based on utilization of ULFT fixtures throughout the residential facility, and further, that no increase in reduction should be allowed because more than one ULFT will be installed per dwelling unit. In the case of commercial and institutional rest rooms, however, the reduction should be on a fixture use basis.

On the other hand, a recent report (NAHB-2002) to the Seattle Public Utilities, Seattle, WA and the East Bay Municipal Utility District, Oakland, CA suggests that such a reduction in flow allowance may be problematic. That report presented the results of performance testing and evaluation of 49 different new models of ULF toilets. The toilets tested included gravity, pressure-assist, and vacuum-assist models as well as a few special models, such a dual-flush, flapperless and air-assist units. The toilets used in the testing are generally available nationwide at large home improvement centers and plumbing supply stores. The results of the testing and evaluation program cast a different light on potential water savings. The study found that "out of the box" flush volumes of nominal 1.6 gallons per flush (gpf) toilets ranged from 1.45 to 1.89 gpf. The average flush volume of those toilets that exceeded 1.6 gpf was 1.68 gpf; this not a very significant difference.

However, of more concern was the finding that "After replacement of the original flapper with a generic flapper[2], the flush volume for the 1.6 gpf fixtures that could be retrofitted with a standard flapper ranged from 1.03 to 4.66 gpf. Twenty-eight of the 33 models that could be retrofitted with a standard flapper used more than 1.6 gpf after flapper

[2] A "universal" replacement available at hardware stores and home centers.

replacement and averaged 2.91 gpf. This is consistent with the results of an earlier study wherein similar flapper replacements were performed (MWDSC-1998*)*. Because flapper valves typically require replacement several times during the useful life of a toilet fixture and the likelihood that the consumer will install a generic replacement flapper, water efficiency of many of the tested models could significantly degrade over time."

Of the 49 ULF toilets tested, 16 could not be retrofitted with generic flappers, as only the original manufacturer's flappers could be installed. Of these 16, the out-of-box flush volume ranged from 0.62 to 1.70 gpf and averaged 1.47 gpf. Ten of these 16 units also had superior flushing performance under the test conditions. Thus, it appears that there are ULF toilets on the market that can effect a considerable saving in toilet water use while providing acceptable performance. However, the following recommendation of the report is worth noting. "The plumbing industry, in cooperation with the water utility industry, should develop a parts identification and distribution system for flush valve flappers that will assure the consumer will purchase the appropriate replacement flapper to maintain the 1.6 gpf that the fixture was designed for." Until such time as that recommendation is universally adopted, it would seem that a reduction in flow allowance for the installation of ULF toilets should be viewed with caution.

It should be noted that reducing wastewater volume most likely would result in an increase in the concentration of the various pollutants since it is unlikely that the mass of pollutants will be changed. The National Small Flows Clearinghouse publication "Water Conservation Treatment Technology Package (NSFC-1997) indicates that monitoring of septic tanks in Indiana and Pennsylvania over several years reinforced the fact that the increase in pollutant concentrations in effluent is proportional to the decrease in flow. On the other hand, some consideration has been given to the possibility that, if wastewater volumes are reduced without changing the volume of the septic tank, the pollutant removal capabilities of the septic tank will be enhanced, and thus there may not be any significant change in the pollutant concentrations discharged to the SWAS.

Even if there is a proportionate increase in pollutant concentrations, it is doubtful if this would have a significant impact on the performance of a single-family dwelling on-site wastewater renovation (OSWR) system. However, this will not be the case where reductions in wastewater flows from large scale OSWR systems serving a number of dwellings or commercial/institutional establishments are being considered because of the use of high-efficiency plumbing devices and appliances. In that case, where wastewater "strength" is apt to be much higher than residential wastewater, any increase in strength caused by water conservation may have a significant effect on wastewater characteristics.

The following fact should also be borne in mind when comparing unit wastewater flow data from buildings served by on-site systems with similar data obtained from flow measurements in sewered areas. There is a significant difference between the unit flows based on metered water use in individual residential, commercial or institutional buildings and the unit wastewater flow data usually reported for "domestic" wastewater based on flow measurements in collection sewers or at wastewater treatment plants. The latter data may be biased by the inclusion of contributions from commercial and institutional sources, and sometimes by small industrial sources, and is also biased by the effect of inflow and infiltration into the sewer system. The result is that the unit flows derived from sewer measurements are apt to be larger than those derived from flow measurements obtained from individual building water use data. (This is also a significant consideration when comparing concentrations of wastewater constituents.)

E. Published Information on Residential Wastewater Flows

Loomis and Dow (1998) reported on an on-site wastewater demonstration project funded by the State of Rhode Island, involving 12 single-family residences. The mean wastewater flows, based on records from water meters installed on drainfield pressure lines, for the period from August to October 1997, were as follows:

Table R-5
12 Single Family Residences - Wastewater Generated per Household

Persons per Household	Wastewater Generated per Household (gal/d)	gpcd
2 adults, 3 children	195	38.8
2 adults, 3 children	234	46.8
2 adults	29.9	15.0
5 adults	346	69.2
2 adults, 2 children	134	33.4
2 adults, 3 children	142	28.3
2 adults, 1 child	126	41.8
2 adults, 1 child	120	39.8
1 adult, 3 children	115	28.6
2 adults, 2 children	278	69.4
2 adults, 1 child	110	36.5
2 adults, 4 children	179	29.9
Mean:	167	39.8

It should be noted that all of these residences were located on small lots; sizes ranged from 2,250 square ft. to 20,000 sq. ft in area, with all but 3 lots being less than 10,000 sq. ft. in area. The small lot sizes, together with the fact that these sites had failed septic systems prior to their replacement with new systems, may have had an impact on the amounts of wastewater generated, compared to the amounts that might have been generated had there not previously been a history of failed systems. (Many homeowners with failed systems will try to reduce normal water use in such situations, and this may have carried over even after the new systems were installed.)

U.S. EPA (2002) reviewed the results of recent studies on residential water use and indicated that an estimated average daily wastewater flow of approximately 50 to 70 gpcd would be typical for residential dwellings built before 1994 (the year that the U.S. Energy Policy and Conservation Act standards went into effect), and 40-60 gpcd for residences built after 1994.

F. Data on Residential Water Use from Department Files

Table R-6

ELDERLY HOUSING DEVELOPMENT
(Fairfield County)

Metered Water Use for Period 7/12/00 to 2/6/02

No. Bedrooms Served	No. Persons Served	No. Persons per BR	Mean Water Use/BR	Mean Water Use/Capita
111	152	1.4	87 gpd	63 gpcd

Table R-7

CLUSTER SYSTEM DESIGNED TO SERVE EIGHT SINGLE FAMILY HOMES
(Middlesex County)

Period of Time	ADF GPD	No. BR. Served	Flow/BR GPD
11/12/98 - 2/18/99	2,049	23	89
2/19/99 - 5/28/99	2,118	23	92
5/29/99 - 8/31/99	1,701	23	74
9/1/99 - 11/30/99	1,619	23	70
12/01/00 -2/25/00	1,759	23	76
2/26/00 - 5/26/00	1,675	23	73
5/26/99 - 8/31/00	1,716	23	75
9/1/00 - 11/27/00	1,555	23	68
11/28/00 - 2/15/01	1,707	23	74
2/16/01 - 5/30/01	1,557	23	67
6/1/01 - 8/30/01	1,567	27	58
8/31/01 - 11/27/01	1,687	27	63

Mean, 6/1/01 -11/27/01 for 7 dwellings* with a total of 27 BR = 73 gpd/BR

* (Six, 4 BR Dwellings and one - 3BR Dwelling.)

TABLE RC-1

ELDERLY RETIREMENT COMMUNITY
(Middlesex County)

90 residential apartments (54 one-bedroom and 36 two-bedroom dwelling units), containing a total of 126 bedrooms and a commons building containing a central kitchen-dining room for serving meals to the residents and their guests.

AVERAGE DAILY METERED WATER USE PER RESIDENT

Month & Year	Total Water Use, Gallons	Avg. Water Use, Gallons per Day	Resident Population	Gals. per Day Per Resident
Jan. '97 (33)*	273,768	8,296	113	73.4
Feb. '97 (28)	245,344	8,762	113	77.5
Mar. '97 (31)	267,784	8,638	110	78.5
Apr. '97 (30)	297,704	9,923	110	90.2
May '97 (30)	323,884	10,796	113	95.5
June '97 (31)	345,576	11,148	112	99.5
July '97 (31)	378,488	12,209	111	110.0
Aug. '97 (31)	365,024	11,775	107	110.0
Sept. '97 (31)	330,616	10,665	109	97.8
Oct. '97 (31)	309,672	9,989	108	92.5
Nov. '97 (31)	316,404	10,207	109	93.6
Dec. '97 (28)	270,776	9,671	109	88.7
Jan. '98 (32)	282,744	8,835	106	83.4
Feb. '98 (28)	246,840	8,816	110	80.1
Mar. '98 (31)	285,736	9,217	102	90.4
Apr. '98 (31)	299,200	9,652	108	89.4
Mean:		9,912 gpd	109	90.7 gpcd
Mean, most recent 12 months		10,248	109	94.2

* Actual Number of Days included in billing period.

Table No. RC-2 provides information on the average daily water use per apartment. Analysis of apartment occupancy data for the 1997 calendar year indicated that approximately 70% of the occupied apartments were occupied by one resident, approximately 59 % of the two bedroom apartments were occupied by one resident, and the ratio of residents to occupied apartments averaged 1.29.

TABLE RC-2

ELDERLY RETIREMENT COMMUNITY
AVERAGE DAILY WATER USE PER APARTMENT

Month & Year	Avg. Water Use Gals. per Day	Apartments Occupied No.	%	Gal. per Day Per Apartment
Jan. '97	8,296	86	95.6	96.5
Feb. '97	8,762	87	96.7	100.7
Mar. '97	8,638	85	94.4	101.6
Apr. '97	9,923	86	95.6	115.4
May '97	10,796	87	96.7	124.1
June '97	11,148	86	95.6	129.6
July '97	12,209	86	95.6	142.0
Aug.'97	11,775	82	91.1	143.6
Sept.'97	10,665	84	93.3	127.0
Oct. '97	9,989	83	92.2	120.3
Nov.'97	10,207	85	94.4	120.1
Dec. '97	9,671	85	94.4	113.8
Jan. '98	8,835	80	88.9	110.4
Feb. '98	8,816	83	92.2	106.2
Mar. '98	9,217	78	86.7	118.2
Apr. '98	9,652	81	90.0	119.2
Mean :	9,914	84	93.6	118.0
Mean, most recent 12 months:	10,248	83	92.6	122.9

Table No. RC-3

Life Care Retirement Community
Middlesex County, CT

189 residential apartments, 45 convalescent beds, and a commons building containing a central kitchen-dining room for serving meals to the residents and their guests.

	Water Use over 12 Month Period Min. Day	Avg. Day	Max. Day	Max:Avg.Day
Mean: gpd	15,783	21,271	27,425	1.29

G. Published Information on Commercial and Institutional Water Use

1. Effect of efficient plumbing fixtures and appliances

The effect of new water efficient plumbing fixtures and appliances is apt to be more pronounced on commercial sources, since the wastewater contributions from bathroom fixtures are usually a much higher percentage of the total wastewater discharged from such sources. Thus, one can expect a lower per capita or per fixture wastewater discharge and a high concentration of wastewater pollutants than found in historical data that does not include the effects of the new water efficient plumbing fixtures. The effect on institutional wastewater flows may or may not be pronounced, depending upon the nature of water use at such institutions. It will therefore be important to carefully evaluate historic water use (wastewater discharge) data in the context of the number and types of plumbing fixtures and appliances that may have been in use when that data was generated.

A major Ultra Low Flow Toilet (ULFT) study looking at toilet retrofits was conducted for the California Urban Water Conservation Council between 1992 and 1996 (Hagler Bailly Services- 1997). The project evaluated the effect of ULFTs for 12 categories of establishments served by 10 California water agencies. The study estimated the following savings per installed ULFT (AWWARF- 2000):

Category	Savings, gpd
Food Stores	32
Health Care Facilities	21
Hotel/Motel	16
Offices	20
Religious Facilities	28
Restaurants	47
Retail Stores	37
Manufacturing	23

In Tampa, Florida, retrofitting a junior high school with ULFTs was found to have reduced water use by 32% (AWWARF- ibid.).

Bamezai and Chestnut (1994) reported the results of a retrofit program by the San Diego, CA, Water Utilities Department. Evaluation of results from 70 sites retrofitted with ULFTs showed that water savings varied across categories within the public sector. The number of users, number of toilets per facility and the nature of the facility were some of the factors effecting water savings in these public facilities. The least savings occurred in police stations (20.5 gallons per toilet per day (gtd), and the most savings occurred in recreation centers, senior centers, and pools, with an average of 116.8 gtd. (AWWARF- ibid.).

2. Commercial and Institutional Water Use

A significant study of commercial water use was conducted by The Johns Hopkins University in 1966 (Wolff, Linaweaver and Geyer-1966). In addition to obtaining and analyzing water use data from recording devices installed on water meters of commercial and institutional consumers, the literature was reviewed to obtain data from prior work by others on commercial and institutional water use. While the results of this study could prove helpful in designing water distribution systems serving commercial and institutional facilities, they are not helpful for estimating wastewater flows because the total water use was not disaggregated into indoor use (assumed equivalent to wastewater discharge), outdoor use and continuous uses such as air conditioning and leakage.

The most recent comprehensive information published on commercial and institutional water use is the report sponsored by the American Water Works Association Research Foundation (AWWARF, 2000) entitled "Commercial and Institutional End Uses of Water". This report summarizes and interprets the existing knowledge base of utility-supplied potable water in urban areas. The public utilities who participated in this study were:

1. Los Angeles Department of Water and Power , California
2. Irvine Ranch Water District, California
3. City of San Diego Water Utilities Department, California
4. City of Santa Monica, California
5. City of Phoenix Water Services, Arizona.

It is important, when reviewing the data in this report, that it is biased both with respect to the source of the data (urban locations served by water utilities) and the location of the agencies providing the data (western U.S.). Nevertheless, it is a definitive study of commercial and institutional water use and may be of use in judging the value of data obtained from other sources.

The following five commercial and institutional (CI) categories were selected for detailed analysis: Schools, Hotels/Motels, Office Buildings, Restaurants, and Food Stores. Field data were obtained from data loggers installed on water meters, and from sub-meters installed where practical to do so. Sub metering proved impracticable for all but a few facilities, because of the layout of the internal water piping.

Each data logger was fitted with a magnetic sensor that was strapped to the water meter at each site. As water was used, it flowed through the water meter causing the internal magnets of the water meter to spin. The sensor picked up each magnetic pulse as water flowed through the meter and the logger counted the number of pulses detected and stored the total every 10 seconds. Using the physical characteristics of each specific brand and model of water meter, the magnetic pulse data from the data logger was transformed into an average flow rate for each 10-second interval. This flow trace is precise enough to detect the individual flow signatures of water using equipment and appliances and plumbing fixtures in the building and that of any irrigation system. The data obtained from the data loggers was used for flow tracing analyses using custom signal processing software to disaggregate the flows into identifiable component end uses. In the few cases where sub-metering proved practical, the information from these meters was also used to disaggregate flows.

The field data obtained was disaggregated into three basic categories; indoor use, outdoor use, and continuous water use. Indoor use included all domestic sanitary, process, mechanical equipment, cleaning uses and periodic leaks. Outdoor use included irrigation, pool filling, driveway/patio washing, etc. Continuous use included leakage and cooling water demand. In many cases, the indoor water use was further disaggregated into subcategories that varied depending upon the CI category of the facility. After the flow traces from an individual site were analyzed, daily estimates were made for all of the identifiable categories during the logging period. These daily estimates were used in conjunction with the billing data for each facility supplied by the utilities and other information collected during site surveys to create estimates of average annual use for each CI category.

Because the indoor water use was determined, it is possible to use this data judicially for estimating wastewater flows. A summary of the indoor water use at the various CI facilities investigated in the field is given below.

a. Office Buildings

Detailed water use data were determined for five office buildings in the manner previously described. The size and occupancy of these five buildings were as follows:

Location:	Irvine	Los Angeles	Phoenix	San Diego	Santa Monica
Use	Commercial	Commercial	Clinic	Gov. Agency	Commercial
Size (sq. ft.)	57,785	176,500	10,000	8,800	186,000

The average in-door water use was given in terms of gallons/sf/year. Dividing this data by 250 days/year (assuming a 5-day work week and 10 vacation days) yields the following data:

Use	Irvine	Los Angeles	Phoenix	San Diego	Santa Monica
Toilet	0.021		0.032	0.105	0.010
Faucets	0.002		0.019	0.020	
Other/Misc.	0.004		0.001	0.026	0.005
Total Use- g/sf/d	0.027	0.088	0.052	0.151	0.015

Based on an audit of metered water billings for 50 office buildings in Arizona, California, Colorado, and Florida, the following percentiles were given for indoor use:

Office Building Water Use

Percentiles	10%	25%	50%	75%	90%
Indoor Use, gpd/sf	0.011	0.026	0.039	0.069	0.125

b. Restaurants

Direct field measurement studies were made at five restaurants. All were family style, sit-down establishments, as opposed to fine dining or fast food restaurants. The restaurants ranged in size from 73 to 253 seats and served from 190 to 800 meals per day. All had on-site dish washing. The average in-door water use per meal served (g/meal) was as follows:

Use		Irvine	Los Angeles	Phoenix	San Diego	Santa Monica
Dishwashing		0.9		1.4		1.1
Toilets/Urinals		0.4		0.5		0.5
All Other		1.4		3.5		1.8
Total	g/meal	2.7	10.5	5.4	16.2	3.4

Based on an audit of metered water billings for 87 restaurants in California, Florida, and Colorado, the following percentiles were given for restaurant indoor water use:

Percentiles	10%	25%	50%	75%	90%
Gal/meal	5.8	7.0	11.2	18.7	35.5

c. Supermarkets

Detailed water use data were determined for five supermarkets in the manner previously described. All were large, full service stores with produce, meat, deli, and bakery departments. Each supermarket had some form of hot food service. The size of these five buildings were as follows:

Location:	Irvine	Los Angeles	Phoenix	San Diego	Santa Monica
Size (sq. ft.)	38,000	50,000	48,000	66,000	45,000

The average in-door water use was given in terms of gallons/sq. ft./year. Dividing this data by 365 days/year yields the following data, in gpd/sq ft.

Use	Irvine	Los Angeles	Phoenix	San Diego	Santa Monica
Toilets/Urinals	0.02	0.02	0.02	0.01	002
Other Misc.	0.09	0.08	0.05	0.05	0.06
Total Indoor, gpd/sf	0.11	0.10	0.07	0.06	0.08

Based on an audit of metered water billings for 33 supermarkets in California and Arizona, the following percentiles were given for indoor use:

Percentiles	10%	25%	50%	75%	90%
gpd/sq. ft	0.047	0.065	0.091	0.126	0.174

d. Hotels and Motels

Detailed water use data were determined for five hotels in the manner previously described. The number of rooms at these five hotels were as follows:

Location:	Irvine	Los Angeles	Phoenix	San Diego	Santa Monica
No. of Rooms	148	297	140	209	168

The hotels at Irvine, Phoenix and San Diego were economy/budget franchises. The Santa Monica facility was a combination economy travel lodge and beach resort. The Los Angeles facility was a large luxury class hotel and was the only one with restaurant and banquet facilities.

The average indoor use was given in gallons per day per room. These values do not include ice-making use, since such usage was considered negligible with respect to wastewater discharges.

Use	Irvine	Los Angeles	Phoenix	San Diego	Santa Monica
Bathtub	0	6.4	2.7	0	0
Faucets	6.0	17.3	7.4	7.6	6.8
Showers	28.0	88.9	37.6	34.1	30.6
Toilet	26.0	76.8	32.6	32.8	29.5
Leaks	21.9	14.7	6.2	1.3	1.2
Total In-Room	81.9	204.1	86.5	75.8	68.1
Laundry	16.6	0	17.5	31.6	33.0
Other/Misc.	2.6	22.7	3.1	27.3	12.5
Total, gpd/room	101.1	226.8	107.1	134.7	113.6

It is interesting to note that for all but the luxury hotel in Los Angeles, the in-room water use values do not vary greatly, ranging from 68.1 to 86.5 gal/room with an average of 78.1 gal/room .

Based on an audit of metered water billings for 100 hotels and motels in Arizona, California, Florida, and Colorado, the following percentiles were given for indoor use:

Percentiles	10%	25%	50%	75%	90%
Gal/room/day	55	85.1	116.8	145.4	187.9

e. Public High Schools

Detailed water use data were determined for four public high schools in the manner previously described. The number of students/staff, annual operating days at each school, and building footprint area were as follows:

Location	Irvine	Los Angeles	Phoenix	Santa Monica
No. Students/staff	2640	3850	2186	3065
Annual Operating Days	180	340	180	340
Building Footprint (sf)	224,652	253,357	325,000	220,000

Two of the high schools operated on a traditional school year calendar and the other two followed a year-round calendar. The number of students/staff at these schools was probably larger than what would be found at high schools in Connecticut served by on-site wastewater renovation systems. The indoor water use per person (student/staff), based on the annual operating days indicated above for each school, was as follows:

Use	Irvine	Los Angeles	Phoenix	Santa Monica
Toilets	1.51			1.15
Urinals	0.59			0.55
Faucets	0.48			0.48
Showers	0.24			0.14
Kitchen	0.32			0.21
Misc.	0.00			0.20
Total Use, gpcd	3.14	3.73	6.69	2.17

Based on an audit of metered water billings for 136 schools in Arizona, California, Colorado, and Florida, the following percentiles were given for indoor use:

School Water Use (Includes grade schools, middle schools and high schools)

Percentiles	10%	25%	50%	75%	90%
G/Student/School Day	5.9	8.1	11.5	16.2	24.3

f. Offices

Behling and Bartillucci (1992) analyzed metered water records covering a 3-year period for each of 23 office complexes located on Long Island, NY. All of these office buildings were equipped with plumbing fixtures conforming to the 1980 New York State plumbing code, which required 3.5 gal/flush toilets, 1.5 gal/flush urinals, and 3.0 gpm lavatory faucets. To eliminate water use due to outdoor irrigation and cooling water (air conditioning), the data were averaged for the fall-winter months (October through March). Thus, the water use data, assumed to be equivalent to wastewater discharge, includes water used for rest room facilities, drinking water fountains, building maintenance and accessory non-office amenities such as snack bars, restaurants and shops.

The daily water use for the fall-winter period was calculated on the basis of a five-day workweek. Building areas ranged from 45,000 sq. ft. to 2,000,000 sq. ft. and the average occupancy rate was reported to range between 85 and 90%. The average water use was calculated to be 0.045 gpd/sq. ft., and ranged from 0.014 to 0.084 gpd/sq. ft.)

Behling and Bartillucci (ibid.) also developed a method of estimating office indoor water use on the basis of frequency of fixture use and water use per fixture. Their method assumes the following:

Population Density - 250 sq. ft. per person
Gender Mix (% men and women occupying office building)
Frequency of fixture use
    Women - 3 toilet uses /day and 3 lavatory uses/day
    Men - 1 toilet use/day, 2 urinal uses/day, and 3 lavatory uses/day
Lavatory Use - 10 seconds/use (hand washing)
Service Sink Water Use - 100 gpd
    Building Maintenance - 250 gpd
Allowance for non-office use amenities, transient (non-occupant) restroom usage, and leaking fixtures = 20% of the total inside-building water use.

H. Commercial and Institutional Water Use Data from Department Files

TABLE SC-1
SHOPPING CENTER
(159,939 Sq. Ft. of Retail Space Available)

Month, 1991	Avg. GPD	Sq. Ft. Occupied	GPD/SF
March	8,400	149,282	0.056
April	9,066	149,282	0.060
May	7800	149,282	0.052
June	11,700	149,283	0.078
July	7,645	146,628	0.052
August	8,484	149,282	0.057
September	7,900	146,658	0.053
October	7,516	146,658	0.051
		Mean:	0.057

TABLE T-1
THEATERS

Source	No. Seats	Length of Record, Qtrs,	High Qtr. Water Use GPD/Seat
Theaters w/1.6 Gal./Flush Toilets			
Theater A	3080	6	1.37
Theater B	3574	5	1.14
Theaters w/3.5 Gal/Flush Toilets			
Theater C	2,344	6	3.09
Theater D	2,000	5	3.10
Theater E	2,540	4	2.02
Theater F	1,233	2	2.22

Theater B had 1.6 gal/flush toilets, 1.0 gal/use urinals, and sinks with 0.5 gpm automatic shut-off faucets. Theater E had 3.5 gal/flush toilets, 1.0 gal/use urinals, and sinks with 0.5 gpm automatic shut-off faucets.

TABLE OEC-1
OUTDOOR EDUCATION CENTER
(Recreational Facility for day and overnight groups)

Period of Record	Source	Avg. GPD	Max. GPD
1/7/86-2/17/86	Resident Camper	67.5	89

TABLE SCH-1
SCHOOLS

Source	GPCD	Comments
Elementary School	3.3	Seven day average use, 575 students, limited dishwashing, no showers.
Elementary School	3.2	weekday average use, 106 students Limited dishwashing, no showers.
Elementary School	3.8	Average school day use, period from 8/23/79 - 5/28/87 with range of 265-304 students & Staff. Limited dishwashing, no Showers.
Elementary School	6.2	424 Students, dishwashing, Gym Peak water use = 6.6 GPCD
High School	8.5	Seven day average use, 1,600 students, full dishwashing and Gym showers.
High School	8.5	Seven day average use, 876 Students, limited dishwashing, Gym showers.
High School	5.7	Seven day average use, 1,500 Students, limited dishwashing, Gym showers.
High School	8.0	Seven day average use, 1,200 Students, full dishwashing, Gym showers.

TABLE RC-4
LIFE CARE RETIREMENT COMMUNITIES

(Note: The data in this table was taken from a Concept Design Report for a large scale OWRS in Connecticut. That report provided information for similar facilities in other states for comparative purposes.)

	Facility Location					
Parameter	MN	OH	MI	OH	CT	MA
Number of Apartments	321	173	253	307	199	341
Avg. Number of Apt. Residents	385	219	316	359	228	445
Avg. Number of Residents/Apt.	1.19	1.27	1.25	1.17	1.15	1.30
Number of Health Care (H.C.) Beds	66	60	57	90	60	60
Avg. Number of Health Care Patients	63	58	54	87	55	55
Ratio of Apts. To H.C. Beds	4.86	2.88	4.44	3.41	3.27	5.68
Ratio of Apt. Residents to H.C. Patients	6.11	3.78	5.85	4.13	4.15	8.09
Seasonally Adjusted Avg. Water Use, based on Apt. Residents only, GPCD*	92	69	84	117	150**	67
Seasonally Adjusted Avg. Water Use based on Avg. Number of Apt. residents plus health care patients. GPCD*	79	55	72	94	122**	59

* Based on metered water use from Sept. 11, 1985 to Nov. 21, 1985. Represents indoor water use only.

Seasonally Adjusted Avg. Water Use based on Avg. Number of Apt. residents plus health care patients. GPCD*	79	55	72	94	122**	59

* Based on metered water use from Sept. 11, 1985 to Nov. 21, 1985. Represents indoor water use only.

** The Connecticut facility was a high rise building, and the building plumbing was such that residents in the upper floors had to run their hot water faucets and showers for approximately 10 minutes in order to obtain hot water. Therefore, these results are biased.

Note: An architect with considerable experience in design of health care facilities advised that the historic value for life care facility apartment occupancy was 1.4 persons during the initial period after a facility was opened and that this value declined to an average of 1.2 persons over a period of years due to the deaths of one resident of the apartments initially occupied by couples.

TABLE HC-1

HEALTH CARE FACILITIES (CONVALESCENT HOMES)

Middlesex, New Haven and Hartford Counties

(Metered Water Use Data)

Facility No.	No. Beds	Gal./Day/Bed* Range	Gal./Day/Bed* Average	Wastewater Discharged To
1	30	------	72	On-Site System
2	41	72-82	78	On-Site System
3	90	------	68	On-Site System
4	120	102-111	106	On-Site System
5	60	104-141	129	Municipal Sewer
6	360	114-155	128	Municipal Sewer
7	120	149-195	169	Municipal Sewer

* Includes inside and outside water use.

Note: This data is presented to indicate the difference in water use by facilities served by on-site systems vs. those served by municipal sewers. On-site systems appear to constrain water use, probably because of concern with hydraulically overloading the on-site systems.

TABLE SM-1

SUPERMARKET

New Haven County

(64,000 sq. ft.)

Period Covered	Average Daily Use	gpd/sq. ft.
11 months in 2001-02	2432 gals.	0.038

TABLE R-1

RESTAURANT

Middlesex County

125 seats

(Serves Breakfast, Lunch and Dinner)

Year	Average Daily Use	Gal/Seat
1992	2396 gals.	19.2
1993	2254 gals.	18.0

Note: These data may be biased because the restaurant had been experiencing problems with the on-site system during the years indicated.

I. Predicting Wastewater Flows

1. General

As stated in the introduction to this section, one of the first tasks that need to be completed before beginning actual design of an on-site wastewater renovation system (OWRS) is predicting the wastewater flow that will be generated by the facilities to be served by the system. In this respect, one must not only be concerned with prediction of the quantity of flow, but also its temporal distribution. The most important temporal distribution characteristics are the annual average day and maximum day (the highest annual average day) flow rates and peak hourly flow rates. In certain circumstances, the minimum daily and hourly flow rates can also be important for design of enhanced pretreatment facilities.

Flow rates can be predicted from published information, including not only that available from the literature but that contained in the Connecticut Public Health Code. Flow rates can also be predicted from the results of field measurements of metered water use, or wastewater flows, at similar facilities

The published data on wastewater flows contained herein, or found elsewhere in the literature, may be used for guidance. However, the extreme variability of the circumstances under which such data were gathered, as well as the natural variability of such data, precludes its unquestioned use. This is especially true with respect to wastewater flows generated by commercial and institutional facilities, as the variance in water usage among individual establishments in the same category can be considerable. Such flows vary widely, depending upon mode of operation, number of water using fixtures, hours of operation, occupancy ratios, etc. While there are considerable data available on residential wastewater flows, similar data on commercial and institutional facilities are generally not as extensive. Therefore, it behooves the designer of an OWRS to locate and investigate several facilities similar to that for which he is designing an OWRS and determine either the water use or wastewater flow rate.

2. Field Measurement of Water Use or Wastewater Flows.

When selecting similar facilities to obtain data on wastewater flows or metered water use, it is important to be able to segregate water used for indoor domestic purposes from other water use, such as water used for lawn and garden irrigation and for cooling purposes. It is also important to determine if such data could be biased due either to problems with the wastewater system or with the water supply system serving a facility. A facility that is being served by a failing OWRS, or by restrictions on water use due to an inadequate source of water supply, will probably be using water or discharging wastewater at a rate less than it would absent such problems. Such facilities should not be selected for flow monitoring.

It is also important to survey the types and numbers of water using fixtures, the population using such fixtures, the hours of operation and mode of operation of the facility. It may also be necessary to determine the number of dwelling units, number of bedrooms, the total floor area(s) served, and similar data.

Other factors that should be considered are leakage from water using fixtures or on-site water distribution facilities located downstream from the metering point. Leakage from water using fixtures (i.e. leaking toilets, faucets, etc.) can have a significant effect on measured wastewater discharges. For example, one toilet leaking at the rate of one gallon per minute will waste 1,440 gallons (5,450 L) per day. Water leaking from water distribution facilities can result in the metered water use overstating the actual water use.

Where water use data are based on existing water meter readings, the metering accuracy should be determined. Such accuracy should preferably be equivalent to that required by the American Water Works Association specifications for the size and type of existing meter and the meter installation (setting) should meet minimum standards of the AWWA and the meter manufacturer. If the meter is owned by a municipal or public water utility, the utility should be requested to verify the accuracy of the meter. Where a meter is installed for the purpose of monitoring water use, that meter should conform to the same requirements.

Where municipal or public utility water use billings are used to obtain the water use data, at least three years of such data should be obtained and such data should preferably be based on monthly water meter readings. In such cases, the public utility should be asked to identify those monthly water billings that were based on estimates instead of meter readings. Water meter billing records for the non-irrigation seasons (late fall, winter, and early spring) should be used to determine indoor water use.

Where a facility has maintained daily water meter readings, the facility management should be asked to identify any days when peak water use had been influenced by filling of swimming pools, backwashing of pool filters, fire suppression activities, construction activities, irrigation of vegetation, and similar uses of water that would not result in wastewater discharges.

Where gauging of wastewater flows is conducted, it is important to insure that wastewater flows generated from all inside sources be included. Thus, flow gauging should be conducted at terminal manholes on the wastewater conveyance system, or at the outlet of all septic tanks receiving wastewater discharges. This will require a thorough knowledge of the sanitary drainage piping system(s) in the establishment(s) at which such flow gauging will be conducted. It is also important to determine the same information discussed above with respect to field measurement of metered water use. In addition to leakage into the building sanitary drainage piping due to ground water infiltration or leaking water fixtures, the possibility of leakage out of such piping should also be investigated, as the occurrence of such leakage could result in understating the actual volume of wastewater being discharged. It is also necessary to check for the possible presence of clean cooling water, drainage from ice-making equipment, and other such sources of water that may but should not be discharged to the building sanitary drainage piping.

3. Factors of Safety

Factors of safety should always be included in calculations made for predicting wastewater flows because all of the factors that influence these flows that can not be quantified easily or economically. The magnitude of the safety factor to be used will depend upon the confidence that can be placed upon the available data.

Safety factors are also needed to account for changes in use and occupancy over the design period, which may extend upwards of 20 years. For example, a review of the U.S. 2000 census data indicates that the family size in Connecticut presently averages about 3.08 persons, approximately the same as the overall U.S. average family size of 3.14 persons. This data also indicates that the average household population in the State averages 2.53 persons, again approximately the same as the U.S. average household size of 2.59 persons. These values are significantly lower than those recorded in the census data from past decades. However, it is difficult to estimate what the trend in family or household size will be in the future.

An interesting finding by Linaweaver and Wolfe (1963) was that "there is an inverse relationship between the number of persons in a dwelling and the average daily per capita use, varying from 84 gpcd when there are two persons per dwelling to 47 gpcd for five persons per dwelling. This relationship appears to be fairly constant for different strata and for different seasons of the year." Orndorff (1966) also stated that, as found in previous studies, average water use per dwelling increases with increasing family size, but because the incremental change is smaller with each increasing unit of family size, the average per capita use decreases with increasing family size. While the per capita flow values may well have changed in the intervening years, the relationship should still hold.

Thus, to properly analyze residential water use reported in the literature, it would be helpful to know the dwelling population statistics. Unfortunately, such information is largely lacking from published information. However, when developing a dwelling unit flow allowance based on the number of bedrooms, an occupancy of two persons per bedroom, and a high end constant value of per capita water use, the result will be a dwelling unit flow allowance that includes a large safety factor.

For example, let us assume a three bedroom dwelling housing 3 persons with a per capita design flow contribution that is reduced from 75 gpd to 65 gpd due to the use of ultra low flow toilets. In this case, the design wastewater flow would be 195 gpd. On the other hand, using the design rate of 75 gpcd and assuming two persons occupy each bedroom, the design flow for the dwelling would be 450 gpd. This indicates a design safety factor of 2.3. The safety factor increases substantially as the number of bedrooms is increased. For example, many new dwellings are being constructed with four or more bedrooms, even though the family size may average about 3 persons. Using the design rate of 75 gpcd and assuming two persons occupy each of four bedrooms, the total design flow would be 600 gpd. This indicates a design safety factor of about 3.1 with respect to the design wastewater flow calculated on the basis of 65 gpcd and 3 persons per dwelling.

Likewise, when developing an estimate of wastewater flow for multiple dwelling unit developments, the per capita flow selected should perhaps reflect the dampening effect of multiple dwellings. The greater the number of dwellings to be served, the more the per capita flow allowance will tend toward the average per capita flow.

It is also interesting to note that wastewater discharges from existing office buildings may decrease because of the growing tendency of workers to be based at home rather than in the office. Thus, the application of water or wastewater unit flow allowances based on the square feet of existing office space may become questionable and a per capita allowance may be more appropriate.

The applicant's engineer should consult with the Department regarding the factors of safety to be used.

4. Peak Flow Ratios

a. Maximum Daily Flows

The best prediction of maximum day flow ratios (Maximum Day/Average Day) can be made from analyzing data from similar facilities where daily water use information is available for a period of at least 365 consecutive calendar days at full occupancy of the facilities. Where such data is not available, the following maximum day flow ratios should be considered:

Facility Type	Max. Day Flow Ratio
1. dwelling unit developments (clusters of single family dwellings, retirement and elderly housing units, etc.)	Not less than 1.5
2. commercial and institutional facilities	Not less than 2.0

These maximum day flow ratios should be applied to the design average daily flows acceptable to the Department. The applicant's engineer should consult with the Department regarding the flow ratios to be used in predicting maximum day wastewater flows.

b. Peak Hourly Flows

The prediction of peak hourly flows is difficult because it depends to a great degree upon the number of water using fixtures, their water using characteristics, the frequency of their use and the temporal distribution of such use where a large number of fixtures are involved. Existing peak hourly flows can be determined only from continuous recording of metered water use or wastewater discharges over a significant period of time. Unfortunately, such data is hard to come by because of the time and expense involved. Therefore, in most cases, prediction of peak hourly flows must be based on published values. The applicant's engineer should consult with the Department regarding the flow ratios to be used in predicting peak hour wastewater flows.

5. Infiltration and Inflow

Where a conventional wastewater collection system of pipes and manholes is to be used, allowance must be made for ground water infiltration and surface water inflow. While it is possible to construct such systems with much less infiltration and inflow (i.e. ≤ 50 gallons per inch of sewer diameter per mile of pipe (gpid/mile), experience has indicated that over time, infiltration and inflow increases as the systems age. Therefore, an allowance of not less than 200 gallons per inch of sewer diameter per mile of collection system piping would be appropriate.

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SECTION IV ESTIMATING WASTEWATER CHARACTERISTICS

TABLE OF CONTENTS

TABLES

SECTION IV ESTIMATING WASTEWATER CHARACTERISTICS

A. Introduction

This section provides information on wastewater characteristics for residential, commercial and institutional sources obtained from various published sources. In addition, significant information on wastewater characteristics gleaned from the engineering reports and Discharge Monitoring Reports (DMRs) in the files of the Department for large scale on-site wastewater renovation systems (OWRS) is also presented herein.

It is important to understand that historical data on wastewater characteristics is accurate for the time and place in which they were obtained. In many cases the historical data is based on statistical analyses of the results obtained from grab samples of relatively small sample sizes. These analyses assume the results can be described by arithmetically normal distributions, which is not necessarily true.

Most of the historical OWRS data are based on grab samples of septic tank effluent. The acceptability of characterization of septic tank effluent using grab samples is based on the premise that the septic tank effluent has been “homogenized” by the physical and biological activity that takes place within the septic tank. This may be a reasonable assumption where the wastewater flow rate is low, no large wastewater flow surges occur, the wastewater characteristics are relatively uniform on a temporal basis, and there is ample detention time in the septic tank. These conditions may be approached most of the time in the case of residential wastewater sampling. However, this is generally not the case for septic tanks receiving commercial, institutional and community wastewater.

Some factors that influence the results obtained from sampling of septic tanks include:

1. Configuration of the septic tank(s) sampled, including shape [rectangular or circular cross-section], volume, length to width ratio, liquid depth, number of compartments, the type and arrangement of baffles, the presence of effluent screens, and actual liquid detention time.

2. Frequency of pumping (cleaning) the tank (i.e. too great a depth of solids in the tank adversely effects the pollutant removal efficiency of the tank).

3. Whether the sample(s) were taken shortly after the tank(s) had been pumped.

4. Sampling protocol, including location and depth in which the samples were taken, whether the samples were randomly taken, the preparation and handling of sample containers, and the time elapsed between sampling and testing.

5. Temperature in the septic tank (varies with the seasons).

6. Number of samples taken.

7. Method(s) of analyzing test results.

8. Laboratory accuracy.

Variations in some or all of these factors may cause the sample results to be biased.

While septic tanks serving individual residences typically provide a retention time of 2 to 3 days or more, many septic tanks receiving commercial, institutional and community wastewater have much lower retention times, generally less than 24 hours. For example, the Manual of Septic Tank Practice (U.S. Public Health Service-1972) recommended that, for wastewater flows greater than 1,500 gpd, the minimum effective tank capacity should equal 1,125 gallons plus 75 percent of the daily flow. This recommendation has been widely followed for design of large-scale OWRS. Consider that, for a flow of 5,000 gpd, the recommended volume = 3,750 gal. + 1,125 gal. = 4,875 gal, which would provide a nominal detention time of somewhat less than one day. Thus, if the Manual of Septic Tank Practice recommendation is followed, it can be expected that the percent removal efficiencies for pollutants often discussed for residential septic tanks will not be realized in the case of septic tanks serving commercial, institutional and community facilities.

The intent of providing information herein on characteristics of residential and the domestic fraction of commercial and institutional wastewater is to indicate the wide range in values of such characteristics. It is not intended that such information be used directly to prescribe values for design of an OWRS for a particular facility without substantiation by obtaining field samples of wastewater from existing facilities as nearly similar as possible to that for which the OWRS is being proposed, or from the existing facility where a replacement system, or system upgrade, is required.

B. Residential Wastewater Characteristics

1. Published Information

Information obtained from publications dating from 1981 to the present is shown in Table No. 1 and Table No. 2 on pages 13 and 14 respectively.

2. Data from Department Files

Data available from Department files on residential-type wastewater characteristics is generally derived from multiple dwelling unit facilities such as elderly housing and retirement communities. Data for such facilities are provided in Table No. 4 on Page 16.

C. Commercial and Institutional Wastewater Characteristics

1. General

Characteristics of the domestic fraction of commercial and institutional wastewater, and in cases of wastewater from community systems serving a mixture of residential, commercial and institutional sources, can differ significantly from the values typically used for residential wastewater. Failure to realize this initially during the design of a large scale OWRS system can lead to early failure of the system, regardless of how carefully all other design factors are determined. Therefore, in estimating the wastewater characteristics for a proposed project, very careful attention must be directed to determine the sources contributing wastewater and the proportion and characteristics of wastewater received from each source. In addition, temporal variations in the characteristics must be investigated; this is particularly important when characterization of the wastewater is made by obtaining samples from similar projects.

Data on Food Processing and Serving Establishments are given in Table 3. Data on other commercial and institutional facilities are given in Table 4. Background information on the data contained in these tables is given in "Characteristics of Wastewater from Residential, Commercial and Institutional Facilities"(Jacobson-2002). That paper is available from the Department upon request.

2. Food Processing and Serving Establishments

On-site subsurface wastewater absorption systems (SWAS) serving restaurants and other food processing and serving establishments often fail within a short time after being installed. Failure has been evidenced by severe clogging of the infiltrative surface of the SWAS, resulting in backup of wastewater into the building sewers and/or surfacing of inadequately treated wastewater to the ground above the SWAS. These problems generally resulted from failure to take the wastewater characteristics into account when sizing the on-site facilities such as grease trap(s), septic tank(s) and SWAS.

Food processing and serving establishments can include the following:

- Full Service Restaurants
- Fast Food Restaurants
- Cafeterias
- Diners
- Delicatessens
- Seafood Shops
- Butcher Shops
- Bakeries
- Pie/Pastry Outlets
- Ice Cream Parlors
- Hotels with Restaurants
- Motels with Restaurants
- Clubs with Dining Room Service
- School Kitchens
- Hospital Kitchens
- Nursing Home Kitchens
- Life Care/Retirement Facilities with common dining room service
- Shopping Centers with Supermarkets and/or Restaurants
- Supermarkets
- Travel Centers with Restaurants

Restaurants are by far the most common food processing and serving establishments that experience problems with an on-site SWAS. Restaurant wastewater typically has a higher organic strength (BOD_5) and TSS, and a much higher content of fats, oils and grease (FOG) than residential wastewater. The high FOG content compounds the effect of the high organic strength of restaurant wastewater.

At the high temperatures used for many food-processing operations, animal fats, such as butter and lard, and oils from cooked meat are in liquid form. Such fats and oils tend to solidify as the temperature drops and thus a major portion (60-80%) can be separated from the wastewater by cooling under quiescent conditions in properly designed grease traps. However, in recent times, many restaurants have increased their use of vegetable oils in lieu of solid fats. Vegetable oils are harder to separate, as they are in liquid form at much lower temperatures than animal fats and oils. In some instances, specially designed grease interceptors and other grease recovery devices must be used to remove these oils.

Many restaurants have ineffective means for removing FOG, with the result that relatively high concentrations of FOG can pass through the septic tank serving the restaurant and reach the biomat that forms on the infiltrative surfaces of the SWAS. When this happens, the FOG can clog the biomat and thereby prevent passage of the wastewater through the infiltrative surfaces. In addition, the high oxygen demand exerted by restaurant wastewater can cause anaerobic conditions to exist below the biomat if the infiltrative surfaces of the SWAS have been sized on the basis of typical residential wastewater infiltrative surface hydraulic loading rates.

When such conditions occur, the results will be a reduced ability of the unsaturated soil beneath the SWAS to remove contaminants from the wastewater and degradation of the ground water quality. Where enhanced pretreatment will not be provided to reduce the strength of the restaurant wastewater to or below that of residential wastewater, it is necessary to provide adequate pretreatment for removal of FOG, and reduce the infiltrative surface hydraulic loading rate to account for the high strength of such wastewaters. Additional discussion on pretreatment of restaurant and other food processing establishment wastewaters is contained in Section IX and additional discussion on infiltrative surface loading rates for such wastewaters is contained in Section X.

U.S. EPA (1978) provided the data on the characteristics of raw wastewater from 12 restaurants in three different locations in the U.S. This information is given in Table 3.

Siegrist, et al (1984) investigated the design and performance of septic tank-soil absorption systems for restaurant wastewaters. The investigation consisted of three phases: a preliminary field survey of 42 restaurants; a field investigation of 12 restaurant systems selected from the results of the first phase, and a laboratory experiment using small dia. column type lysimeters. The results of sampling of the 12 restaurants selected in phase 2 are given in Table 3.

Stuth and Guichard (1989) provided information on fast food and full service restaurants in Oregon. This information is given in Table 3.

Garcia and Louch (1994) indicated that a sampling study in St. Louis involving 660 samples taken from untreated wastewater from 88 food establishments found the following FOG concentrations:

- 32% ≤ 200 mg/L
- 29% ranged between 200 -500 mg/L
- 21 % ranged between 500 -1000 mg/L
- 18% ≥ 1000 mg/L

Garcia and Louch (ibid) stated that an ideal temperature of less than 110°F is required to facilitate efficient oil and grease separation**.** They stated that the average FOG removal efficiency of a grease trap was in the range of 70-80% provided that the grease trap is properly designed and cleaned at proper intervals. (It is not clear whether this statement applies to vegetable oils.)

If an average removal efficiency of 75% is assumed, the grease trap effluent FOG concentrations for the raw wastewater FOG concentrations given above should be as follows:

Raw Wastewater mg/L	Grease Trap Effluent mg/L
≤100	<25
≤ 200	≤ 50
200-500	50-125
500-1000	125-250
≥1000	>250

FOG concentration in residential wastewater is ≤ 50 mg/L (usually ranging from 20 to 30 mg/L). From the table above, it can be seen that in order for the FOG in grease trap effluent to approach the FOG concentration in residential wastewater, the raw wastewater FOG from food establishments should be ≤ 200 mg/L, preferably ≤ 100 mg/L. This concentration can only be approached if best waste management practices are established for kitchens and other facilities that generate FOG laden wastewaters.

Laboratory experiments on grease trap effluent from a full service restaurant in Baltimore serving typical American fare yielded the following results (Unpublished - 2002):

Sample No.	Grease Trap Temp. °F.	Grease Trap FOG, mg/L	Cooled Sample* Temp. °F.	Cooled Sample* FOG, mg/L
1	130	1100	75	235
2	125	1050	75	220
3		1175	85	630
			75	275
			65	210

* Samples cooled in Laboratory

Stuth-(1992) stated that if the temperature in the grease trap is below 80°F (27°C) the best results that can be anticipated will be 100 mg/L, and when temperatures exceed 80°F, grease trap efficiency decreases.

Lowery (1994) provided information on influent to an underground grease trap serving the kitchen of a student cafeteria kitchen at a university in Texas. The cafeteria serves 1000 students per week during summer session and up to 25,000 students per week during the normal school year. This information is shown in Table No. 3.

Stuth and Garrison (1995) provided information on full service and fast food restaurants in Oregon. This information is shown in Table No. 3**.**

Stuth and Wecker (1997) surveyed the FOG in kitchen wastewater from six establishments with a range of flows and menus. All six discharged their kitchen gray-water to grease traps, with the effluent then co-mingled with the blackwater from restrooms. The results of grab samples taken at different times on the same day, or on two different days, are shown in Table 3. Stuth and Wecker (ibid.) stated that few conclusions can be drawn from this survey as the FOG, pH and temperatures were significantly different. Stuth and Wecker (ibid.) stated that while grease traps are beneficial, their level of FOG reduction is over-rated. Grease trap recommendations on sizing, multi-compartments, and proximity to facility served could not be drawn from the results of this survey. It was evident that a small-sized grease trap with a limited detention capacity is of limited value in removing FOG.

Chen et al. (2000) provided data on the characteristics of raw restaurant wastewater. They collected a total of 48 samples from five restaurants at the Hong Kong University of Science and Technology for characterization. Since restaurant wastewater is a mixture of wastewater from cleaning meat and vegetables, washing dishes, pans, and other vessels, and rinsing floors, Chen et al. (ibid) expected that the composition of the wastewater would vary significantly depending upon the cuisine served. Also, the food served at a given restaurant depends on the time of service, i.e., breakfast, lunch or dinner. Hence they considered it almost impossible to have one set of data to characterize restaurant wastewater. Instead, they provided a range of values of each parameter for each restaurant.

Matejcek et al (2000) conducted a thorough, well-documented study on long term acceptance rates for restaurant wastewater. Phase I of the study investigated several effluent properties from food service establishments that employ onsite sewage treatment and disposal systems (OSTDS). Septic tank effluent from a total of 19 restaurants located in North Central Florida was sampled. Each restaurant was sampled twice. Results varied greatly between sites, establishment categories and sampling events. Additional qualitative analyses (GCMS) were run to determine the presence of trace organics from degreasers and cleaning agents. The results of the GCMS analyses showed no detectable levels of toxic organics from cleaning products, nor were any compounds detected that might inhibit anaerobic activity or negatively impact effluent characteristics.

The results of statistical analyses showed that the number of samples collected were insufficient to make a statistical determination of variations between establishment categories.

Food Service Establishment Categories Established by Matejcek et al (2000)

Category	Restaurant Type
1	Restaurants operating less than 16 hrs/day
2	Single Service Restaurants operating less than 16 hrs/day
3	Single Service Restaurants operating more than 16 hrs/day
4	Bars and Cocktail Lounges
5	Drive-in Restaurants
6	Food Outlets
7	Convenience Stores

In Phase II of the study, Matejcek et al (ibid) determined wastewater physical and chemical characteristics of 133 samples of septic tank effluent from fifteen randomly chosen food service establishments in Florida. The effluent data were sorted into high, medium- and low-strength categories using carbonaceous biochemical oxygen demand ($CBOD_5$) total suspended solids (TSS) and oils and greases (O&G). Sample collection was changed from single grab samples taken in Phase I to 24-hour composite samples. Sample concentrations over 1200 mg/L $CBOD_5$, 1000 mg/L TSS and 200 mg/L O&G were considered outliers and not a statistical representative sample and therefore were not included in the statistical analysis. The results are shown in Table 3. These results indicate that when 24-hour composite samples are taken, the $CBOD_5$, TSS and O&G values may be less than those of grab sample values as obtained in a manner similar to that used in the Phase I study.

3. Health Care Facilities (Excluding Hospitals)

Health care facilities generate wastewater from such facilities as restrooms, laundries, kitchens and barber/beauty shops. Generally, the wastewater characteristics are similar to medium strength residential wastewater, although in some instances the FOG content may be somewhat greater due to increased use of body oils and lotions that eventually are included in the wastewater due to removal from the body surfaces during bathing.

4. Hotels, Inns and Resorts

Hotels, inns, and resort wastewaters are generated from hotel room restrooms, public restrooms, restrooms in individual retail shops, restaurants, kitchens serving banquet facilities, barber/beauty shop, laundries and other similar facilities. Generally, the wastewater characteristics are similar to medium strength residential wastewater except for the wastewater component from restaurant and other food service facilities.

5. Offices

Wastewater from office buildings is generated in office restrooms, public restrooms, and, in some instances retail shops, restaurants and snack bars. While similar in many respects to residential wastewater, office wastewater is apt to have higher nitrogen concentrations because of the lack of dilution from bath and shower wastewater and other low strength wastewater components found in residential wastewater.

6. Supermarkets

Supermarket wastewater characteristics are highly variable from day to day and throughout the day. In addition to the normal residential type of constituents, this wastewater often contains cleaning agents that can be toxic to wastewater treatment biological processes. The Department is aware of several instances where floor cleaning chemicals and/or sanitizers (quaternary ammonium compounds) have inhibited the biological treatment processes resulting in degradation of the treated effluent. Where wastewater from existing supermarkets is being sampled, analysis should include various types of cleaning compounds. Prior to sampling, an inventory of cleaning compounds used in the establishment should be conducted; this will provide insight into what type of chemicals might be present in the wastewater. Nitrogen and FOG concentrations in supermarket wastewater are apt to be higher than residential wastewater where food processing is done at the supermarket.

7. Shopping Centers and Factory Outlets

Wastewater characteristics from shopping centers and factory outlets can vary widely, depending upon the presence or absence of supermarkets and other food preparation and serving establishments. Where such facilities are present, the wastewater is apt to be higher in organic strength, FOG, and nitrogen, and may contain chemicals that can inhibit microbial action required for adequate wastewater treatment. Refer to discussion on Supermarkets for further information.

8. Travel Centers (aka Truck Stops) and Truck Terminals

Travel centers, also sometimes referred to as truck stops, may generate wastewaters from full service restaurants, fast-food restaurants, ice cream shops, coffee shops, and barber shops, and their associated restrooms, as well as from separate rest room, shower and clothes washing facilities available to truck drivers, and from motels. Thus, estimating the wastewater characteristics for a travel center will require knowledge of the full development potential of the site, including any or all of the uses listed above. It will then be necessary to develop estimates of wastewater characteristics based on each proposed use, and, based on the estimated wastewater flows from each proposed use, develop a composite of each anticipated wastewater constituent. Flows from travel centers can be quite variable, and thus a reasonable safety factor should be included when estimating the wastewater strength. At some travel centers, facilities may be provided for accepting wastewaters from recreational vehicle holding tanks. Such wastewaters may require special consideration. (See 12. Roadside Rest Areas, on page 10.)

Information on wastewater characteristics of Travel Centers located in Texas, Connecticut, Tennessee and Arizona is presented in Table 4. The results obtained at the Texas travel center for BOD_5 and TSS are lower than those obtained at the other three locations, for unknown reasons. Separate samples were also taken and tested for volatile organics (EPA Methods 8010 and 8020). Traces of the substances listed below were detected; all other organics tested for were below the detectable limit.

TTHMs (Total Trihalomethanes)	29 µg/l
Benzene	4.6 µg/l
Ethyl Benzene	17.0 µg/l
Toluene	3.5 µg/l
Xylenes	8.6 µg/l

The sample from the Connecticut travel center was a flow proportioned 24-hour composite sample taken at travel center. Separate samples were also taken and tested for volatile organics (EPA Methods 8010 and 8020) Traces of the substances listed below were detected; all other organics tested for were below the detectable limit.

TTHMs	42.0 µg/l
Toluene	5.9 µg/l

The low levels of the synthetic organic chemicals found in the CT and TX travel center wastewater should not be inhibitory to the wastewater treatment processes and should be removed in treatment of the wastewater.

The Tennessee travel center results for BOD_5 ranged from 235 to 650 mg/L with a median value of 380 mg/L, while the results for TSS ranged from 70 to 707 mg/L with a median value of 285 mg/L. The facilities at the travel center from which these results were obtained included a 150-seat restaurant, 6 fuel islands and a two bay maintenance building. The daily flow was reported to vary from 17,500 to 22,500 gpd.

The BOD_5 concentrations in the Arizona travel center wastewater ranged from 215 to 428 mg/L, the TSS concentrations ranged from 146 to 275 mg/L, and the TN concentrations ranged from 34.0 to 53.1 mg/L. The daily flow during the seven-day period in which composite samples were obtained varied from 14,400 gpd to 24,110 gpd and averaged 18,960 gpd. The facilities at the travel center from which these results were obtained include a 165-seat restaurant, 10 truck-fueling islands, 4 automobile fuel islands and a fast food restaurant. Included in the main terminal building were a restaurant, general shopping area (no ice cream store, coffee shop, or barber shop) 24 toilets, 7 urinals, 8 showers and 2 clothes washing machines. (Test results of grab samples taken periodically by the wastewater treatment plant operator at this facility yielded BOD_5 concentration values ranging from 220 to 2,900 mg/L and TSS concentration values ranging from 87 to 2000 mg/L. These samples were taken for control of plant operations and were not intended to be representative of BOD_5 or TSS concentrations suitable for design purposes. However, these grab sample results are indicative of the wide range in BOD_5 and TSS concentrations that may be encountered at travel centers.

9. Schools

The characteristics of school wastewater will depend upon whether the school has showers and has a kitchen for serving meals to the students. Where the wastewater is generated only in restrooms, without showers, the organic strength and nitrogen content will be higher than normal residential wastewater. The organic strength and nitrogen content will be diluted somewhat if showers are provided, which usually is the case when the school has a developed athletic program. When meals are served, the wastewater may have a FOG content higher than residential wastewater; this will depend upon the type and number of meals served and the method of washing dishes and kitchen clean-up. The same caution should be taken with respect to cleaning compounds as in the case of supermarkets, restaurants and other food preparation and serving establishments.

10. Power Plants

Wastewater generated at power plants can be expected to have higher organic strength and a much higher nitrogen concentration than normal residential wastewater due to the high proportion of blackwater to gray water.

11. Summer Camps

The wastewater from summer camp facilities can be generated in several different facilities, and separate OWRS may be provided for each of these facilities. Characteristics of wastewater from residential cabins will depend upon whether the cabins are equipped only with toilets and urinals or also have showering facilities. Where only blackwater is generated, the wastewater will have a significantly higher organic strength and nitrogen content than normal residential wastewater, while in the case of cabins also equipped with showers, the wastewater strength and nitrogen content will be somewhat lower, but still probably higher than normal residential wastewater. Where an OWRS serves a camp dining hall that will discharge kitchen wastes with perhaps a small blackwater contribution from restrooms in the dining hall, the wastewater organic strength, FOG, and nitrogen content will be substantially greater than residential wastewater.

In the case of kitchen wastewater, the same caution should be taken with respect to cleaning compounds as in the case of supermarkets, restaurants and other food preparation and serving establishments.

12. Roadside Rest Areas, Camp Grounds and Marinas

Roadside rest area wastewater characteristics can vary widely, depending upon whether the area contains restaurants and whether there are provisions for accepting wastes from recreational vehicle holding tanks. In the latter case the wastewater would probably have a higher organic strength and nitrogen concentration than residential wastewater and could contain chemicals that inhibit bacterial action, and that possibility should be considered when reviewing test data on wastewater samples obtained from existing roadside rest areas. This same consideration should be given to wastewaters discharged at campgrounds and marinas.

13. Ski Resorts

Sources of wastewater at ski resorts include restrooms, showers, and food service facilities. The wastewater characteristics are similar to medium to high strength residential wastewaters. Where food service facilities are provided (fast food and/or full service restaurants and other food specialty shops), the wastewater may contain higher FOG concentrations than residential wastewaters. In such cases, the same caution should be taken with respect to cleaning compounds as in the case of supermarkets, restaurants and other food preparation and serving establishments. Where showers are not provided, the organic strength and nitrogen content are apt to be higher than normal residential wastewaters because of the high blackwater content.

D. Sampling for Estimation of Wastewater Characteristics

When an existing on-site system is being upgraded or replaced, the characteristics of the wastewater generated by the facility to be served should be determined from sampling of the facility's wastewater. Composite sampling is preferable if raw wastewater is being sampled. This sampling should be on a flow-weighted basis, and thus data on the changes in water use during the sampling period are required. In most cases, this will require installation of one or more water meters to monitor the variation in hourly water use during the sampling period.

The water meter(s) should also be read and recorded on a daily basis for a reasonable length of time to establish the water use characteristics of the facility, as this information will be needed for design of an upgraded or remedial on-site system. The "reasonable length of time" should include at least three of the busiest months of the facility's business.

In the case of restaurants and other food preparation and serving establishments, where the effluent from an existing grease trap or septic tank is being sampled, a series of grab samples, taken on several days that are representative of the restaurant's busiest days, may be substituted for composite sampling. The grab samples should be taken during the facility's busiest hours and during cleanup operations of each sample day. Water use should also be recorded for the sample days. Wastewater characteristics should include BOD_5, FOG (Fats, Oils and Grease) TSS, TN, TP, pH, temperature and alkalinity. Other data that should be obtained includes the number of restaurant seats, number of meals served per day, a description of kitchen operations and a description and count of the water using facilities in the kitchen and restrooms. The existence of grease traps and septic tanks should be confirmed and the types and liquid capacities determined. The existence of floor drains should be confirmed and the route and discharge endpoint of the floor drain piping should be mapped out. Finally, the types, chemical characteristics and amounts of all cleaners used for various purposes in the restaurant should be determined, along with data on the effects of such cleaners on the viability of anaerobic and aerobic microorganisms.

Procedures for sampling of wastewater from other types of facilities should be similar to those described above. However, information such as occupancy data, hours of operation and information on the numbers and types of water using fixtures from which the wastewater will be discharged is necessary rather than the information specifically applicable to restaurants and other food processing and serving establishments.

Where an on-site system is being designed for a new facility, wastewater flow data and pollutant characteristics must be estimated based on data available from existing similar types of facilities. The first choice would be to obtain this data by sampling the wastewater discharged from one or more facilities of approximately the same size and type. If it can be demonstrated that it is not feasible to obtain such data, it will be necessary to use information developed by others. For this latter case, information presented in Tables 1 through 4 herein may be helpful. It should be noted that the information presented in these tables is quite variable from facility to facility and at any particular facility, as can be seen from the relatively large deviations from the means of the given variables. Therefore, when using such information, an appropriate safety factor should be incorporated in the design of the on-site system to account for such variability.

TABLE No. 1

REVIEW OF CURRENT LITERATURE ON CONCENTRATIONS OF BOD_5 AND TSS IN RESIDENTIAL SEPTIC TANK EFFLUENT

Reference	BOD_5, mg/L No. of Samples	Median	Mean	Standard Deviation	Min.	Max.	TSS, mg/L No. of Samples	Median	Mean	Standard Deviation	Min.	Max.
Hargett, Tyler & Siegrist-1981 ASAE	10		153			92-225	10		44		22	45
Oregon DEQ Study-1982	70		217				70		146			
Hampton & Jones -1984 ASAE		185	164					26	47			
Siegrist, et al -1984 ASAE												
Multiple Home Developments												
Westboro, WI	15		168				15		85			
Bend, OR	4		157				4		36			
Glide, OR	4		118				4		52			
Manila, CA	4		189				4		75			
Washington State	7		129				7		47			
Converse et al. 1991 ASAE	25		150	54	47	239	30		99	102	44	572
Sherman & Anderson 1991 ASAE	36		141		111	181	36		161		64	594
Viraraghavan & Rana 1991 ASAE	44		222	63.4	141	421	44		134	62.6	51	290
Bruen & Piluk 1994 ASAE												
Site A			300						77			
Site B			202						123			
Site C			135						141			
Cagle & Johnson 1994 ASAE												
Placer County Study	15		160				15		73			
Osesek, et al. 1994 ASAE												
Site #1			271									
Site#2			126									
Rubin, et al. - 1995 NW												
1 residential site	10		169		158	178						
Stuth & Garrison-1995 NW												
1 residential site			183		102	264			57		18	80
1 residential site	16	255	243	59.5	165	347	16	57	59	15.7	30	80
Bounds - 1997 NW			156						84			
Loudon, et al. -1997 NW												
Normal Ranges					100	250					30	150
Converse & Converse - 1998 ASAE												
(20 septic tks w/screened vaults)	69	186	215	95	36	548	24	51	61	35	11	135
Jantrania, et al. 1998 ASAE												
Site #1	17		314	250	165	1211	17		81	63	37	285
Site #2	15		143	141	22	530	16		48	36	15	139
Site #3	15		270	119	99	570	16		60	21	37	16
Site #4	15		248	151	102	720	16		592	2067	29	8597
Site #5	10		155	58	120	224	11		53	23	26	108
Site #6	11		89	80	16	305	11		58	33	12	111
Site #7	11		264	64	164	409	11		72	32	16	120
O'Driscoll, et al. 1998 ASAE												
Baldwin County, 10 Residences(93-94)	120		132				120		200			
Tuscaloosa County			331						58			
Roy, et al. 1998 ASAE												
2 Family Home	18		162						92			
Sievers 1998 ASAE			297						44			
Thom, et al. 1998 ASAE												
Paris Site			192	44.1					32	10.5		
Scott Co. Site			193	56.5					68	83.4		
Anderson County			224	58.5					154	147.9		
Stuth - 1999 NW												
21 residential sites (unponded)			141		26	216						
8 residential sites (ponded)			247		150	416						
Henneck, et al. 2001 ASAE												
10 home cluster system (G. Lake)			184	43					27	8		
20 home cluster system (Lake Wash.)			63	31					64	62		
Lindbo & MacConnell 2001 ASAE												
Residential Site #2			114						143			
Residential Site #1			172						80			
Siegrist -2001 ASAE					140	200					50	100
Christopherson, et al. 2001 ASAE												
Winter	96		175	119			96		115	59		
Summer	92		120	88			92		72	65		
Watson and Choate-2001ASAE												
Terrell Site	25		147		13	261	25		255		20	2000
Gray Site	24		103		13	240	24		191		20	1150
Jones Site	17		203		34	382	18		910		31	4800
Mean of Means (unweighted)			183 mg/L*						90 mg/L**			

MANUALS & TEXTBOOKS

Reference	BOD_5, mg/L No. of Samples	Median	Mean	Standard Deviation	Min.	Max.	TSS, mg/L No. of Samples	Median	Mean	Standard Deviation	Min.	Max.
USEPA Manual - 1980, Table 6-1			142		7	480			76		10	485
Cantor and Knox -1985			140						75			
Crites & Tchobanoglous- 1998												
Without Effluent Filter or Garb. Gri.			180		150	250			80		40	140
Without Effluent Filter, w/ Garb. G.			190						85			
With Effluent Filter, w/o Garb. Gri.			130		100	140			30		20	55
With Effluent Filter & Garb. Gri,			140						30			

NOTES:

1.) ASAE = Proceedings of ASAE International Symposiums on Individual and Small community Sewage Systems in year shown.

2.) NW = Proceedings of the Northwest On-Site Wastewater Treatment Short Course and Equipment Exhibitions in year shown.

3.) Crites and Tchobanoglous (1998): with Effluent Screens, the BOD_5 and TSS would be reduced by 28% and 62% respectively.

4.) * Excluding values when septic tank effluent filters were known to be present.

5.) ** Excluding values when septic tank effluent filters were known to be present, and outliers of 592 and 910 mg/L.

TABLE No. 2

REVIEW OF CURRENT LITERATURE ON CONCENTRATIONS OF TOTAL NITROGEN AND PHOSPHORUS IN RESIDENTIAL SEPTIC TANK EFFLUENT

Reference	TN, mg/L						TP, mg/L					
	No. of Samples	Median	Mean	Standard Deviation	Min.	Max.	No. of Samples	Median	Mean	Standard Deviation	Min.	Max.
Hargett, Tyler & Siegrist-1981 ASAE	9*		41		32.8	64.8	11		18.4		8.5	27
Ronayne, et al .Oregon DEQ Study-1982	54		57.5									
Hampton & Jones -1984 ASAE			57*									
Siegrist, et al -1984 ASAE												
Multiple Home Developments												
Westboro, WI	15		57				15		8.1			
Bend, OR	4		41									
Glide, OR	4		50									
Manila, CA												
Washington State	7		34				7		11.4			
Converse et al. 1991 ASAE	30		59		24	132	25		5		3	7
Sherman & Anderson 1991 ASAE	36		36		33	54	36		11		7	15
Viraraghavan & Rana 1991 ASAE	44		46.8	8.8	34	81	44		10.9	2.8	5.2	17.1
Bruen & Piluk 1994 ASAE												
Site A			41.7						7			
Site B			46.9						5.1			
Site C			30.2						13.9			
Cagle & Johnson 1994 ASAE												
Placer County Study	15		61.8									
Osesek, et al. 1994 ASAE												
Site #1			76.6						9			
Site#2			28.7						4			
Rubin, et al. - 1995 NW												
1 residential site	10		48.6		39.8	65.5	10		6.5		5.9	7.7
Loudon, et al. -1997 NW												
Normal Ranges					25	70					5	15
Converse & Converse - 1998 ASAE												
20 septic tks w/screened vaults	70	55	58	23	9.7	144						
* Ammonia-Nitrogen only.												
Jantrania, et al. 1998 ASAE												
Site #1	16		95.6	60.3	52	316	16		8.7	6.6	4.8	33
Site #2	16		39.3	30.7	14	114	16		7.5	4.7	3	24
Site #3	16		153.3	59.8	33	328	16		16.7	7.1	7.4	30
Site #4	16		78.4	73.9	35	330.4	16		10	11	3.5	48
Site #5	11		78.1	9	59	106	11		7.8	1.2	5.2	9.5
Site #6	11		32.1	11.2	13.1	65	11		6.5	1.7	4.9	11
Site #7	11		76.2	12.9	61	97.5	11		11.4	1.9	8.5	15
O'Driscoll, et al. 1998 ASAE												
Baldwin County, 10 Res.-1993-94	120		50									
Roy, et al. 1998 ASAE												
2 Family Home	18		42									
Thom, et al. 1998 ASAE												
Paris Site	>72		46.2	10.9			>72		7.9	5		
Scott Co. Site	>72		70.3	15.8			>72		9.3	3.3		
Anderson County	>24		49.9	17.3			>24		7.4	3.5		
Henneck, et al. 2001 ASAE												
10 S.F. Home cluster system(G.Lake)	81		59	12			81		7.9	1.4		
20 S.F. Home cluster system (L. Wash.)	50		33	11			50		5.4	1.5		
Lindbo & MacConnell 2001 ASAE												
Residential Site #1			27.4						1.9			
Residential Sites #2,3, & 4			29.2						4.4			
Christopherson, et al. 2001 ASAE												
Winter	96	51		43			96	9		24		
Summer	92	47		36			91	8		5		
Siegrist -2001 ASAE					46	100					5	15
Mean of Means (unweighted)++			50.9						8.8			

MANUALS & TEXTBOOKS

Reference	TN, mg/L						TP, mg/L					
	No. of Samples	Median	Mean	Standard Deviation	Min.	Max.	No. of Samples	Median	Mean	Standard Deviation	Min.	Max.
USEPA Manual - 1980, Table 6-1	150		42		9	125						
Cantor and Knox -1985			40						15			
Crites & Tchobanoglous- 1998												
Without Effluent Filter or Garb. Gri.			68		50	90			16		12	20
Without Effluent Filter, w/ Garb. G.			75		50	90			16		12	20
With Effluent Filter, w/o Garb. Gri.			68		50	90			16		12	20
With Effluent Filter & Garb. Gri,			75		50	90			16		12	20

NOTES:

1.) ASAE = Proceedings of ASAE International Symposiums on Individual and Small community Sewage Systems in year shown.

2.) NW = Proceedings of the Northwest On-Site Wastewater Treatment Short Course and Equipment Exhibitions in year shown.

3.) ++ Excluding outliers of 153.3 for TN and 1.9 for TP.

TABLE No. 3

Wastewater Characteristics of Food Processing and Serving Establishments

Ref.	Facility Type	BOD_5, mg/L						TSS, mg/L					FOG, mg/L					Mean	Mean
No.		No. of Samples	Sample Type	Mean	Std. Dev.	Min.	Max.	No. of Samples	Mean	Std. Dev.	Min.	Max.	No. of Samples	Mean	Std. Dev.	Min.	Max.	TKN/TN mg/L	TP mg/L
	Restaurants																		
R-1a	2 Restaurants in Honolulu, HI	10	R,C	640		525	759	10	500		202	800							
R-1b	5 Restaurants in Greensboro, NC	15	R,C	546		390	737	15	257		48	402							
R-1c	5 Restaurants in Philadelphia, PA	10	R,C	655		280	960	10	1,030		172	1,985							
R-2	12 Restaurants in Wisconsin																		
R-2a	Restaurants only	37	STE,G	506		245	880	36	177		28	962	32	83		26	256		
R-2b	Restaurants w/other Facilities	25	STE,G	196		101	333	25	73		9	176	25	39	39	3	96		
R-3	Restaurants in Oregon																		
R3-a	Full Service Restaurant		STE,G	1,074					289										
R3-b	Full Service Restaurant		STE,G	1,301					350										
R3-c	Fast Food Restaurant		STE,G	1,917					624										
R3-d	Fast Food Restaurant		STE,G	1,716					358										
R-4a	Full Service Restaurant		GTE,G	1,657					382										
R-4b	Full Service Restaurant		STE,G	1,377					120										
R-5	Student Cafeteria, Univ. in Texas																		
R5-1	Summer Session	15	R,G	576				15	460										
R5-2	Beginning of Fall Semester	25	R,G	992				25	620										
R5-3	During Fall Semester	13	R,G	1,628				13	992										
R-6	Restaurants in Oregon																		
R6-1	Full Service Restaurant	22	GTE,G	913			1,800	23	185			774	22	207			378		
R6-2	Fast Food Restaurant	7	STE,G	985			1,216	7	143			195	7	138					
R-7	Restaurant Kitchen Greywater																		
R7-1	Full Service, American Cusine	2	GTE,G										2	2487		1,424	3,550		
R7-2	National Fast Food Franchise	2	GTE,G										2	1,270		297	2,242		
R7-3	Full Service, American Cusine	2	GTE,G										2	193		152	234		
R7-4	International Fast Food Franchise	2	GTE,G																
R7-4a	First Grease Trap Effluent	2	GTE,G										2	712		692	732		
R7-4b	Second Grease Trap Effluent	2	GTE,G										2	323		306	340		
R7-5	Full Service, American Cusine	2	GTE,G										2	12,802		10,646	14,958		
R-8	Restaurants in Hong Kong																		
8-1	Chinese Restaurant	10	R,U			58	1,430	10			13	246				120	712		
R8-2	Western Cusine Restaurant	10	R,U			489	1,410	10			152	545				53	2,100		
R8-3	American Fast Food Restaurant	11	R,U			405	2,240	11			68	345				158	799		
R8-4	Student Canteen	14	R,U			900	3,250	14			124	1,320				415	1,970		
R8-5	Bistro	3	R,U			1,500	1,760	3			359	567				140	410		
R-9	Restaurants in Florida																		
R9-1	Restaurants operating <16 hrs/d	U	STE,G	761	266				226	19				83	75				
R9-2	Single Serv. Rest. Oper <16 hrs/d	U	STE,G	602	313				123	125				33	35				
R9-3	Single Serv. Rest. Oper>16 hrs/d	U	STE,G	548	290				141	158				80	94				
R9-4	Bars and Cocktail Lounges	U	STE,G	451	71				79	38				24					
R9-5	Drive-in Restaurants	U	STE,G	1,920	1,273				454	269				78	67				
R9-6	Convenience Stores	U	STE,G	441	237				43	20				18	18				
R10	15 Restaurants in Florida	109	STE,C	374	255	53	1009	128	77	49	9	268	122	36	33	5	196		
R11	Full Service Restaurant in CT	39	STE,G	362	149	97	729	39	192	141	18	670							
R12a	Kitchen in Full Service Restaurant in CT	1	STE,C	960				1	240										
R12b	Kitchen in Full Service Restaurant in CT	1	STE,G	878				1	116										
R13	Full Service Restaurant in CT																		
	Kitchen Graywater(Same Day)	4	GTE,G	925		790	1000	4	118		87	136	4	30		<3	60		
	Graywater and Blackwater(Same Day)	4	STI,G	700		520	800	4	93		64	117							
R14	Full Service Restaurant in Baltimore	7	R,G	1320		704	1679	7	490		223	722	7	328		96	469		
R15	Full Service Restaurant in Baltimore	10	GTE,G										7	187	128	85	510		
R16	Fast Food Restaurant in CT	1	STE,C	430				1	40									41	8.4
R17	Oriental Restaurant in CT	1	GTE,G	1380				2	106									52	13.2
R18	Fast Food Restaurant in Michigan																		
R18-a	Kitchen Graywater	6	R,G	3960				6	2090				6	460					3.4
R18-b	Washing Machine Effluent	6	R,G	2525				6	806				6	461					2.7

*** R=Raw; GTE = Grease Trap Effluent; STI = Septic Tank Influent; STE = Septic Tank Effluent; C = Composite; G=Grab, U = Unknown**

Ref No.	Reference (See Bibliography)
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R-2	Siegrist, et al. (1984)
R-3	Stuth and Guichard (1989)
R-4	Stuth and Guichard (1989)
R-5	Lowery (1994)
R-6	Stuth and Garrison (1995)
R-7	Stuth and Wecker (1997)
R-8	Chen et al. (2000)
R-9	Matejcek et al. (2000)
R-10	Matejcek et al. (2000)
R-11	CT DEP Files
R-12	CT DEP Files
R-13	CT DEP Files
R-14	Unpublished (2002)
R-15	Unpublished (2002)
R-16	Unpublished (2002)
R-17	Unpublished (2002)
R-18	Unpublished (2002)

TABLE No.4

Wastewater Characteristics of Commercial and Institutional Facilities

Ref. No.	Facility Type	BOD$_5$, mg/L						TSS, mg/L					FOG, mg/L					Mean TKN/TN mg/L	Mean TP mg/L
		No. of Samples	Sample Type	Mean	Std. Dev.	Min.	Max.	No. of Samples	Mean	Std. Dev.	Min.	Max.	No. of Samples	Mean	Std. Dev.	Min.	Max.		
HC-1	Skilled Nursing Facility	17	STE,G	171	114	64	271	17	100	99	14	426	17	13	9.6	2	37	35	N.A.
HC-2	Life Care Facility	26	R,G	154	62	41	272	26	159	58	74	288						34	N.A.
HC-3	Health Care Facility																		
	Facility No. 1	2	R,C	218				2	84				2					32	2.6
	Facility No. 2	1	R,C	276				1	199				1	10				43	9.5
	Facility No. 3	2	R,C	197				2	134				2					26	6.6
	Facility No. 4	1	R,C	159				1	72									28	8.3
	Facility No. 5	2	R,G	151				2	374									31	1.9
	Facility No. 6	2	R,G	432				2	638									38	7.6
I/ R-1	Inn & Resort w/Full Service Restaurant	20	R,G	195	147	41	726	20	249	303	20	1,200						62	11.8
I/ R-2	Inn w/Full Service Restaurant	10	STE,G	194	104	86	433	10	93	151	26	520						41	6.9
I/R-3	Inn w/no Restaurant		R,G	221		130	340		154		<5	274						33	N.A.
O-1	15,000 SF Office Building		R,C	240					96									97	10.0
	15,000 SF Office Building		STE,C	150					30									112	11.8
SM-1	Supermarkets in CT, MA, RI																		
SM1-a	Supermarket in CT	8	STE,G	479				8	156					64				39	N.A.
SM1-b	Supermarket in CT		STE,G	576															
SM1-c	Supermarket in CT		STE,G	164					66									55	N.A.
SM1-d	Supermarket in CT	17	STE,G	646				17	162									81	N.A.
SM1-e	Supermarket in MA	9	STE,G	250				9	132									69	N.A.
SM1-f	Supermarket in MA	8	STE,G	426				8	104									53	N.A.
SM1-g	Supermarket in MA	8	STE,G	215				8	86									69	N.A.
SM1-h	Supermarket in MA		STE,G	433															
SM1-I	Supermarket in RI		STE,G	720															
SM-2	Supermarket in CT																		
SM2-a	Influent to ST #1	3	R,G	838				3	172									85	29.5
SM2-b	Effluent from ST#2	3	STE,G	712				3	98									148	29.4
SM-3	Supermarket in CT																		
SM-3a		19	R,G	1132	650	149	2,571	20	313	255	25	1,075						245	N.A.
SM-3b		22	STE,G	883	338	582	2,166	24	178		13	480						189	N.A.
SHPG-1	Shopping Center in CT	46	STE,G	442	219	150	1,260	46	157	99	40	460						51	7.3
SHPG-2	Factory Outlet Complex in CT	4	R,G	118	17	108	143	4	99	55	49	175						117	38.9
SHPG-3	Factory Outlet Complex in CT	23	R,G	409	172	172	795	23	470	556	47	2,480						173	36.9
TC/TT-1	Travel Center in CT	3	R,G	>593				3	374									87	10
TC/TT-2	Express Delivery Truck Terminal	13	R,G	257		70	572	13	350		60	980						68	9.3
TC/TT-3	Travel Centers in TX, CT,TN, AZ																		
TC/TT-3a	Travel Center in Texas	1	U	240				1	120									39	4.1
TC/TT-3b	Travel Center in CT	1	R,C	332				1	294									59	7.9
TC/TT-3c	Travel Center in Tennessee	27	R,U	469				27	346									N.A.	N.A.
TC/TT-3ad	Travel Center in Arizona	7	R,C	349				7	215									40.3	N.A.
SCH-1	Middle School and High School in CT																		
SCH-1a	Middle School		STE,G	215					40									88	17.9
SCH-1b	Middle School		STE,G	115					110									133	3.1
SCH-1b	High School		STE,G	225		70												80	15.4
SCH-2	High School																		
	Septic Tank #1	2	STE,G	220		170	270	2	30		14	46	1	11				84	N.A.
	Septic Tank #2	1	STE,G	90					24									110	N.A.
	Septic Tank #3	2	STE,G	175		130	220	2	33				2	9				86	N.A.
SCH-3	Consolidated School																		
	Septic Tank #1	2	STE,G	146		126	165												
	Septic Tank #2	2	STE,G	117		105	128	2	59		38	80						108	N.A.
SCH-4	Middle School in CT	23	STE,G	304		92	599	24	135		19	1,960						141	N.A.
SCH-5	Boarding School in CT	8	R,G	329		184	510	8	177		121	240							
SCH-6	Schools in Vermont																		
	2 Elem., 2 High and 1 Private																	83	7.5
PP-1	Electrical Generating Facility, CT	12	R,G	324				12	305									136	N.A.
CMP-1	Summer Camp Dining Hall	3	R,G	1,633		1,500	1,800	3	465		74	1,200	2	106		41	170	79	14
	Summer Camp Dining Hall	3	STE,G	1,256		1,070	1400	3	70		33	100	2	17		17	34	76	18
CMP-2	Campground Holding Tank Pumpouts	3	STE,G	717		377	1117						3	91		8	240	650	74
MARINA	Marinas (2), Pump-out only	2	STE,G	648		395	901						2	65		40	91	610	66
	Marinas (4), Pump-out & Rest Rooms	4	STE,G	336		118	644						4	71		6	130	250	27
RRA-1	Interstate Roadside Rest Area, CT	2	STE,G	235		190	280	2	88		86	90	1	15				100	8.7
SKI-1	Ski Resorts																		
SKI-1a	Ski Resort in Oregon	U	R,U	395				U	321									77	12.7
SKI-1b	Ski Resort in Washington	U	R,U	382				U	372									80	13.2
SKI-2	Ski Resort in Vermont	14	R,U	242	53	151	347	14	196	81	68	330							

*** R=Raw; GTE = Grease Trap Effluent; STE = Septic Tank Effluent; C = Composite; G=Grab, U = Unknown**

Ref No.	Reference (See Bibliography)
HC-1	CT DEP Files
HC-2	CT DEP Files
HC-3	Unpublished (2002)
I/R-1	CT DEP Files
I/R-2	CT DEP Files
I/R-3	CT DEP Files
O-1	Unpublished (2002)
SM-1a to 1i	CT DEP Files
SM-2	CT DEP Files
SM-3	Unpublished (2002)
SHPG-1	CT DEP Files
SHPG-2	CT DEP Files
SHPG-3	CT DEP Files
TC/TT-1	CT DEP Files
TC/TT-2	CT DEP Files
TC/TT-3	Unpublished (2002)

Ref. No.	Reference (See Bibliography)
SCH-1	CT DEP Files
SCH-2	CT DEP Files
SCH-3	CT DEP Files
SCH-4	CT DEP Files
SCH-5	CT DEP Files
SCH-6	Unpublished (2002)
PP-1	Unpublished (2002)
CMP-1	Unpublished (2002)
CMP-2	Matassa, McEntyre and Watson
MARINA	Matassa, McEntyre and Watson
RRA-1	Unpublished (2002)
SKI-1	Clark (1969)
SKI-2	Unpublished (2002)

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SECTION V BASIC GROUND WATER HYDROLOGY
FOR
THE DESIGN OF SUBSURFACE WASTEWATER ABSORPTION SYSTEMS

TABLE OF CONTENTS

FIGURES

TABLES

SECTION V BASIC GROUND WATER HYDROLOGY FOR THE DESIGN OF SUBSURFACE WASTEWATER ABSORPTION SYSTEMS

A. Introduction

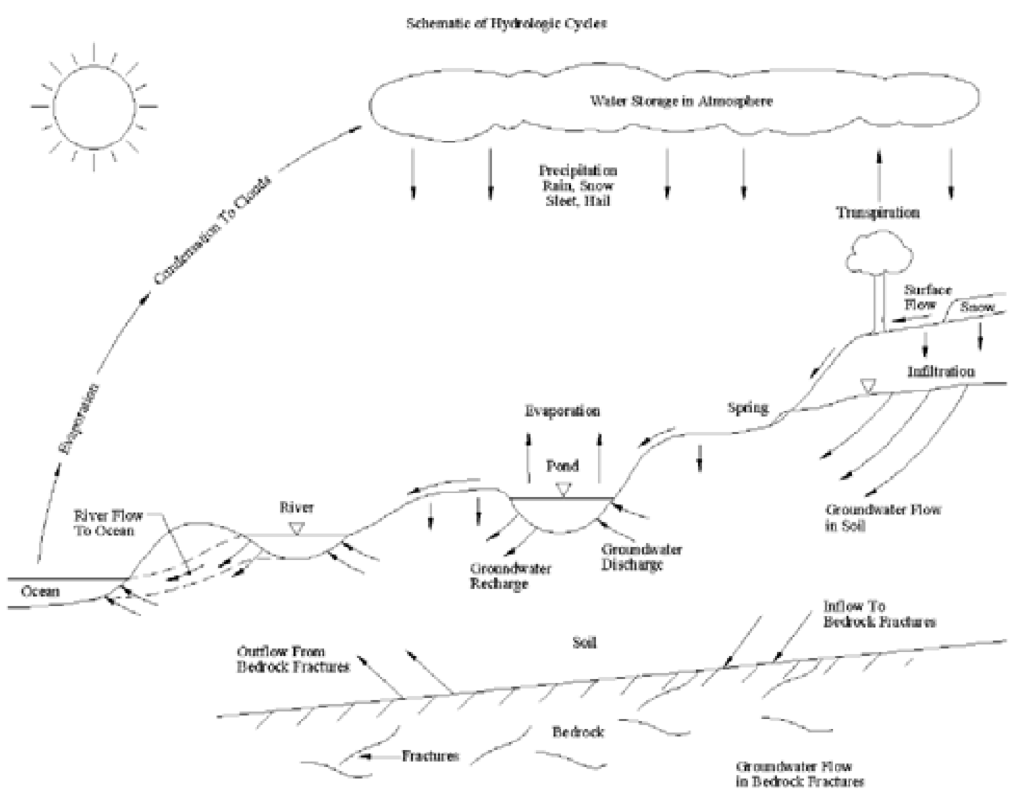
Some knowledge of the basic principles of ground water hydrology must form the background on which an engineer will make decisions about an OWRS.

The hydrologic cycle is the continuous movement of water from the ocean to the land and back. The ocean water is evaporated by the sun, precipitates on the land, and flows back to the ocean under the influence of gravity. As the water moves through this cycle there are many sub-cycles, as shown in Figure No. 1.

Water is stored for varying lengths of time throughout the cycle in the atmosphere, ice, surface water and ground water, but the amount of water in storage at any specific location changes as the weather changes.

FIGURE No. 1

THE HYDROLOGIC CYCLE

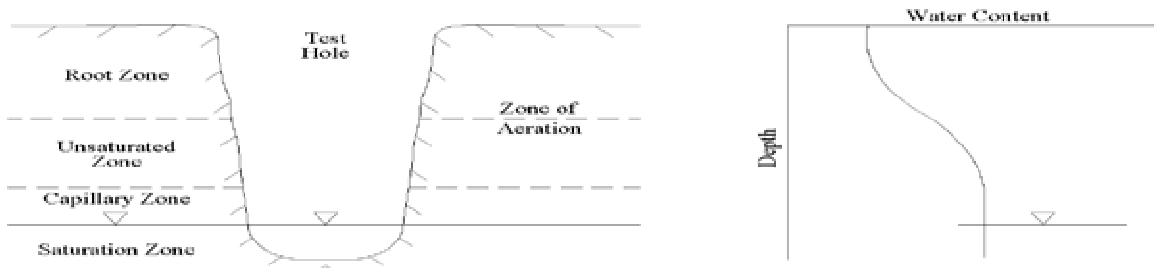


B. Soil Water

Soil is composed of solid mineral particles with voids or pores between them. Precipitation that infiltrates the ground surface may be held near the surface in the root zone or may move downward through a zone of aeration, in which the voids are not filled with water, to the zone of saturation in which the voids are essentially full of water. The Zones are shown in Figure No. 2.

<u>FIGURE No. 2.</u>

SOIL WATER



Within the root zone the water is discontinuous and is held in the form of meniscus between soil particles. The air passages are interconnected and generally continuous throughout the zone. This same situation continues in an unsaturated zone of aeration beneath the root zone, until a capillary zone is reached.

In the capillary zone, the water is drawn up from the saturated zone below by the capillary action in the soil voids. Within this zone, the water is generally continuous and at less than atmospheric pressure. The air is in the form of individual bubbles. This zone may range from less than an inch high in sand to more than 20 feet high in clay.

In the saturated zone what little air exists is in the form of individual bubbles and the water is at a pressure greater than atmospheric. The ground water table, the upper limit of this zone, is by definition the elevation at which the water pressure is atmospheric. It is the elevation to which the water level will rise in a test hole. The majority of the lateral movement of ground water occurs within the saturated zone, There is no sharp demarcation between these three zones, as a plot of water content versus depth in Figure No. 2 indicates.

There are two major purifying processes within the hydrologic cycles**.** Evaporation from the ocean and lakes brings distilled water into the atmosphere. Rain that infiltrates the ground is cleansed by purification that takes place as it passes through the soil. Such purification is accomplished by mechanical filtering, biological activity, and chemical absorption and adsorption. The soil generally does not remove the highly soluble salts. Other purification processes may also take place in wetlands, surface water bodies, etc.

In this document we are concerned primarily with how water enters a subsurface wastewater absorption system (SWAS) area, and how it leaves. Water can enter an area as rain, surface water or ground water, and can leave by evaporation, evapotranspiration, surface water or ground water. If the water is entering an area faster than it is leaving, the amount of water in storage will increase in the form of a rising ground water table or pond level. The opposite occurs if the water is leaving faster than it is entering. Rarely does Q in (Q= Volume/Time) equal Q out so the amount of water in storage is always changing; however, it is convenient and reasonably accurate to solve many hydrology problems by assuming that for a short time period storage is constant.

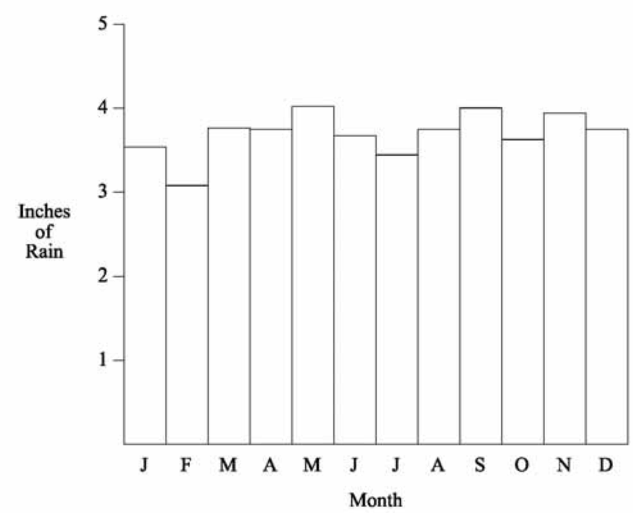
The rate at which water enters and leaves a site varies from day to day but the average value of Q in and Q out over a short period of time can be calculated by taking into account the following factors.

C. Precipitation

The mean monthly precipitation in Connecticut is relatively uniform throughout the year. An example of this uniformity is shown in the bar graph of Figure No. 3.

FIGURE No. 3

MEAN MONTHLY PRECIPITATION AT BRADLEY INTERNATIONAL AIRPORT (1961-2000)



D. Infiltration

Water on the ground surface either infiltrates the ground or runs off as surface flow to ponds or streams. The infiltration rate depends primarily on the temperature, position of the ground water table, vegetation and type of soil, ground slope and impermeable cover and will vary from season to season. Typical values are given in Table No. 1 for uniform bare soil with the ground water table well below the surface.

TABLE No. 1

TYPICAL INFILTRATION VALUES

Soil Type	Infiltration Rate-in/hour
Sandy soils	0.50-1.00
Clay & silt loams	0.10-0.50
Clays	0.01-0.10

The rate of infiltration for all but clay exceeds the rate of rainfall most of the time, which means that there will be very little surface runoff during moderate rain in unfrozen soil in areas where the ground water is well below ground surface.

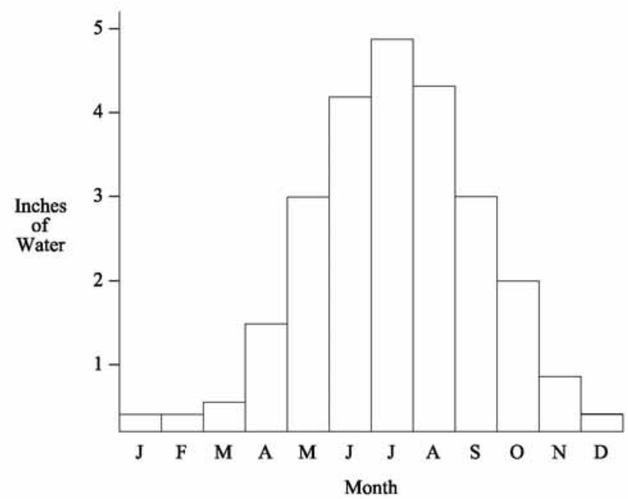
E. Evapotranspiration

Evapotranspiration is the loss of water through evaporation from surface water and vegetation. The rate of evapotranspiration varies greatly throughout the year and depends on the type of vegetation, soil type, and position of the ground water table. The average monthly rate of evapotranspiration for a well-vegetated area in the Northeast is given in Figure No. 4.

If the average monthly precipitation is superimposed on the evapotranspiration chart it can be seen that, in the months of May, June, July, August, and September, the total evaporation exceeds the precipitation and these are the drying months when the pond levels and ground water tables drop. The wet season typically is caused not by more precipitation but by less evapotranspiration.

FIGURE No. 4

AVERAGE MONTHLY EVAPOTRANSPIRATION



F. Surface Flow

When rainfall exceeds the infiltration rate, the water runs off the surface into streams and rivers. Surface flow also occurs when the ground water flow exceeds the capacity of the ground to carry it and the ground water breaks the ground surface. There are many seasonal watercourses that flow only when the melt water and precipitation in the spring exceed the infiltration capacity of the surrounding ground. Surface flow by definition indicates the ground water table is at ground surface so the presence of surface water flow generally precludes the use of such areas for a SWAS without major changes.

G. Ground Water

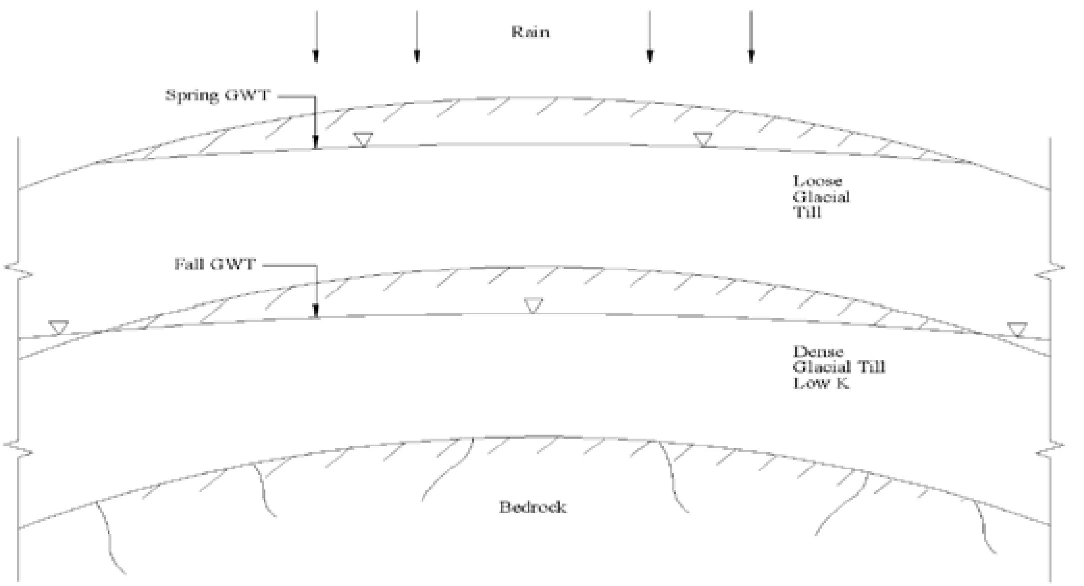
Ground water flows under the forces of gravity through the voids in the soil or rock. In the glaciated areas of New England, the surficial soils are generally much more permeable than the soils or rocks underlying them so that, except in deposits of sand and gravel, most of the ground water flow is parallel to the ground surface and occurs in the top 10-20 feet of soil. The rate at which ground water can enter or leave a site depends on the slope of the water table, the hydraulic conductivity of the soil, and the thickness of the saturated soil. It should be noted that ground water flow in soil voids differs from ground water flow in rock voids; the latter being considerably more difficult to predict.

The hydraulic conductivity of the soil is the greatest variable in ground water flow, and, particularly in New England, the soil can be very heterogeneous within a given area. The effects of heterogeneity on the ground water flow are illustrated in the following figures.

Figure No. 5 shows the cross section of a drumlin composed of glacial till over bedrock. The upper 4-6 feet of the till have been loosened and lightly granulated by frost action air, and bioturbation and its hydraulic conductivity is several hundred times greater than the underlying till, which is still very dense due to the original depositional forces of the glaciers.

FIGURE No. 5

CROSS-SECTION OF A DRUMLIN

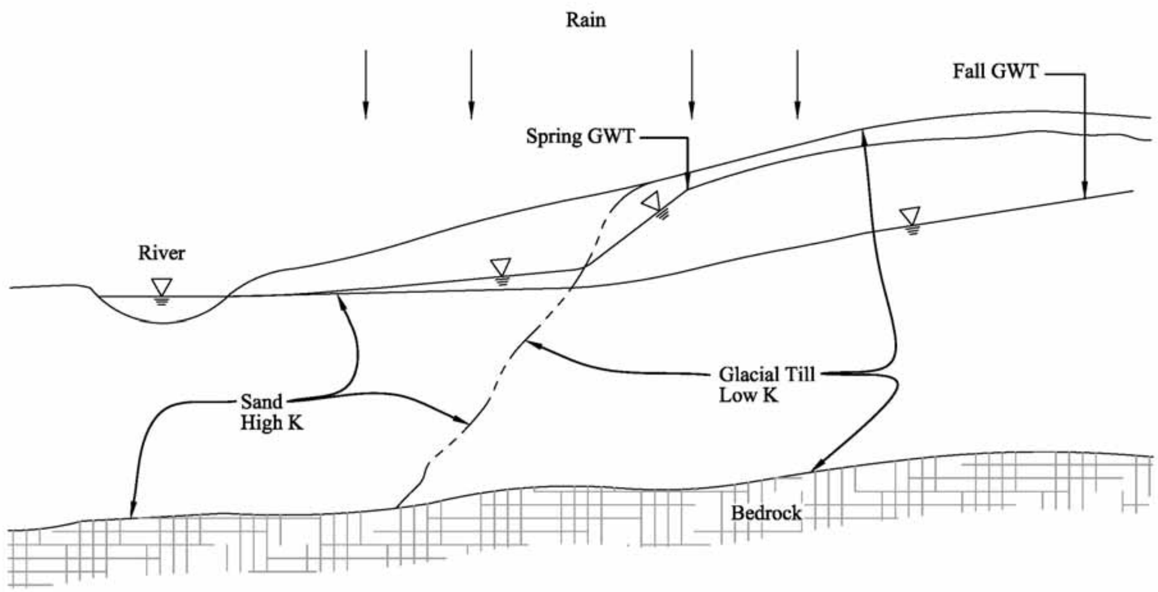


During the wet season, precipitation infiltrates the upper part of the hill faster than it can flow laterally and downward within the soil on the hillside, and the ground water table intersects the ground surface part way down the hill. During the dry summer months, the ground water table gradually drops below the top of the underlying till as the water flows away faster than it is supplied.

Figure No. 6 shows the cross section of a hill with sand and gravel along the river downhill from glacial till. The hydraulic conductivity of the sand and gravel is several hundred times greater than that of the till. During the wet season the ground water that is at the surface of the till can flow easily through the sand and gravel. During the dry season, the water table drops in the glacial till (10-20’) whereas the water table in the sand and gravel drops very little due to the nearness of the river that is supplied with water from a much larger area. During certain periods, the river may be recharging the sand and gravel deposit.

FIGURE No. 6

CROSS-SECTION OF A HILL ABUTTING A RIVER



Discharge of wastewater into a SWAS is a man-made sub-cycle in the hydrologic cycle and follows the same physical laws as other water does. The path that the discharged water will follow can be predicted, based on knowledge of the soil properties and the original hydrologic conditions.

H. References

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SECTION VI SITE HYDRAULIC CAPACITY ANALYSIS

TABLE OF CONTENTS

SECTION VI SITE HYDRAULIC CAPACITY ANALYSIS

TABLE OF CONTENTS

FIGURES

TABLES

SECTION VI SITE HYDRAULIC CAPACITY ANALYSIS

A. Introduction

This section discusses methods that can be used to determine the hydraulic conductivity of soils and methods available for analysis of the hydraulic capacity of a potential site of a subsurface wastewater absorption system (SWAS).

As discussed in Section II, the hydraulic capacity of a site is the first of the two basic "capacity factors" that determine the soil's ability to accept and renovate pretreated wastewater discharged to a SWAS. The second factor is the soil's renovative capacity; its ability to remove or largely attenuate the contaminants found in domestic wastewater. The hydraulic capacity factor deals with the soil's ability to accept, contain and transport the wastewater percolating from a SWAS in such a manner that renovation will occur before the percolate reaches a point of public health or environmental concern.

In the past, the terms "permeability", or "coefficient of permeability" were often used to define the soil characteristic used in the Darcy equation to determine the hydraulic capacity of a soil. Hydraulic Conductivity (K) is the current designation of a soil's ability to transmit water (NRCS -1998). It is a mathematical coefficient that relates the rate of water movement to the hydraulic gradient and is one of the terms in Darcy's law on flow through a porous medium.

To confound this issue of terminology, hydraulic conductivity is determined in the laboratory by the use of a "permeameter", and ASTM uses the terms "Permeability" in the title of one Standard Test Method (D2434) and "Hydraulic Conductivity" in the title of another method (D5084) to determine the same soil property. Also, a soil that has very little to no capacity to transmit water is described as "impermeable". Further, the former Soil Conservation Service (SCS) published soil surveys for each of the eight counties in Connecticut that provided data for soil permeabilities that were actually values for hydraulic conductivities. (However, the information in these soil surveys is presently being updated by the NCRS and now refers to the former SCS soil permeability data as "Hydraulic Conductivity" data.)

The original "Seepage and Pollutant Renovation" document issued by the Department (Healy and May - 1982, rev. 1997) also used the term permeability as synonymous with hydraulic conductivity. This section is primarily an update of the original section on "Permeability Testing" and "Seepage Analysis" in the Healy and May document that has been in use for over two decades by many SWAS designers in Connecticut. In this update, the words "permeability" and "permeabilities" has been replaced with "hydraulic conductivity" and "hydraulic conductivities". Further, the term "seepage analysis" has been replaced with the term "hydraulic capacity analysis".

B. Hydraulic Conductivity Testing

The determination of soil hydraulic conductivity is critical to the evaluation of how a SWAS will work. Determining the hydraulic conductivity, or K, is a difficult task involving testing and measurement, tempered by reasonable judgment. Most engineers are familiar with methods of laboratory testing for hydraulic conductivity. These methods are often applied and valid when an earthen material is to be excavated, transported, and re-compacted to a known density. Examples of this are construction of a road base or earthen dam. The problem in discharging pretreated wastewater to subsurface soils is quite different. The effort here is to determine a range of hydraulic conductivities for in-place soils that will not be substantially altered by man. The testing culminates in a judgment process about what values are reasonable and what numbers should be utilized in design.

The process of deciding what, where, and how to test, then being confronted by a wide range of values, causes many engineers great difficulty. There will always be uncertainty arising from a best effort to understand natural phenomena and deal with them. It should be remembered that with experience, and after forming a reasonable mental picture of how a site handles water, confidence in hydraulic conductivity results will be forthcoming.

For application to the Department it is preferable to attempt to determine hydraulic conductivity by three different tests, observation, or measurements. All methods and measurements should be contained in an engineering report and clearly explained.

Inevitably during any discussion of hydraulic conductivity testing, confusion occurs over the issue of whether a percolation (perc) test is a hydraulic conductivity test. A perc test is not a hydraulic conductivity test and in fact measures no parameter of soil. Percolation tests are run with unknown boundary conditions and are greatly affected by soil moisture and capillarity. Hydraulic conductivity tests are run with defined boundaries under saturated conditions. These two tests should never be confused. The perc test is a procedure that has, by trial and error, been given an empirical relationship to the amount of wastewater that can be applied at a SWAS interface. It has limited relation to the hydraulic capacity of a site. The Department requires that site hydraulic capacity for large-scale OWRS be based on reasonable values for the soil's saturated hydraulic conductivity.

C. Procedures for Hydraulic Conductivity Tests

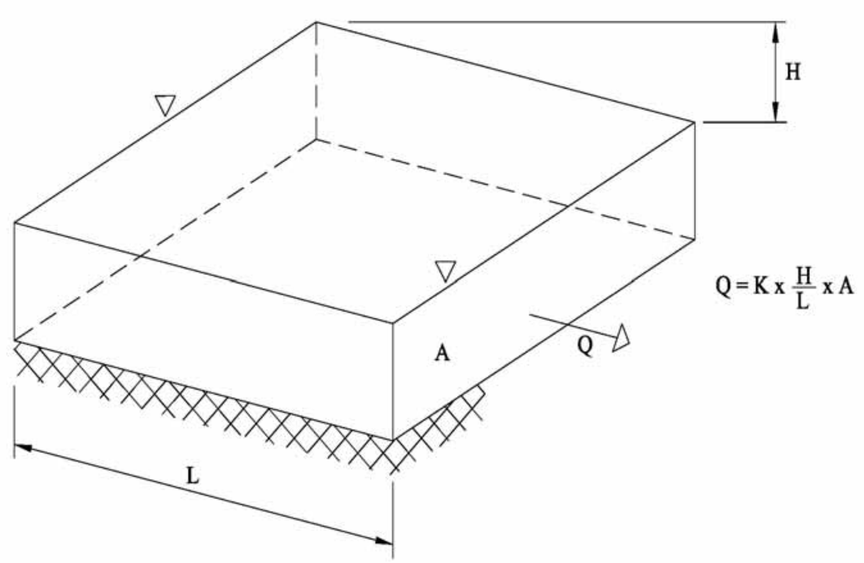
The test procedures discussed in this section are simple, practical, require low capital expenditure, and are well within the capability of any professional engineer. These procedures yield hydraulic conductivity values of suitable accuracy for the intended use. The saturated hydraulic conductivity of a soil is measured by filling the voids with water and measuring the steady rate of water flow through a soil sample of known dimensions under known hydraulic conditions.

There are some field situations under which water flows through unsaturated soil, such as when rain falls on a soil at a slower rate than the soil can absorb it, or during intermittent application of effluent in a disposal system when slugs of water passing through the soil are separated by unsaturated zones. Steady state saturated flow does not simulate either one of these situations; however, flow rates calculated assuming saturated steady state flow are the maximum flow rates possible for given hydraulic conditions, and can be used for design.

The rate of saturated water flow (Q = Volume/Time) through soil depends on the hydraulic gradient (i,) the cross sectional area (A,) and the saturated hydraulic conductivity (K) of the soil (Q = K i A). The hydraulic gradient is equal to the change of elevation between the upstream and downstream water surface, divided by the length of flow between these surfaces. Figure 1(a) shows these measurements for flows down hill through a layer of soil, and Figure 1(b) shows these measurements for the flow through a cylinder of soil used to measure the hydraulic conductivity.

FIGURE No. 1 a.

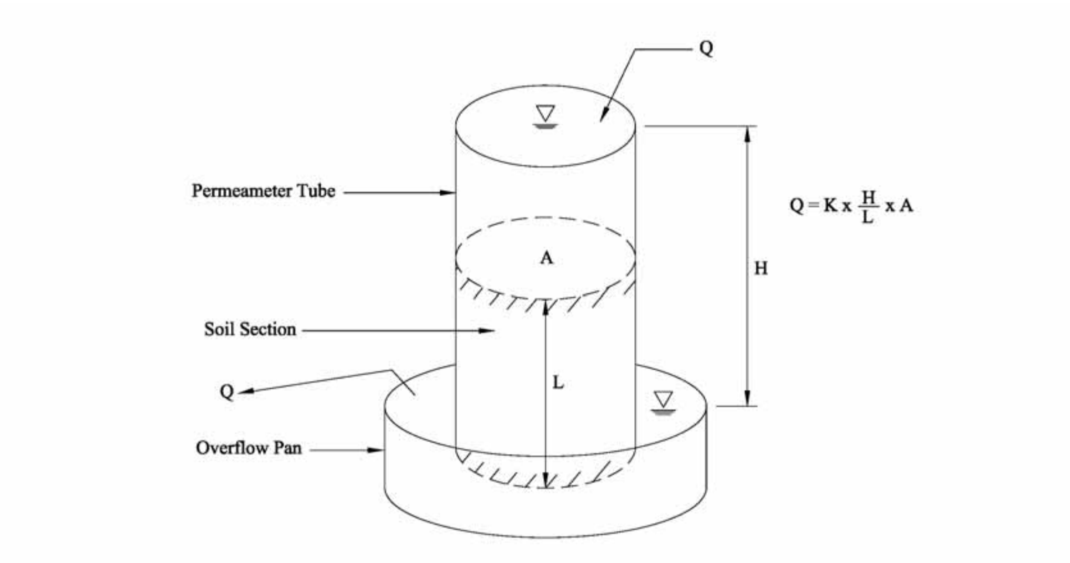
FLOW DOWN HILL THROUGH A LAYER OF SOIL



In Figure No. 1a and 1b, the hydraulic gradient, i = H/L

FIGURE No 1 b.

FLOW THROUGH A CYLINDER OF SOIL



D. Factors Determining Hydraulic Conductivity

The properties of the soil that determine the hydraulic conductivity are primarily the grain size distribution, how tightly the grains are packed, and the arrangement (layering) of the grains. Table No. 1 gives typical hydraulic conductivity values for various types of soil.

Table No. 1.

Typical Hydraulic Conductivity Values for Various Types of Soils

Soil Type	Hydraulic Conductivity K, ft/day	Cross-Section Area, in Sq. Ft. required for same flow as 2" Pipe
$1\frac{1}{2}$ " Stone	100,000.00	1.20
3/8" to No. 4 Gravel	7,000.00	2.00
Coarse Sand	1,000.00	180.00
Fine Sand	10.00	18,000.00
Silt	0.01	18,000,000.00

A small amount of silt in sand can reduce the hydraulic conductivity dramatically. Table No. 2 gives the hydraulic conductivity of a clean sand vs. different percentages of material passing the #100 sieve.

Table No. 2

Hydraulic Conductivity of Sand vs. % Passing No. 100 Sieve

% Passing No. 100 Sieve	Hydraulic Conductivity, ft/day
0.0	300.0
4.0	15.0
7.0	1.5

The tightness of packing tends to have a greater effect on fine-grained soils then on coarse-grained soils and gravels as shown in Table No. 3.

Table No. 3

Effect of Packing on Hydraulic Conductivity

Soil	Hydraulic Conductivity, ft/day	
	Loose	Dense
Silty Sand	0.30	0.03
Sandy Gravel	30.00	5.00

Many deposits of soils, particularly in New England, contain lenses and pockets of different soil, and the soil may contain fissures or cracks. The effect of these discontinuities can be very large and is destroyed by remolding (recompacting) as is shown in Table #4.

Table No, 4

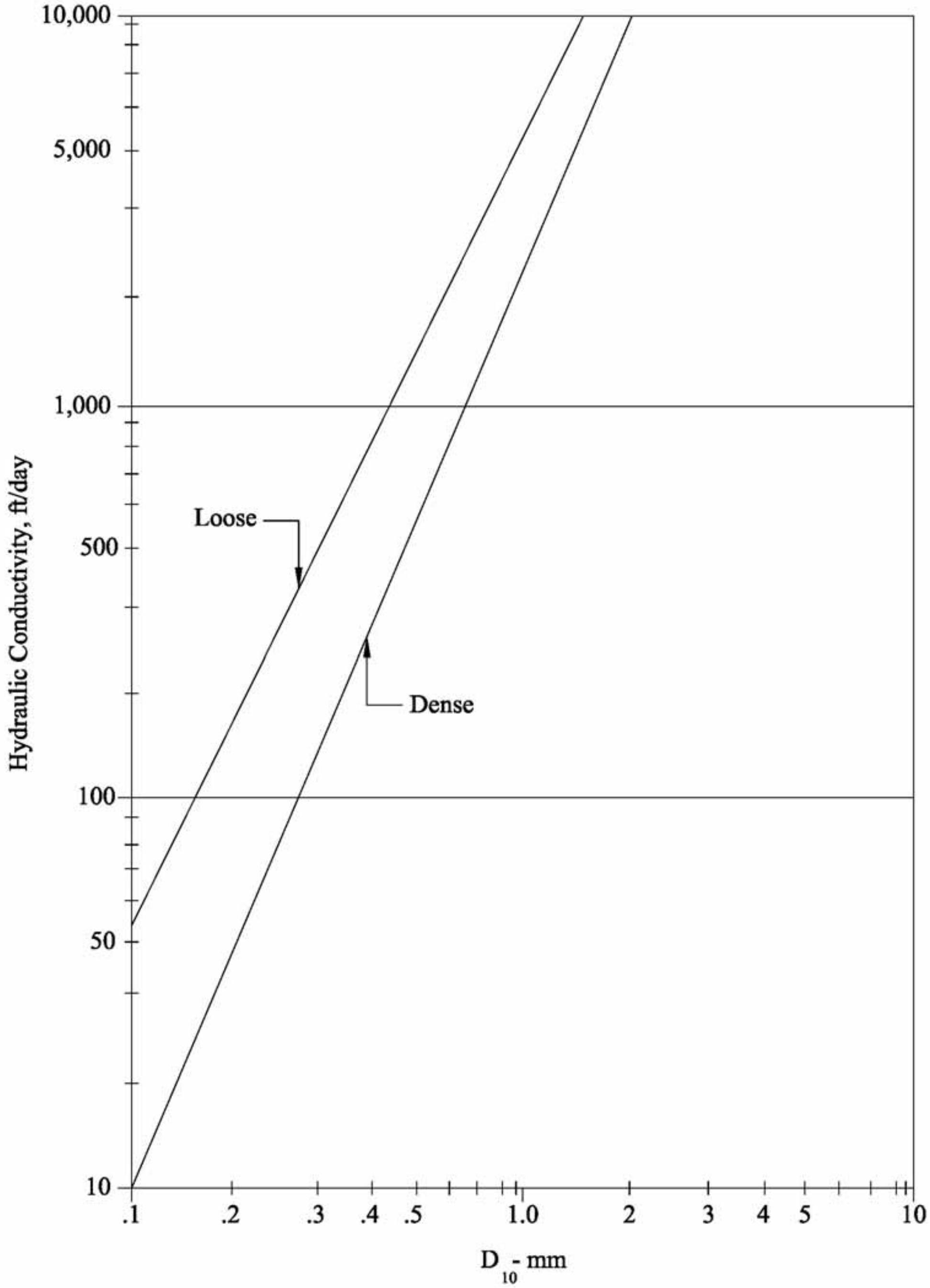
Effect of Remolding on Hydraulic Conductivity

Soil	Hydraulic Conductivity, ft/day	
	Undisturbed	Remolded
Glacial Till	1.50	0.0015
Silty Sand	30.00	0.03

A relationship between hydraulic conductivity and the d_{10} grain size is shown in Figure No.2. [The d_{10} size of a soil is defined under E 1 - Grain Size Distribution]

FIGURE No. 2

K vs. D_{10}- SANDS & GRAVELS



E. Methods of Measuring Saturated Hydraulic Conductivity

There are four general methods of measuring the saturated hydraulic conductivity of a soil deposit: (1) inference from grain size distribution, (2) tests on disturbed samples, (3) tests on undisturbed samples, and (4) In Place (field) tests. Methods (3) and (4) are more appropriate for use in selecting values of K for design of a SWAS.

1. Grain Size Distribution

Estimating the hydraulic conductivity from the grain size distribution ignores the effect of compaction or discontinuities and may give very misleading results if the percentage of fine grains is not measured carefully. The method is most accurate with soils that don't contain any silt or clay. Figure (2) gives the hydraulic conductivity of sands and gravel in ft/day versus the d_{10} size. The d_{10} size of a soil is that sieve size that only 10% by weight of the grains are smaller than. A chart such as shown provides accuracies of K within a factor of 2 or 3.

2. Tests on Disturbed Samples

A disturbed sample of soil is re-compacted into a tube to approximately the field density, and a falling or constant head permeability test is run as shown in Figures 3a and 3b. (Tests on re-compacted soil samples also ignore the effect of soil structure and generally will give a lower hydraulic conductivity than exists in the field.) In structure-less soils, such as clean sand or gravel, this test will indicate a hydraulic conductivity that is within a factor of 2 or 3 of the field hydraulic conductivity. Note that warm water that has been cooled to 20 degrees C should be used to reduce the accumulation of air bubbles in the soil.

3. Tests on Undisturbed Samples

Undisturbed samples must be used for hydraulic conductivity testing of all but sand or gravel due to the structure that may be present in the finer grain soils. If the water appears muddy when mixed with the soil, undisturbed samples should be used. There are a variety of undisturbed testing methods as described below.

3.a Tube Tests

Un-cemented soil with little gravel can be sampled with a sharp edge tube 1-4" in diameter. If excessive force is needed to push the sampler into the soil, block samples should be taken. The tube that the soil is sampled with can be used as the permeameter for a falling or constant head test as shown in Figure 3a and b.

Tube tests of undisturbed samples can be useful in cohesive sands, sandy loams, and loose till where the sample can be easily taken and will not crack or fall from the tube.

The material or facilities required are as follows:

1) A sink, or place where water can overflow.

2) A shallow pan, such as a baking dish, kitty litter pan, etc.

3) 1 1/2 - 1 1/4" thin wall tubes 6" - 12" long; plated sink drain tubes are readily available and inexpensive.

4) A bag of Ottawa testing sand or other uniform clean sand, and pieces of filter fabric that have a hydraulic conductivity considerably greater than the soil sample so that the permeability of the fabric will not affect the test results.

5) Scale or ruler.

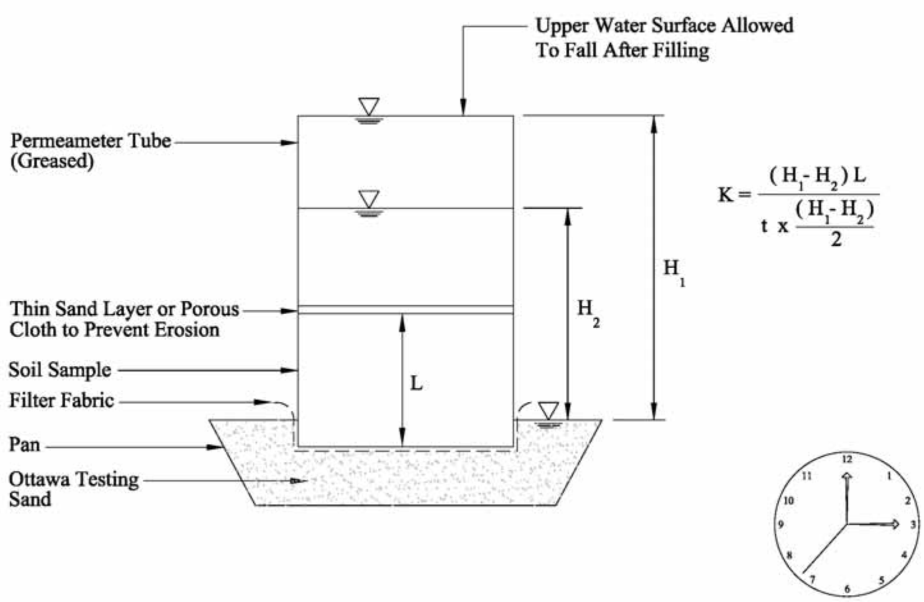
6) Grease for tubes.

7) A supply of hot water that has been cooled to 20 degrees C. and utensils for pouring it. (Water that has been heated will have been essentially de-aerated. Care should be taken not to agitate the water after cooling to a degree that would cause its reaeration. Water containing air bubbles will interfere with the test if the air bubbles become trapped in the soil pores.)

8) A vacuum pump or aspirator if air blockage of the soil pores is suspected.

FIGURE No. 3 a

FALLING HEAD TEST



Procedure For Falling Head Test

Select the suitable soil strata for testing and collect a number tube samples from each stratum (soil horizon), the more the better. The number of samples will depend to some extent on the size of the SWAS and the nature of the soil deposits. It is recommended that the number and location of samples to be taken be discussed with Department staff before proceeding with the field sampling. Tubes should be greased internally and pushed into the soil to remove a 3 - 6" plug of soil. Tubes can then be placed upright on a bed of Ottawa sand in the shallow pan for transport. Tubes can be taken vertically or horizontally to reflect the appropriate K value for the flow regime. Back at the office, measure the length (L) of each sample and record it. Replace the tubes upright and put 1/2" of Ottawa sand on the surface of the sample. Saturate the sample and base sand with water until the shallow pan overflows and a head exists in the tube. Continue until you are sure the sample is saturated and a steady state of flow exists.

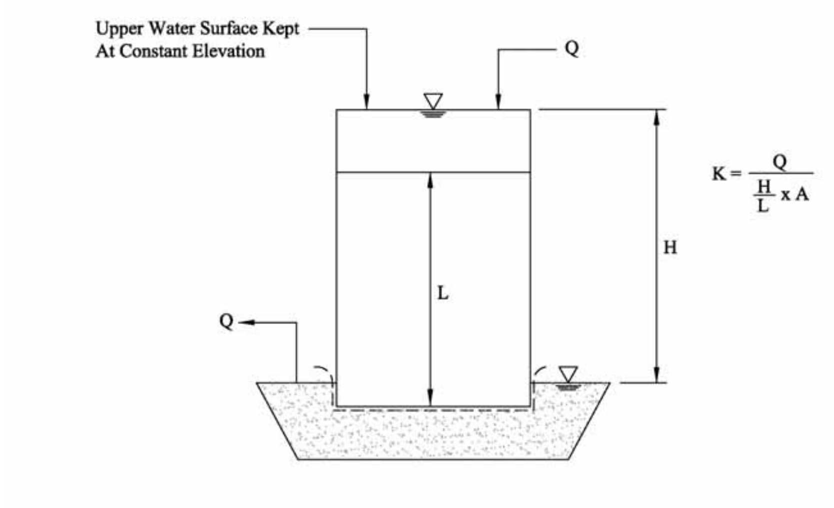
It is often difficult to determine if the sample is thoroughly saturated so that no air is blocking any of the pores in the sample. Air blockage will result in the test K value being less than the actual K value for the soil in the tube. A more valid procedure is to apply a vacuum to the tube, as given in the following procedure. Fill the shallow pan with water until the Ottawa sand is saturated and the pan overflows. Then place the samples under a vacuum of 20 inches of Mercury for a minimum of 15 minutes to evacuate all of the air in the sample pores. Continue applying the vacuum until water has been drawn up into the headspace above the sample in the tube to a height sufficient to run the test. (In cases where the sample tube is completely filled with the soil sample, it will be necessary to add an extension to the sampling tube). Then release the vacuum and measure the drop (H_1 - H_2) and the time of drop. Use the formula in Figure No. 3a to calculate K. This calculation uses an average approach velocity and an average head loss but is more than accurate enough, and more exact calculations are not necessary.

Procedure for Constant Head Test

Constant head tests (figure 3b) may be used for samples of soils of high hydraulic conductivity where the drop in head in a falling head test is too rapid to record accurately. They are run much like the falling head test but more apparatus is needed. A valving system must supply water at low constant rates (Q in) to maintain H. Alternately the overflow from the tube must be collected and not allowed to enter the pan overflow. Pan overflow (Q out) must be collected and accurately measured in a graduated cylinder.

FIGURE No. 3 b

CONSTANT HEAD TEST

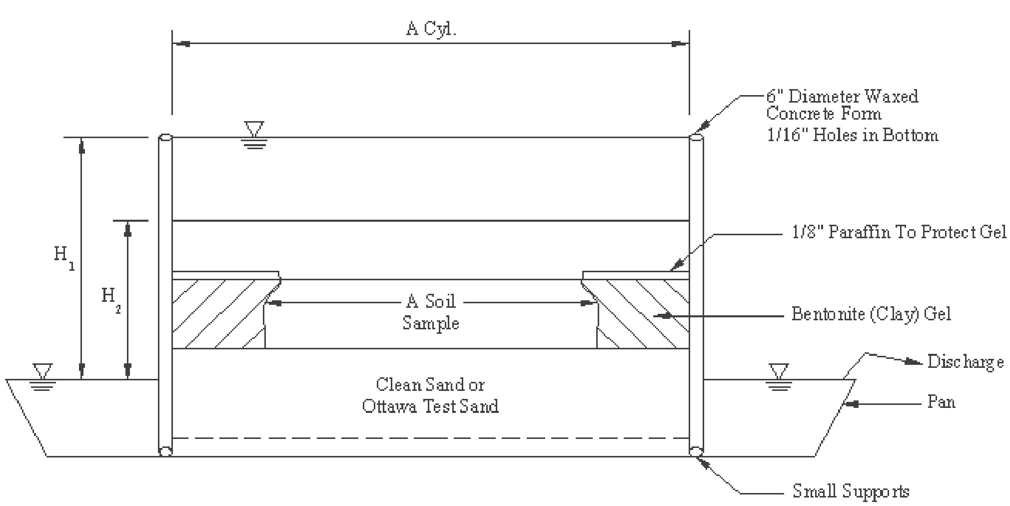


3.b Undisturbed Block Samples

It is often possible to use a shovel or knife to cut out undisturbed blocks of cemented sand and gravel or soils containing silt or clay. The blocks are trimmed with a knife to measurable dimensions and placed in the permeameter and tested as shown in Figure No. 4. This test is very useful in compact tills and can measure vertical or horizontal K.

FIGURE No. 4

UNDISTURBED BLOCK SAMPLES TAKEN FROM TEST PIT.
(GLACIAL TILL OR CEMENTED SAND)



Materials and Procedures for Undisturbed Block Sample Testing.

1. Block of soil taken from a soil horizon is trimmed to measurable dimensions and bedded 1/8" on clean well-graded concrete sand.
2. Concrete or Ottawa sand must be coarse enough so that K is 20-50 times the anticipated K of the soil, and fine enough so that gel won't penetrate it.
3. Molten paraffin is poured over gel to protect if from erosion by water above.
4. Test is run in a similar manner to the tube test in figures 3a and 3b.

$$K = \frac{Q}{H/L \times A\ \text{soil}} = \frac{\text{Surf. Velocity} \times A\ \text{cylinder} \times L\ \text{soil}}{H\ \text{avg.} \times \text{Area soil}}, \quad \text{where:}$$

$$\text{Surface Velocity} = \frac{\text{drop of water surface}}{\text{Time}}$$

$$H\ \text{avg.} = \text{Average head during test period} = \underline{H_1 + H_2}\ \text{time}$$

4. In-Place Field Tests

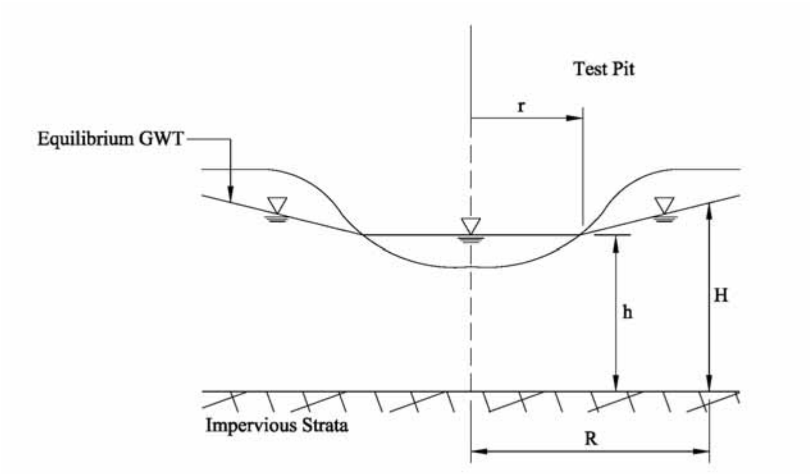
The primary advantage of in-place tests is that the mass of soil being tested is undisturbed and generally large. In order for this test to be meaningful, however, the dimensions of the strata in which the test is run must be accurately measured, utilizing test pits and in some cases borings.

(a) Pit Bailing Tests

The test makes use of the test pit that is dug to determine the depth to the water table and the position of the various soil strata. The water table must be within 8-10 feet of the ground surface to carry out the test, and the hydraulic conductivity of only the soil below the water table is measured. The test procedure is based on gravity well theory and is illustrated in figure No. 5 on the following page. The radius of drawdown can be assumed equal to four times the effective radius of the test pit without significant error. The depth to impermeable strata should be estimated, although in sites where the depth to an impermeable strata is not well defined it can be assumed equal to the depth of the pit, and the hydraulic conductivity will be slightly over estimated. There are two ways to carry out this test. The first entails measuring the rate of water level rise in the pit when it is first dug and returning a day or two later to establish the equilibrium water level. The second entails digging the test pit, waiting several days for the water level to stabilize, lowering the water level by bailing, and measuring the rate at which it rises. The water level should be lowered at least one foot for reliable measurements. [NOTE: When this test is run in sandy soils and the test pit barely penetrates the water table, the results will be inaccurate.]

Figure No. 5

PIT BAILING TESTS



$$Q = \frac{\pi K (H^2 - h^2)}{\ln R/r};$$

Q = Rate of water level rise in pit x water surface area in pit.

H and h can be taken from bottom of pit.

ln R/r can be assumed = 1.4

Procedure

1) Excavate test pits providing a suitable step or sloped end to facilitate measurement.
2) Establish a fixed point to measure depth to equilibrium water table, time related water table and pit bottom.
3) Measure depth to pit bottom and equilibrium water table (highest seep or stable level).
4) Measure width of pit.
5) At suitable intervals measure and log:
   a) Time
   b) Depth of water surface
   c) Length and width of water surface.

For example, assume:

Pit depth = 56.5 inches
Equilibrium water table is 17.25 inches below ground surface.

Time	Depth to water level	Area of water surface
4:10	55.5"	1.5 x 3 = 4.5 ft^2
4:20	54.5"	1.5 x 4 = 6 ft^2
10 min.	1 inch	Avg. area = 5.25 ft^2

$$Q = 5.25\ \text{ft}^2 \times \frac{1}{10\ \text{min}} \times \frac{1}{12}\ \text{ft} = 0.044\ \underline{\text{ft}}^3/\text{min.}$$

$h = 56.5 - 54.5" = 2$ inches $= 0.17$ ft
$H = 56.5 - 17.25" = 39.25$ inches $= 3.25$ ft . $(H^2\text{-}h^2) = 10.5\ \text{ft}^2$

$$K = \frac{1.4\,Q}{\pi\,(H^2\text{-}h^2)} = \frac{1.4 \times 0.044\ \text{ft}^3/\text{min}}{\pi \times 10.5\text{ft}^2} = 2.7\ \frac{\text{ft.}}{\text{day}}$$

(b) Auger Hole Bailing

The auger hole bailing test was developed by agricultural engineers to determine the field hydraulic conductivity of soil where the water is near the ground surface. The test is an accurate test and can be done by one person with a 4-6" auger, and a bailing apparatus that will fit down the hole. The hole should be dug to a depth below the water table of at least five hole diameters. The water level in the hole is allowed to rise to the equilibrium elevation. The water level is then lowered several hole diameters by bailing, and the rate of water level rise is measured. This test, because of the hole shape, measures primarily the horizontal hydraulic conductivity, which is generally what is desired for hydraulic capacity analysis, as the subsurface flow away from the SWAS is predominately in a horizontal direction.. The method of computing the hydraulic conductivity is illustrated in Figures 6a and 6b.

AUGER HOLE BAILING

Figure No. 6 a

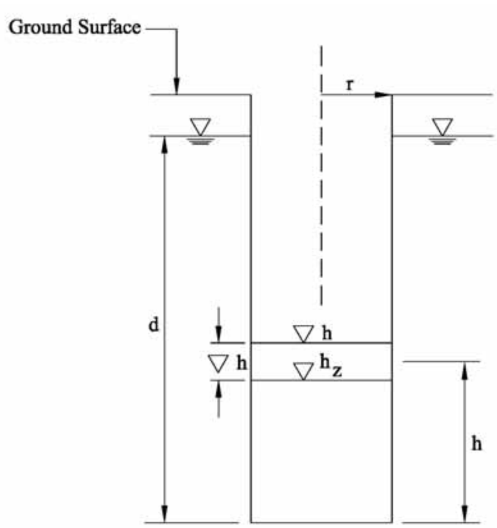
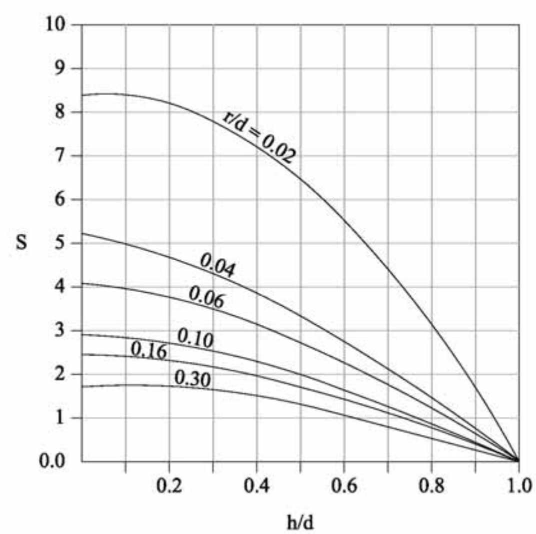


Figure 6 b.



$K = [888\ r/(S \times d)] \times (\Delta H/\Delta\ t)$

Where;

K – Hydraulic Conductivity, ft/day
r - ft
d - ft
Δh - ft
Δt - minutes
S - a factor read from Figure 6b

(c) Well Pumping Tests

These tests are more complex, normally involving pumping wells at a constant pumping rate until steady state, or near steady state conditions are achieved and deriving K from the results. These tests should be conducted under direction of someone who has the requisite hydrogeologic training and experience. The Department should be consulted before proceeding with such tests.

(d) Slug Tests

Because of their simplicity relative to well pumping tests, a number of slug tests can usually be performed with less effort and expense than a single well pumping test. The well pumping test, if properly performed and results properly analyzed, may provide a higher degree of accuracy in determining K while testing a larger aquifer volume as compared to a slug test. However, simple techniques yielding reasonable results are often preferable to more complicated techniques given the heterogeneous nature of soil.

A slug test consists of measuring the recovery of head vs. time in a well after a near-instantaneous change in head (water level) at that well. The sudden change in head can be accomplished by rapidly introducing a solid object (the slug) into the well or rapidly removing it from the well, causing an abrupt increase (decrease) in the water level in the well. Following this sudden change, the water level in the well returns to its original level in response to the gradient imposed by the sudden change in head. Timed measurements of the water level (head) changes, which are termed the response data, can be used to estimate the hydraulic conductivity of the soil formation through comparisons with theoretical models of test responses (Cedergren - 1989). It is assumed that the soil medium in which the slug test is conducted is homogeneous and isotropic.

Slug tests are normally used to estimate the value of K where the strata that will convey the treated wastewater away from the SWAS extends both beneath and above the seasonal high ground water table. The value of K determined for the soil below the water table will thus be applicable to determining the flow in the soil above the seasonal high water table that will occur due to the rise in the water table (mounding) resulting from the discharge of the wastewater to the subsurface.

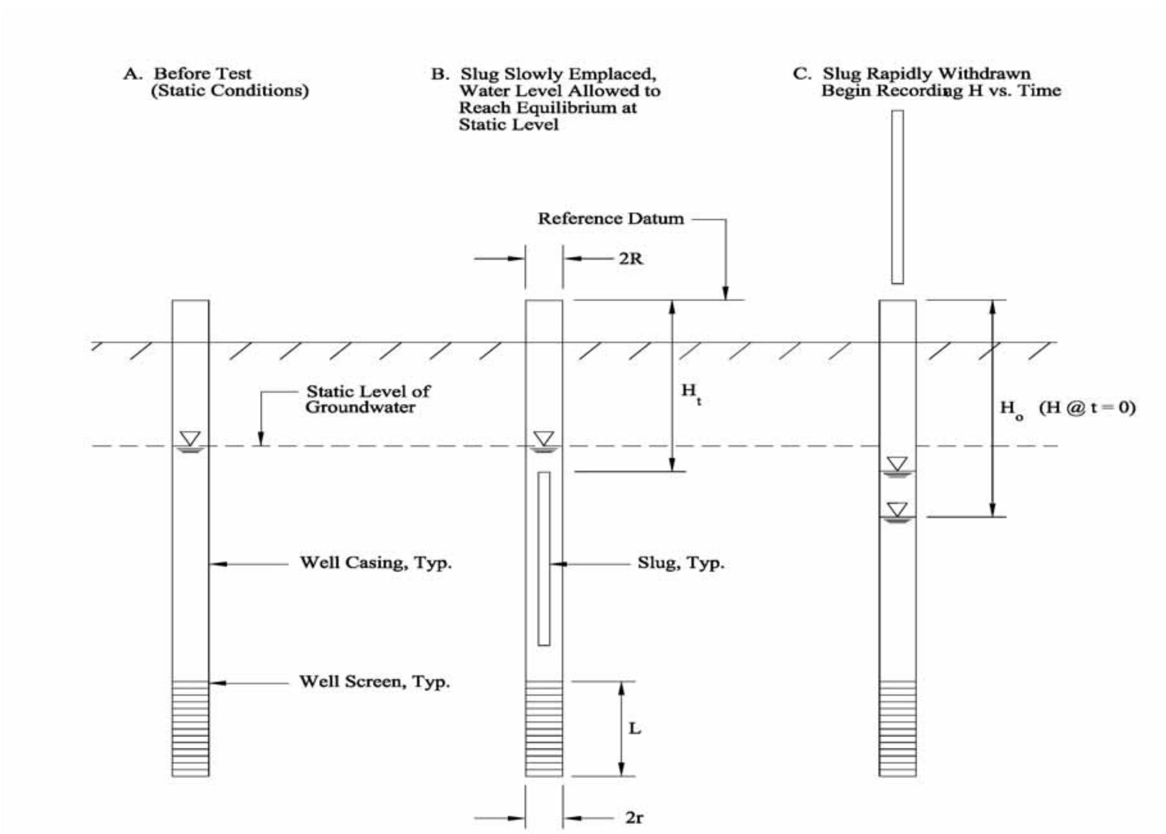
Slug tests will also provide a means of estimating the hydraulic conductivity of the soil when pit bailing, auger hole bailing, or collection of tube or block samples is impracticable. This is usually the case in relatively deep aquifers in glacial stratified drift or outwash deposits. These tests are only useful if the monitoring well extends below the water table.

Slug-in tests consist of rapidly introducing the slugs into the groundwater monitoring well. Slug-out tests consist of rapidly removing the slugs from the groundwater monitoring well. It is recommended that a minimum of two slug-in tests and two slug-out tests be performed for each groundwater monitoring well location with three of each test being preferable. If possible, response data should be reviewed in the field to determine if smooth slug introduction or removal was achieved (no oscillation in water level). Figure No. 7 depicts the conditions for a slug-out test.

While it is possible to conduct depth to water level measurements in low hydraulic conductivity environments (glacial till or soils with a high fines content) with a typical well probe, high hydraulic conductivity environments (stratified drift and outwash deposits) typically require the use of a fixed transducer and water level data recorder. Most water level data recorders record data in linear or logarithmic intervals with logarithmic data recording being preferable in very high permeability environments (coarse grained stratified drift), due to the small time increments involved.

FIGURE No. 7

SLUG-OUT TEST CONDITIONS



The groundwater monitoring wells should be properly developed prior to conducting slug tests by surging the screen interval with a surge rod to ensure that any skin effects are removed. (The “skin” is the altered soil conditions in the immediate vicinity of the well screen, such as can be caused by the mobilization of near-well fine materials during construction of the well. For example, auger rotation and retrieval can smear fine materials on the wall of the auger hole.) Surge rods can be constructed of threaded steel stock with an assortment of rubber, nylon, brass and stainless steel washers and the groundwater monitoring wells are developed by repeatedly raising and lowering the surge rod attached to a slug.

While numerous slug test equations and references exist, groundwater monitoring well geometry relative to formation changes (i.e. confined formation, fully penetrating screen, partially penetrating screen, unconfined formation) and nature of formation dictate the correct equation to utilize in estimation of hydraulic conductivity. The U.S. Department of the Navy, Naval Facility Engineering Command (1982) provides tables and figures for hydraulic conductivity determination based on borehole condition, formation shape factor (geometry of the groundwater monitoring well relative to the formation position), and applicability. Some additional reference publications include Cedergren (1989), Butler (1997) and ASTM (1999). A Site Assessment CD in multi-media format, primarily developed to investigate fuel releases at underground storage tank sites, is available from the Department at modest cost that contains spreadsheets that can be used to analyze slug test data obtained under various soil and well geometry conditions to determine hydraulic conductivity. Other computer programs useful for processing slug test data are available from commercial sources.

For simplicity in analysis it is preferable to construct the groundwater monitoring well such that the induced recharge (slug-in) or drawdown (slug-out) results in the water surface always being within the solid case interval. Butler (1997) recommends that the primary direction of flow during a series of slug tests should be from the formation into the well (Slug-out test). Flow from the well into the aquifer formation during a slug-in test will often lead to decreased estimates of hydraulic conductivity as a result of mobilized fine material being lodged deeper in the formation. Thus, where both slug-in and slug-out tests are performed on the same well, the slug-out tests should be performed first. It is important to note that the method is dependent on the response data being unaffected by the storage properties of the well-formation configuration. Therefore, it is important that the normalized semi-log response data (H/Ho) vs. time is linear.

Aquifer slug tests are valuable tools in determining an estimate of hydraulic conductivity. Slug tests, when properly performed and data properly analyzed, allows for more accurate determination of in-situ formation hydraulic conductivity than tube tests, since the volume of soil for which K is determined is much greater in slug tests. However, the test results are only representative of the average hydraulic conductivity of the portion of the aquifer adjacent to the open interval of the well. Therefore, slug tests should be conducted in a number of ground water observation wells to obtain representative site-specific data. The results obtained from slug tests should be checked against several other methods used to estimate hydraulic conductivity.

Similar to well pumping tests, slug tests should be conducted under the direction of someone who has the requisite hydrogeologic training and experience. It is recommended that the Department be consulted before proceeding with such tests if the results are to be submitted to the Department as part of a permit application.

5. Determination of Hydraulic Conductivity from Site Performance.

Observations of ground water elevations, slope and boundary conditions can be utilized to determine hydraulic conductivity. In this type of testing the response of a site to a known recharge can be utilized to estimate hydraulic conductivity. Two examples of this determination are included in Subsection F - Basic Site Hydraulic Capacity Analysis, Case 2. A and Case 2. B.

6. Verifying Hydraulic Conductivity

Various soils texts provide tables giving ranges of hydraulic conductivity for various typical soils groups. The soils map prepared by the United States Department of Agriculture Soils Survey list engineering properties, including hydraulic conductivity ranges for the A, B and C horizons of mapped soils. This information can and should be utilized as a check to see if the values you have determined are reasonable. The final check on a hydraulic conductivity value comes in its application in a hydraulic capacity analysis. If a reasonable job has been done identifying boundary conditions and hydraulic gradients, then the K value obtained should project the actual predevelopment site conditions. For example, if you have tested during high ground water and found it 6' below grade then a predevelopment hydraulic capacity analysis should come close to that condition. If your analysis shows ground water should be at 6" below grade than an error has been made in the hydraulic conductivity value or assumed boundary conditions.

7. Number of Tests Required

The number of tests that should be made in order to estimate K values will depend to a great extent upon the test methods. Because of the heterogeneity of soils in general, a significant number of tests should be made when estimating K from results of tube tests, block tests or grain size analyses, since the soil samples used in these tests are representative of only a very small portion of the soils on a site. On the other hand, a fewer number may suffice when field tests, such as auger hole tests, slug tests, etc. are utilized or when observing the behavior of a site before, during and after a significant rainfall event, as such tests are usually representative of a much greater volume of soil. In general, the rule should be the more the better. If the results of initial testing are inconclusive, additional sampling and testing should be performed.

8. Estimation of Mean Hydraulic Conductivity

It has been shown that, for a random distribution of K values, the “true” mean lies between the arithmetic and harmonic means of the individual K-values. The geometric mean also lies between the arithmetic and harmonic means. The geometric mean is used when the distribution of values of the sample population is skewed from the normal distribution, and prevents one or two high values from giving overestimates of the mean value.

The geometric mean (G) is determined as follows:

G = Nth root $(X_1X_2X_3..X_N)$ which is the Nth root of a product of N values of X.

For example, the geometric mean of the numbers 3.20 + 5.10 + 8.30 = $(3.20 \times 5.10 \times 8.30)^{1/3} = (135.4)^{1/3} = 5.135$

The geometric mean K of a number of individual K values can also be obtained using the following procedure:

Log X = [∑ (log x)]/N, where X is the geometric mean, x is each individual value, and N is the number of values.

Thus, the logarithm of each value of K is determined, the resulting logarithms of all of the values are summed, and this sum is divided by the number of values to yield the logarithm of the geometric mean K. The antilog of the logarithm of the geometric mean is the geometric mean of all of the values. Either Log_{10} or Ln (natural log) can be used to calculate the geometric mean.

A more detailed explanation of determining the geometric mean is given in Appendix C - Selecting Hydraulic Conductivity Values for Design.

9. Utilization of Hydraulic Conductivity Values

After testing with several methods the engineer is confronted with a range of hydraulic conductivity values. After checking values and due consideration, "outliers", extremely high or low values, can normally be discarded. This leaves the designer with a reasonable range of values for use in three areas of design. These areas are:

1) Determination of system size by the long-term acceptance rate method.

2) Determination of the capacity of the site to handle the discharge volume (hydraulic capacity analysis).

3) Calculation of the capacity to renovate wastewater (definition of the zone of influence).

For example, when performing a hydraulic capacity analysis or selecting a value for long term acceptance rate, or LTAR, (see Section X), a conservative value in the lower 50 percentile of the range of values should be used and certainly should not exceed the geometric mean. On the other hand, when computing the zone of influence (travel time from the SWAS to the closest points of concern), a conservative value from the high end of the range of K values should be used. It is recommended that the Department be consulted with respect to the values of K to be used for hydraulic capacity analysis, LTAR, and travel time after the results of testing for K by several methods have been determined. (See Section X and the Design Standards for additional information on selection of K values for design.)

All of the hydraulic conductivity tests described measure the saturated hydraulic conductivity of the soil that allows, in conjunction with hydraulic capacity analysis, a calculation of the maximum rate that water can be carried by the soil strata. As with any testing program, the more time that is spent the better the results, but the investigator should never have such complete faith in the results of a few tests that he or she neglects to observe other bits of information that are available. This is particularly true of hydraulic capacity testing. Observation of the reaction of a site to rain may provide a more complete picture of the hydraulic conductivity of a soil deposit than a hundred hydraulic conductivity tests.

F. Basic Site Hydraulic Capacity Analysis

1. Introduction

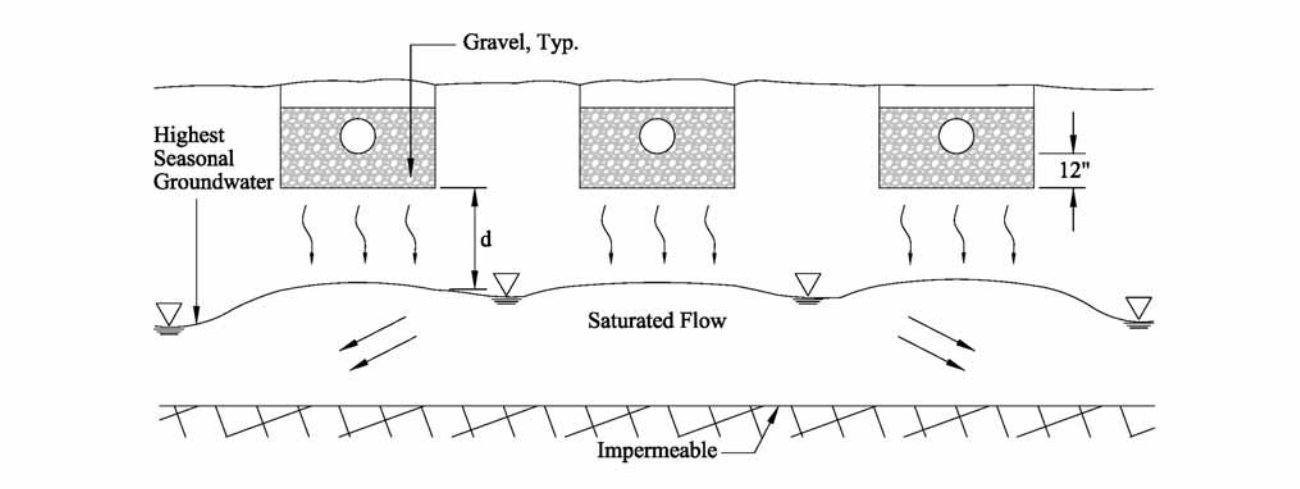
The hydraulic capacity of a site depends on the hydraulic conductivity and dimensions of the soil, the slope and position of the ground water table beneath and near the site, and the position of any impermeable boundaries beneath or around the site. When designing a SWAS, it should be determined that the soil deposit has adequate hydraulic capacity to carry the septic tank effluent below ground surface for a sufficient period of time, and for a sufficient distance, to bring the pretreated wastewater into compliance with the required ground water quality standards of the Department before it reaches a point of concern. This hydraulic capacity should be available under all conditions, so that any hydraulic capacity analysis of a potential SWAS site must consider all water that flows into the site, including ground water, surface water, precipitation, and the pretreated effluent. Water must be able to flow away underground faster than it enters the area of the SWAS. While the examples contained in this section do not address reserve hydraulic capacity, adequate reserve capacity shall be provided in the system design. This may be done either by applying an additional factor of safety to the design flow in the hydraulic capacity analysis or by provision of additional area for installation of a SWAS elsewhere on-site.

The hydraulic capacity analyses that follow are based on two important assumptions. First, it is assumed that the SWAS trenches, galleries or chambers will be ponded to a depth sufficient to push the effluent through a layer of reduced hydraulic conductivity at the soil interface (the biomat) at some known rate.

Second, it is assumed that there will be saturated flow at some depth below the SWAS during the wet season when the ground water table is highest. These two assumptions are illustrated in Figure No. 8. [Note: While the examples discussed herein are generally based on a SWAS for a single-family dwelling, the principles applied are also applicable to large-scale systems.]

FIGURE No. 8

SECTION THROUGH SUBSURFACE WASTEWATER ABSORPTION SYSTEM
(Section shows typical trench type of SWAS)



Assumptions for analysis

1. Effluent ponded in trenches, galleries or chambers

2. Saturated flow at depth d below trenches, galleries or chambers.

Neither of these are desirable operating conditions; however, based on these assumptions, the maximum capacity of the SWAS under the worst condition (high GWT) can be calculated. If the SWAS is loaded at some fraction of this maximum capacity, the soil immediately below the system will not be saturated even during the wet season, and the effluent will not pond to one foot in the trenches, galleries or chambers.

The choice of that fraction of the maximum capacity that can be used for normal operation will depend on the consequences of failure, which generally occur in the form of toilet backup or surface flooding. Failing systems can contaminate surface or ground water, create nuisance conditions, present a source of infectious material, and can have a severe financial impact.

2. Analysis Based on Ground Water Levels During Wet Season

A valuable and simple hydraulic capacity analysis of a site involves observation of the ground water levels in standpipes or pits during the wet season. During this period, ground and surface water inflow is a maximum and there is minimum evapotranspiration. If the ground water table remains well below the ground surface during the wet season, the site can probably absorb the additional water from a SWAS.

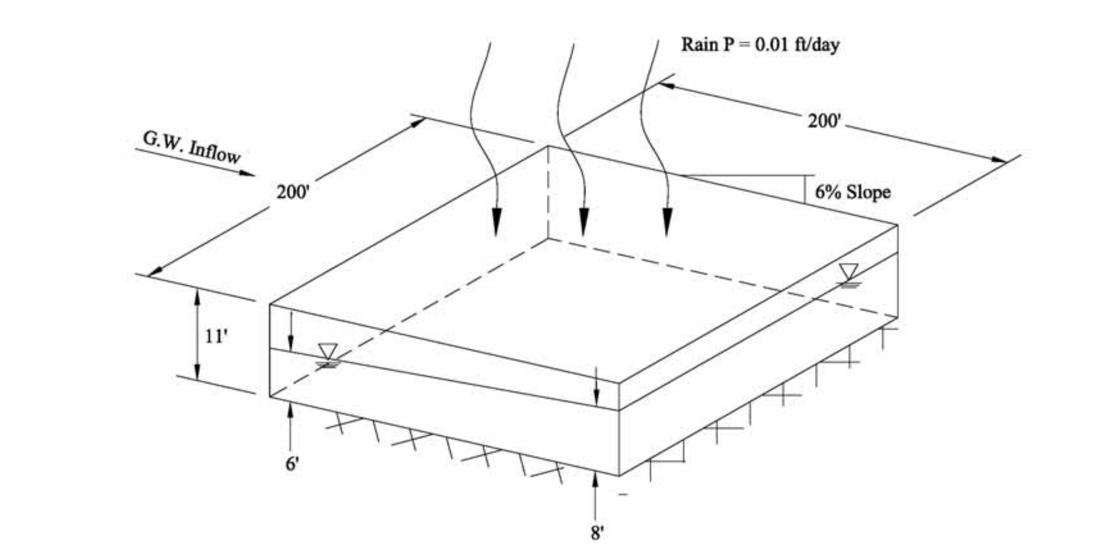
Calculations of the hydraulic capacity of a site can be made based on ground water levels during wet weather conditions, if test pits are dug to identify various soil strata and measurements are made of the rate of ground water table drop between rainfalls**.**

Following are examples in which observations of the ground water levels and depths to impermeable strata allow calculation of the soil hydraulic conductivity, site hydraulic capacity, and the effectiveness of curtain drains.

Case A - Figure No. 9 shows a 3-D view of a one-acre hillside site with 11 feet of soil overlying an impermeable stratum on a 6% slope.

FIGURE No. 9

3-D VIEW OF HILLSIDE SITE



Test pits show that the GWT is 6 feet above the impervious strata at the upper end of the site, and 8 feet above the impervious strata at the lower end. The increase in the depth of flowing water due to rainfall (8'-6') allows calculation of the hydraulic conductivity of the soil, which in turn allows calculation of the increased depth of the flowing water that would occur due to the discharge from a house. Knowledge of soil hydraulic conductivity, slope and depth of flowing water allow the calculations of the effect of a curtain drain at the upper boundary of the site.

a) Assume the inflow from rain averages 0.01* ft/day. The total volume of inflow = 200 ft. x 200 ft. x 0.01* ft/day = 400 ft^3/day

   The hydraulic gradient = 0.06 ft/ft. The increase in flow area due to the rainfall = 2 ft (depth) x 200 ft.(width) = 400 sq. ft.

   Thus, Hydraulic Conductivity, $K = \dfrac{Q}{I \times A} = \dfrac{400\ ft^3/day}{0.06\ ft/ft \times 400\ sq.\ ft.} = 17\ ft/day$

* In actual design, the inflow from rainfall should reflect the amount of rainfall that actually infiltrates and reaches the ground water.

b) Assume a discharge from 3 bedroom house = 40 ft^3/day and that this flow is discharged to a SWAS situated across the full width of the lot. The increase in depth of the flowing water is calculated as follows:

$$A = \frac{Q}{K \times i} = \frac{40 \text{ ft}^3/\text{day}}{17 \text{ ft/day} \times 0.06 \text{ ft/ft}} = 39 \text{ ft}^2$$

$$\text{Depth Increase} = \frac{39 \text{ ft}^2}{200 \text{ ft}} = 0.2 \text{ ft} \quad \text{(Area/width)}$$

c) Ground water inflow from uphill = K x i x A = 17 ft/day x 0.06 ft/ft x 6 ft x 200 ft = 1224 ft^3/day. Intercepting all incoming ground water with curtain drains and bypassing it would increase site capacity by 1224 ft^3/day.

In considering this example you must think about the figure of 0.01 ft of rain/day. This figure represents annual average rainfall divided by 365. In this case you have tested the site during the spring high ground water period when evapotranspiration is at a minimum but frost is gone. In this case the figure is probably reasonable. At other times of the year when evapotranspiration occurs, the analysis can be made more conservative by using a lower or more refined rainfall infiltration rate (A more detailed discussion of the infiltration of precipitation into the subsurface is given in G.2 of Section X)

Case B - Figure No. 10 shows the cross section of a level site. Observation of the water level in standpipes indicates that the highest seasonal GWT is 4 ft from the ground surface and that between rains in the spring, the ground water table drops 2 inches per day. Although there may be no visual indication of how the water is leaving the site, downward or laterally, since evapotranspiration is near zero, the water must be flowing away underground.

The rate water leaves the site can be calculated by multiplying the rate of water table drop by the area involved, times the drainable porosity. Drainable porosity is that volume of water that drains by gravity from the soil divided by the total volume of the soil.

TABLE No. 5

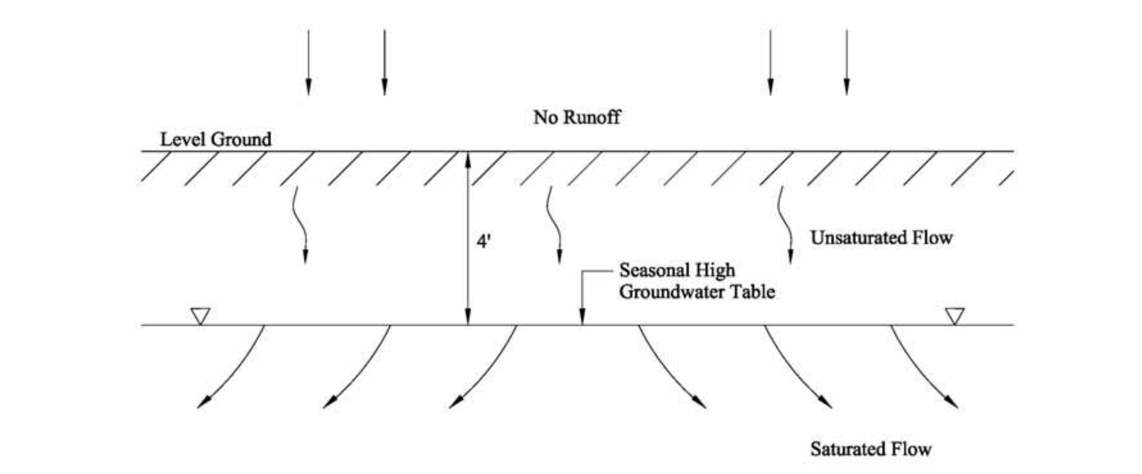
APPROXIMATE VALUES FOR DRAINABLE POROSITIES

coarse sand	- 0.25
fine sand	- 0.15
silt	- 0.10
clay	- 0.05

FIGURE No. 10

SECTION THROUGH LEVEL SITE

RAIN = 0.01* ft/day



* As noted previously, this value should be adjusted on a case-by-case basis as necessary to reflect actual infiltration.

Procedure

Between rains in spring, GWT drops at rate of 2 inches/day.
If drainable porosity = 0.1,
A drop of 2 in./day = 0.2 in. of water/day $= \dfrac{0.2 \text{ inches}}{12 \text{ inches/ft}} = 0.017$ ft/day.
This is the rate at which water is leaving site. Thus, effluent discharge plus rain infiltration must be not greater than 0.017 ft/day.

Assuming an average daily rainfall infiltration of 0.01 ft/day, the average effluent discharge rate can be not greater than 0.017 ft/day - 0.01 ft /day = 0.007 ft/day.

The area needed for a wastewater discharge of 40 ft^3/day $= \dfrac{40 \text{ ft}^3\text{/day}}{0.007 \text{ ft/day}} = 5700 \text{ ft}^2$.
This could be provided, for example, by a SWAS footprint = 57' x 100'.

The measurement or estimate of the drainable porosity of the soil permits a calculation of how fast water leaves the site.

Effluent can be discharged at a rate such that the combined inflow of effluent and rain does not exceed the rate that water can leave the site. This analysis cannot be done in a vacuum however. If under certain intense rainfall conditions the site is saturated to grade or saturated within a reasonable distance from the system then effluent will break out during that period and at that point**.** The analysis is valid if the site has been observed for a time during high ground water and the maximum groundwater table elevation for a 2-week period is several feet below grade**.**

Both of these cases illustrate how, by careful observations over a period of time, the hydraulic capacity of a site can be determined without any sophisticated measurements or calculations. This type of analysis, if possible, provides the most reliable results because it considers all the hydrologic conditions of the site. However, such analyses should be carefully made and include all variables in order for their results to be reliable, and the results should be carefully checked with other methods presented in this section.

3. Analysis Based on Soil Hydraulic Conductivity and Topography

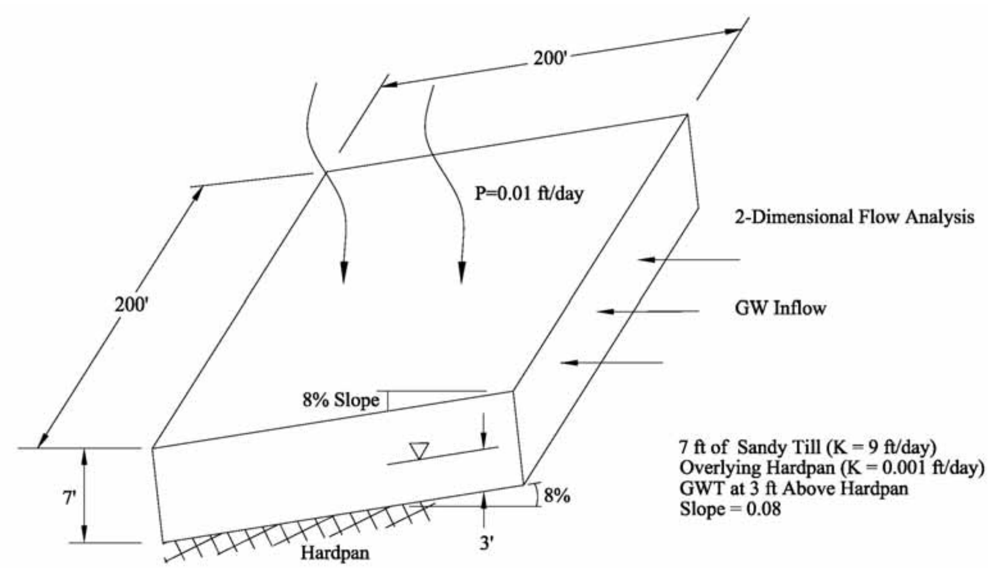
If it is not possible to monitor a site during the wet season, it is still possible to deduce the hydraulic capacity based on ground water levels inferred from mottling, soil strata identified in test pits, and hydraulic conductivity measurements.

As with any analysis, the inflow from all sources must be considered. Using approximate two-dimensional and three-dimensional hydraulic capacity analysis, and hydraulic conductivity data, it is possible to predict how a site will react to the inflow of effluent during the wet season. The greatest possible error is not in the actual calculations, but in the hydraulic conductivity values used and boundary conditions assumed. The Northeast is noted for the non-homogeneity of its soil deposits and this is particularly true of hydraulic conductivity. The various soil deposits both in and around the site must be accurately identified and evaluated, and the highest elevation of the ground water must be estimated.

The following two situations show how two and three-dimensional hydraulic capacity analysis can be used.

FIGURE No. 11 - (Case C)

ANALYSIS BASED ON HYDRAULIC CONDUCTIVITY AND TOPOGRAPHY



Note: Hardpan is a word often used to describe dense till.

Procedure – Case C

Ground water inflow (K x i x A) = 9 ft/day x .08 ft/ft x 3 ft x 200 ft = 432 ft^3/day (Q inflow). Rain inflow = 200 ft x 200 ft x 0.01 ft/day = 400 ft^3/day (Q rain)

Outflow = Inflow = 432 ft^3/day + 400 ft^3/day= 832 ft^3/day.

GWT Down Hill

Flow area required = $\dfrac{Q}{K \times i} = \dfrac{832\ ft^3/day}{9\ ft/day \times .08\ ft/ft} = 1155\ ft^2$

Depth of Flow = $\dfrac{1155\ ft^2}{200\ ft}$ = 5.8 ft above hardpan.

An additional 40 ft^3/day from a house discharged into a single 200-ft. trench across the lot would raise GWT by:

$\dfrac{40\ ft^3/day}{9\ ft/day \times .08\ ft/ft \times 200\ ft}$ = 0.28 ft. Therefore; the water table would be

0.28 ft + 5.8 ft = 6.1 ft above hardpan, or 0.9 ft below ground surface.

If the SWAS consisted of four 50-ft trenches side by side, GWT would rise:

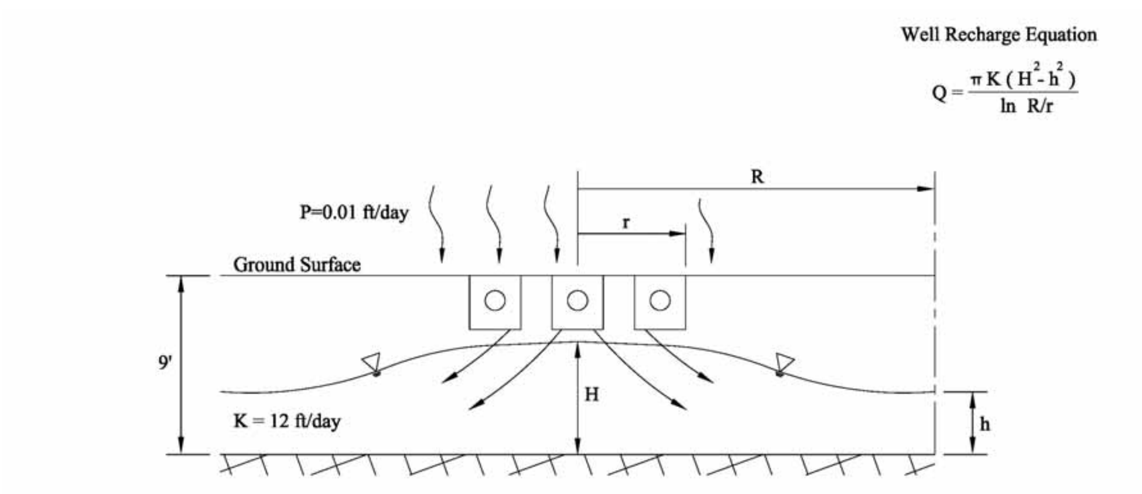
$\dfrac{40\ ft^3/day}{9\ ft/day \times 0.08\ ft/ft \times 50\ ft}$ = 1.1 ft

1.1 ft + 5.8 ft = 6.9 ft above hardpan; thus the water table would be essentially at the ground surface.

FIGURE No. 12 (CASE D-1)

PROCEDURE FOR 3-DIMENSIONAL HYDRAULIC CAPACITY ANALYSIS USING WELL RECHARGE EQUATION

(Note: See page 35 for definitions of terms in Well Recharge Equation)



Case D-1

R = 100', SWAS r = 50’ (This is the radius of a circular area having the same bottom area as the actual SWAS.)

Rain = 200 ft x 200 ft x 0.01ft/day = 400 ft^3/day = Q rain

ln R/r = 0.7, h = 4 ft, What is H?

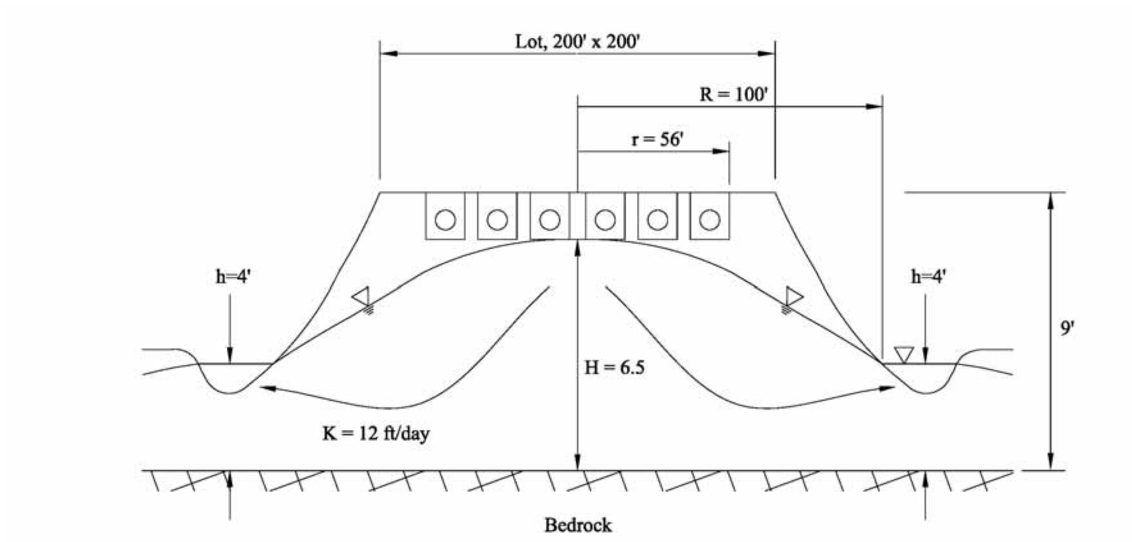
$$(H^2 - h^2) = \frac{Q \times \ln R/r}{\pi K} = \frac{400\ ft^3/day \times 0.7}{\pi \times 12\ ft/day} = 7.4\ ft^2$$

$(H^2 - 4^2) = 7.4\ ft^2$; $H^2 = 7.4\ ft^2 + 16\ ft^2 = 23.4\ ft^2$; H = 4.8 ft

GWT due to rain will be 9 ft - 4.8 ft = 4.2 ft below ground surface.

FIGURE No. 13 (CASE D-2)

PROCEDURE FOR 3-DIMENSIONAL HYDRAULIC CAPACITY ANALYSIS USING WELL RECHARGE EQUATION



Case D-2

What is the capacity of a 100' x 100' SWAS that is 2.5' deep and has an area equivalent to a circle with a radius of 56 feet, if the GWT is allowed to mound up to bottom of SWAS? (H = 6.5')

$$Q = \frac{\pi K (6.5^2 - 4^2)}{\ln 100/56} = \frac{\pi \times 12 \times (42.25 - 16)}{0.58} = 1{,}690\ ft^3/day$$

(Rain)

Capacity of SWAS = 1690 ft^3/day - 400 ft^3/day = 1,290 ft^3/day

Capacity of SWAS by bottom interface area required = 0.6 GPD/ft^2 x 100 x 100 = 6,000 GPD = 800 ft^3/day

Case C in Figure No. 11 shows how to calculate the effect of a house discharge on the position of the water table downhill, and the effect of the shape of the SWAS on the ground water table, using two-dimensional flow analysis. Cases D-1 and D-2 (Figures 13 and 14) show the use of a three-dimensional radial well recharge equation in calculating the hydraulic capacity of a level site.

Again, it should be pointed out that the greatest source of error in these analyses is not in the theory or calculations, but in the assumed values of depth of strata, hydraulic conductivity, and ground water elevation.

4. Site Drainage

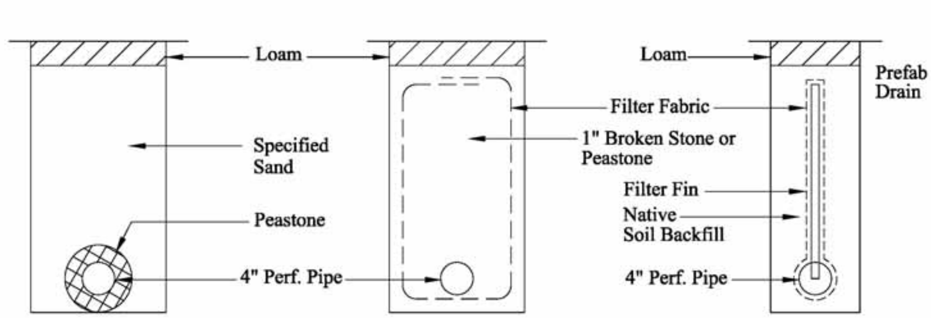
Many regulatory agencies do not recognize engineering predictions of the effect of subsurface draining systems on the hydraulic capacity of a site because of poor analyses in the past. The prevalent attitude is to "install the drainage system and see what effects it has during the wet season". In the absence of good analyses, this is the logical attitude; however, it is possible to predict the effect of surface and subsurface drainage systems if accurate site investigations are made.

Drainage systems are generally used to increase the hydraulic capacity of a site or to reduce the natural inflow of surface or subsurface water and make room for the effluent that is to be treated. Installing the drainage system "downhill" can increase the site hydraulic capacity, so that water can leave the site faster. Intercepting the inflow from uphill and bypassing it around the site through pipes or ditches can reduce natural inflow. It is not possible to bypass the precipitation although the surface can be graded to prevent ponding or water during very heavy rainstorms or when the ground is frozen.

The most reliable type of drainage system is a ditch that will intercept both surface and subsurface water; however, a ditch requires slopes of 1:3 or flatter and a deep ditch therefore requires considerable land space. The alternative is a buried pipe. Subsurface drains often consist of perforated pipe surrounded by a processed stone filter in a trench. This type of filter may allow the migration of fine particles into the pipe resulting in clogging of the pipe. In recent years filter fabrics have proven to be more cost-effective and more effective than processed stone filters. Figure No. 14 shows cross sections of typical curtain drains that operate satisfactorily.

FIGURE No.14

TYPICAL CURTAIN DRAINS



In order for a subsurface drain to function properly it must (1) let the water into the drain as fast as it can leave the soil, and (2) carry the water away as fast as it enters the drain. A proper filter constructed of either aggregate or fabric will fulfill the first requirement. The pipe must be large enough to fulfill the second requirement.

The analysis of the effects of a subsurface drain is most easily done assuming steady state flow conditions. The ground water and precipitation are assumed to be entering the site at a constant rate. This obviously doesn't occur often, but the effect of the drains can be determined for the worst condition, a long heavy rain for instance. During the operation of a SWAS, having the ground water table up high for a few days during an exceptionally rainy period is not detrimental, but it should not be high for longer than this. For this reason, in designing drains it is appropriate to assume a 4-5 day rain storm of about 0.1 ft/day.

It should be pointed out that many curtain drains are installed on sites where the ground water is high during the spring. A ground water table that rises in the spring indicates that water is entering faster than it can leave, which implies that lateral drainage is restricted by soil strata of low hydraulic conductivity. In such a situation the water entering a site as ground water may be only a fraction of the water entering as precipitation or surface water, and an uphill curtain drain may not lower the ground water table appreciably. The influence of a subsurface drain is very dependent on the hydraulic conductivity of the soil.

The following cases illustrate the calculations used to predict the effect of curtain drains.

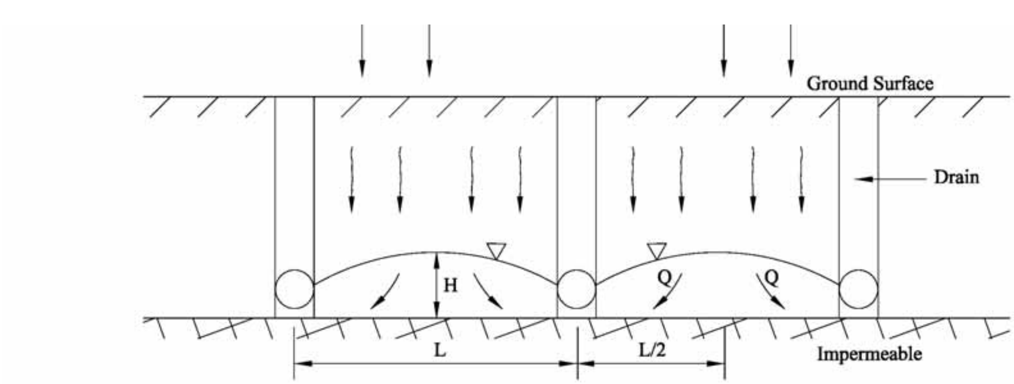
Case A. Figure No 15, shows a level site of 6 feet of sandy glacial till underlain by hardpan which is assumed to have zero hydraulic conductivity.

The general theory for drain operation on level ground underlain by an impermeable stratum is shown in Figure No. 15. All the water is assumed to come from rain that percolates downward to the ground water table and then moves laterally to the drains. It is assumed that the rain occurs at a constant rate long enough to develop a steady state flow. The desired position of the ground water table is assumed and the drain spacing necessary to cause this condition is calculated. The calculation can involve a series of flow nets, mathematical analyses, or an approximation as shown. All these methods give about the same answer.

FIGURE No. 15

EFFECTS OF CURTAIN DRAINS, CASE A

Rain = P = 0.1 ft/day



Calculation of Drain Spacing

$$Q = P \times \frac{L}{2}$$

$$Q = KiA = K \times \frac{H}{L/2} \times \frac{H}{2} = K\frac{H^2}{L}$$

$$K\frac{H^2}{L} = P \times \frac{L}{2}$$

$$\frac{L^2}{H^2} = 2\frac{K}{P}$$

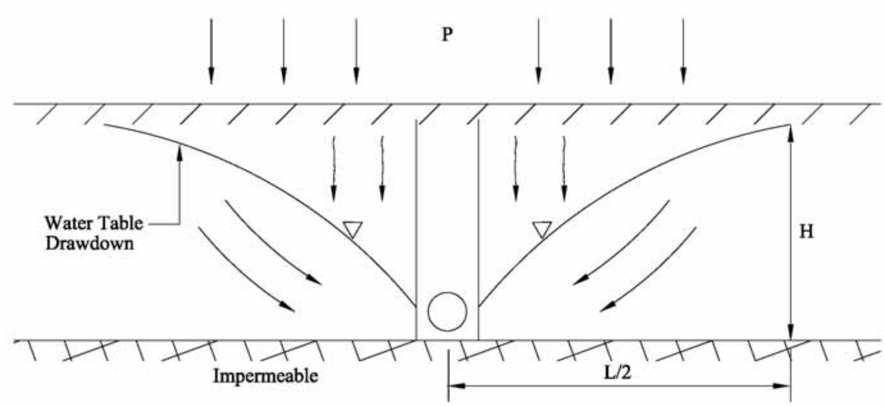
$$\frac{L}{H} = 1.4\sqrt{\frac{K}{P}}$$

Case B. If a single drain only is installed in an area with the ground water table at the ground surface, the effects of the drain will be felt at a distance slightly less than L/2, as shown in Figure 16:

$$\frac{L}{2} = \frac{1.4}{2}H\sqrt{\frac{K}{P}} = 0.7H\sqrt{\frac{K}{P}}$$

FIGURE No. 16

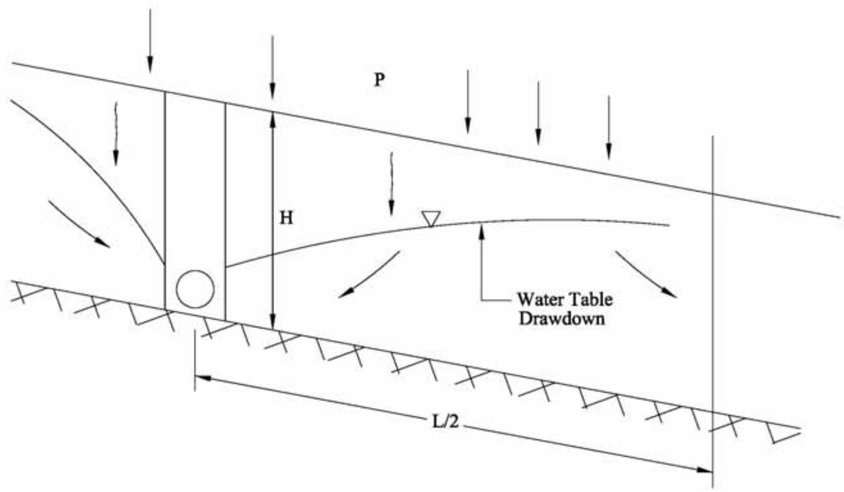
EFFECTS OF CURTAIN DRAINS, CASE B



In both the preceding cases, the water collected by the drains came from rain falling on adjacent ground. In Case C (Figure No. 17), a drain is installed on a slope to intercept ground water and lower the ground water downhill from the drain. If the drain is constructed down to the impermeable strata it will intercept all the flow from uphill. The effect of slopes up to 10% is minimal and the zone of influence is only slightly greater than on level ground.

FIGURE No. 17

EFFECTS OF CURTAIN DRAINS, CASE C



It should be pointed out that groundwater is drawn to the drain from a distance of about

$$0.75H\sqrt{\frac{K}{P}}.$$

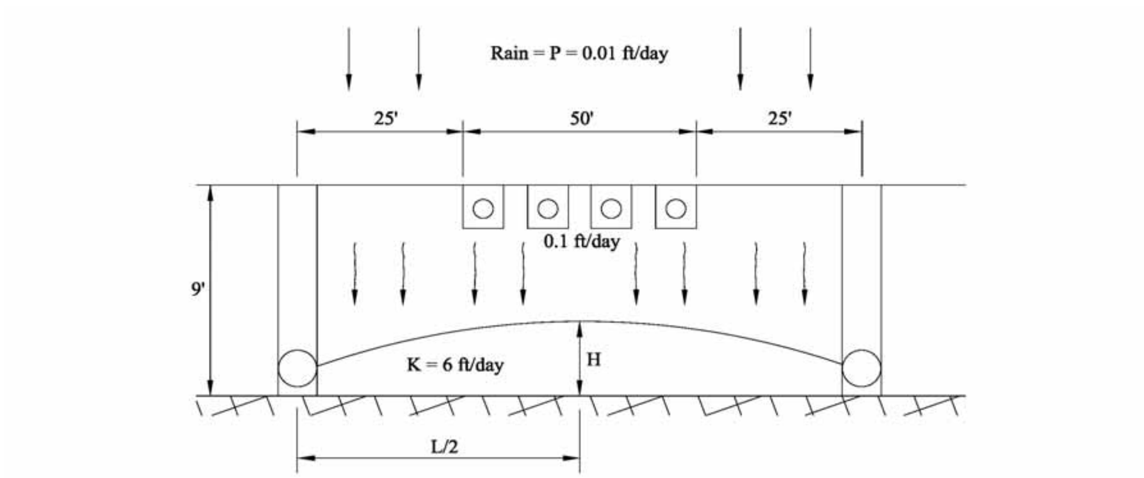
If the soil had a hydraulic conductivity of 4 ft/day and the drain was 3 feet deep, for a rain of 0.1 ft/day, the drain would influence the water table only 0.75 x 3 x $(4/0.1)^{0.5}$ = 14 feet downhill from the drain.

One very important conclusion that can be drawn from these calculations is that on a site in which rain is sufficient to raise the water table in the spring, any drainage system that will lower the water table in the area of a SWAS will generally collect effluent also. This is not a health hazard if the effluent travels through enough soil for treatment, and if there is sufficient dilution by rainwater.

An example of what happens if a curtain drain is installed around a SWAS is given in Figure No. 18.

FIGURE 18

EFFECTS OF CURTAIN DRAINS AROUND A SWAS



Input from SWAS = 0.1 ft/day x 50 ft = 5 ft^3/day/ft of SWAS

Equivalent P $= \dfrac{5\ ft^3/day}{100\ ft^2} = 0.05$ ft/day.

Rainfall = 0.01 ft/day

$\sum P + P_{equiv.} = 0.01ft. + 0.05\ ft = 0.06$ ft/day.
K = 6 ft/day.
L = 100'

Procedure:

$$\frac{L}{2} = 1.4H\sqrt{\frac{K}{P}} = 50\,ft$$

and,

$$H = \frac{L}{1.4}\sqrt{\frac{P}{K}} = \frac{100}{1.4}\sqrt{\frac{0.06}{6}} = 7.1\,ft$$

Therefore, the ground water table will be approximately (9-7.1) or 1.9' ft below ground surface in center of field. In this example, it is likely that the effluent will reach the underdrains before it is fully renovated in the soil.

5. Regulatory Constraints on Drains

The use of a drain to increase site hydraulic capacity presents some significant regulatory problems. If a drain is located on a property with a SWAS, and is down gradient from that system, the drain will collect treated effluent and discharge it to a surface water body. Unless the surface water body is designated as Class B by the Water Quality Standards adopted pursuant to Section 22a-426 of the Connecticut General Statutes, the system is in violation of these standards. Class AA, or A waters are not allowed as receiving water bodies for point source discharge of treated effluent. Assuming the drain as described discharged to a class B water course then a federal N.P.D.E.S. permit, administered by the Connecticut Department of Environmental Protection, would be required.

It is possible to design a drain at a distance that will provide adequate renovation of effluent to a drinking water standard prior to discharge. The problem is that the calculations supporting such an application are approximations dealing with complex site data. A regulatory agency that is charged with adopting and enforcing water quality standards must take a very conservative attitude toward such a proposal. The Department's policy with respect to acceptable use of drains for interception of ground water is provided in the Design Standards.

6. Fill

A satisfactory SWAS can be constructed with fill placed on bare impermeable bedrock. However, it should be noted that impermeable bedrock conditions are rarely encountered in CT, as virtually all of the bedrock in the state is fractured. Since it is very difficult to determine travel time in fractured bedrock, it would be necessary to place a hydraulic barrier (compacted clay liner, synthetic membrane, etc.) between the fractured bedrock and the fill so as to ensure that the travel time in the vertical direction through the barrier met the travel time requirements of the Department before any flow entered the fractured bedrock. The site preparation would consist of installing the hydraulic barrier and placing and compacting carefully selected soil to an adequate depth and lateral extent. The soil should contain enough sand and fines to provide adequate treatment of the effluent, and should have sufficient hydraulic conductivity to allow the water to move away laterally faster than it is applied. The soil should be of sufficient depth to allow a depth of unsaturated flow beneath the bottom of the system that meets the Department's requirements, and should be of sufficient lateral extent so that the effluent travels through the soil for a period of time that complies with the travel time requirements of the Department before it reaches a point of concern.

Such a SWAS, utilizing entirely imported soil, would be very expensive, would be prohibited by most health codes, and is not normally built for residences. However, marginal sites can be significantly improved by the placement of select fill. This improvement can be in the form of improved treatment by providing additional soil for the effluent to flow through before reaching wells or the ground surface, or in the form of increasing the hydraulic capacity of the site by providing soil with greater hydraulic conductivity or hydraulic gradient by elevating the system. Systems utilizing imported soil have been approved by the Department for large-scale on-site systems. Further discussion of fill systems is given Section X of this document.

7. Submission of Hydraulic Capacity Analysis in the DEP Permit Process

The presentation of a hydraulic capacity analysis should be logical and clear if it is to support a permit application. The following procedure is recommended:

Step 1. Assemble site-testing data, including soils, boundaries, topography, sensitive areas, hydraulic conductivity and estimated discharge volume.

Step 2. Calculate the size of SWAS per the Department's Long Term Acceptance Rate Criteria. This gives linear feet of SWAS; its configuration is determined by the balance of analysis.

Step 3. From this data, consider and develop a model for the predevelopment hydrologic conditions on the site. In other words, how does the site handle existing recharges, and where is effluent likely to go?

Step 4. Lay out a trial SWAS superimposed on the model.

Step 5 Perform hydraulic capacity analysis, check by alternate method if feasible.

Step 6 Demonstrate that under the worst case conditions the site can transmit the wastewater discharge with a reasonable safety factor.

Step 7 Define the plume direction, velocity and area to facilitate definition of the zone of influence of the system. This topic is covered in Section X of this document.

G. Other Methods For Hydraulic Capacity Analysis Under Low Hydraulic Gradient Conditions

1. General

Under low hydraulic gradient conditions (where the water table is essentially in a horizontal plane), the effects on the height of a ground water table caused by discharging pretreated wastewater into an unconfined aquifer are similar to the effects of an artificial ground water recharge basin. When water is discharged into an unconfined low gradient aquifer, the water table rises under and in the immediate vicinity of the discharge area, forming a ground water mound. Therefore, the height of the ground water mound that will develop beneath a subsurface wastewater absorption system must be superimposed on the pre-existing seasonal high water table when determining the vertical separating distance between the bottom of the system and the seasonally high water table. Ground water mound heights will be greatest in shallow aquifers of low hydraulic conductivity.

A detailed analysis of the growth characteristics of ground water mounds is a sophisticated mathematical process, involving complex equations that are difficult to evaluate unless certain simplifying assumptions are made. Many researchers have devised both analytical and numerical solutions to the equations describing pressure and flow in porous media with certain boundary and initial conditions. Many of these solutions have been incorporated into computer models.

Because of the significant data requirements and considerable expertise required for use of the complex numerical models, they should not be used as part of the submittal for a Discharge Permit without first receiving approval of the Department.

The Department recommends that initial analysis of the effect of ground water mounding be made using the methods presented in this subsection. If the results indicate that the hydrogeologic conditions appear suitable to accept the proposed design discharge without causing the resulting mound to impinge on the unsaturated zone (vertical separating distance) required by the Department, the results can then be checked using one of the computer based analytical models discussed below. Such models may also be used where the results obtained from the initial analysis indicate the vertical separating distance is problematic, ("too close for comfort") and a more detailed approach may be warranted.

Analytical models contain a closed-form or analytical solution of the field equations subject to specified initial and boundary conditions. Because of the complex nature of ground-water problems, the analytical solutions generally are available for problems that entail a simplifying nature of the ground-water system, its geometry and external stresses (U. S. EPA-1993).

The areal extent and height of a ground water mound are dependent upon the size and shape of the discharge area, the duration and rate of discharge, the stratigraphic configuration of the subsurface formations, the saturated and unsaturated hydraulic properties of the geologic materials, and the aquifer boundary conditions. Under constant recharge conditions, and in an aquifer of infinite extent, the height of a ground water mound will continue to increase over time, although at ever reducing rates.

Under constant recharge to an aquifer whose extent is limited by boundary conditions, a ground water mound will continue to grow until some control, potential or lateral, provides a limit. Lateral controls can consist of an impermeable boundary (no flow condition), a ground water divide, or the presence of a stream, lake or other surface water body (constant head condition). In the latter case, for a large horizontal extent of these types of boundary conditions, the ground water mound approaches equilibrium with a constant recharge rate. Potential controls occur when the ground water mound builds up to the recharge surface; with a fixed maximum height the ground water mound gradient and hence the recharge rate must decrease with time (Todd - 1980).

The analysis of ground water mounding beneath a subsurface wastewater absorption system (SWAS) using an analytical model can be carried out by assuming that the SWAS is equivalent to a recharge basin of similar dimensions and that the recharge rate is constant with time. While the rate of discharge into a SWAS actually is intermittent, the assumption of a uniform recharge rate is conservative. In addition to the proposed discharge, the recharge rate should include the effect of infiltration of precipitation, as previously discussed.

2. Well Discharge Equation

It should be noted that the equation used in the Pit Bailing Test method for determining hydraulic conductivity (Subsection E 4), and in the example given for 3-Dimensional Hydraulic Capacity Analysis (Subsection F 3) is an analytical model known as the "simple well formula" (Cedergren- 1989). While developed to estimate the yield of a well under steady-state conditions, it can also be used to estimate the local rise (mounding) of a water table in response to a recharge well operating at steady-state conditions in an unconfined aquifer. This formula is based on the following assumptions (Cedergren, ibid.):

a. The pumping well penetrates the full thickness of the water-bearing formation.
b. A steady-state flow (equilibrium) condition exists,
c. The water bearing formation is homogeneous and isotrophic and extends an infinite distance in all directions.
d. The Dupuit assumption is valid [i.e.: the hydraulic gradient at any point is constant from top to bottom of the water-bearing layer and is equal to the slope of the water surface].

If the water table is slightly sloping, the bottom boundary of the water bearing strata may be drawn parallel to the original water table and the computation carried out the same as for a level water table (Cedergren, ibid.).

Since the simple well formula is based on radial flow, it is basically applicable to a circular recharge basin. However, it has also been found suitable for a square recharge basin, where the area of the square basin is equivalent to the area of a circular basin of radius R. The formula is less suitable for rectangular basins where the basin length is much greater than the basin width, which is most often the case with subsurface soil absorption systems.

The simple well formula, (as given in Subsection F) is:

$$Q = (\pi K (H^2 - h^2))/\ln (R/r)$$

where k = saturated hydraulic conductivity, H = depth from top of mounded ground water table to an impermeable lower boundary, h = original saturated thickness of aquifer, R = radial distance from center of recharge basin to an aquifer boundary or an assumed outer limit of the mound, and r = the radius of the recharge basin. This equation can be used as a first approximation of the ground water mound height development beneath a SWAS.

Analytical solutions of the governing equations for saturated flow have been made for certain shapes of recharge basins (square, rectangular, and circular). These solutions are based on assuming the aquifer to be homogeneous, isotropic (hydraulic conductivity being independent of direction of measurement), of infinite areal extent, and having an initially horizontal water table and impermeable bottom. They can be adapted to "real world" conditions by making other reasonable assumptions and using certain superposition techniques.

In unconfined aquifers with low hydraulic gradients, the hydraulic properties of unconfined sediments that control their ability to accept water are the hydraulic conductivity, saturated thickness, and specific yield of these deposits. The specific yield of an aquifer is a dimensionless parameter that is defined as the volume of water that an unconfined aquifer releases from storage per unit surface area of aquifer per unit decline in the water table (Freeze and Cherry - 1979). It is also sometimes referred to as "drainable porosity", the ratio of the volume of water that would be released from the soil pores under gravity drainage conditions to the total volume of the pores. Thus, this parameter is related to the porosity of the soil, although some soils having high porosities (very fine sandy silts and clays) may have low specific yields. It is also related to the "Available Water Capacity" parameter utilized by soil scientists and agronomists.

3. Computer Models

There are several computer models available for analytical solutions of ground water mounding. Among these are the following:

Flow From Wells and Recharge Pits, Version 2.0 is a proprietary program available from the Groundwater Program, Colorado State University, Ft. Collins, CO, or from the IGWMC, which lists the program as CSUPAW (Colorado State University Pit and Well). The program runs under DOS on desktop computers.

MOUNDHT, a proprietary program originally developed in 1992-93 by Professor E. John Finnemore, Department of Civil Engineering, Santa Clara University, Santa Clara, CA 95053. The program runs under DOS on desktop computers.

Estimation of Groundwater Mounding Beneath Septic Drain Fields. This program is an adaptation of the MOUNDHT program to a WINDOWS environment. It was developed by the Center for Resources Studies, Technical University of Nova Scotia (Mooers-1994).

GRAMP, (Groundwater Recharge and Mounding Program) is an interactive program designed to predict ground water mounding below rectangular basins. This program is based on the Hantush equation (Hantush-1967) equation for ground water flow for recharge from a rectangular basin. The program runs under Windows 95 and 98.

Hantush's Rectangular Recharge Basin Analysis, a proprietary program based on the Hantush method of solving the governing differential equations for ground water flow for recharge from a rectangular basin (Hantush-1967), it is an interactive program that predicts the growth and decay of ground water mounds in response to uniform percolation. The program runs under DOS on most desktop computers.

It should be noted that the Hantush method of solving the governing differential equations for saturated ground water flow to calculate ground water mounding resulting from recharge from a rectangular basin (Hantush-1967) may also be solved manually (Cantor and Knox - 1985; Walton - 1970).

However, the use of computer programs for solving the Hantush equations makes the computations quicker, easier and less subject to error and can provide more information than is readily available using the manual solution. The use of such computer programs also facilitates performing sensitivity analyses by allowing rapid "what if" iterations by changing input data values.

All of the analytical models described above have limitations with respect to accounting for boundary conditions. Accordingly, it is sometimes necessary to modify the results obtained from these analytical models by use of methods of superposition using the theory of images.

When an aquifer is bounded on one side by an impermeable boundary and on the other side by a stream (an equipotential, or "constant head" boundary), the ground water mound profile resulting from operation of a recharge basin between the two boundaries will be higher and flatter on the impermeable boundary side and lower and steeper on the stream side of the basin. To predict the height of the ground water mound above the normal ground water level in such aquifer systems, the principal of theory of images (image well theory) can be used to simulate the effects of such boundaries (Ferris, et al.-1962; Lohman -1972; McWhorter and Sunada - 1977; Freeze and Cherry -1979).

Finally, while the data needs of these analytical models are modest and use of the models is deceptively simple, sound judgement must be used in selecting the data to ensure that it is reasonably representative of the hydrogeologic conditions at the project site. As stated by Finnemore in the documentation for GROUND WATER MOUNDHT, "Such prediction methods need to be used with judgement by experienced engineers who are aware of their limitations."

4. Hydraulic Load Testing

A technique sometimes used for determining the hydraulic capacity of a proposed SWAS site is a hydraulic load test that simulates the ground water mounding that will result from the proposed discharge. Demonstration of hydraulic capacity is accomplished by hydraulic loading of trenches or basins while monitoring of water levels in ground water monitoring wells located immediately adjacent, and proximal to, the hydraulic loading area(s).

It is recommended that the results of hydraulic load tests should not be used as part of the submittal for a Discharge Permit without first receiving approval of the Department with respect to methods and procedures for conducting such tests in order to avoid the Department's refusal to accept such results.

The technical feasibility of conducting hydraulic load tests will depend upon the availability of an adequate supply of water that will permit conducting the test at the hydraulic loading rate anticipated for the proposed SWAS over a sufficient length of time to establish a relatively "stable" ground water mound. This in turn requires that an adequate water supply source is available within a reasonable transport distance and that a suitable means of transporting the water from the source to the test site is available or can be constructed.

Where the quantity of water required for the test is greater than 50,000 gallons per day, a diversion permit will need to be obtained from the CT DEP. In addition, permission to use the water will need to be obtained from the owner of the water supply source, and temporary easements may be required for constructing a temporary water transmission line from the source to the test site. A permit from the local Inland Wetlands and Watercourses Agency is also likely to be required. A determination will also have to be made whether the quantity of water to be taken from the source will be such as to cause detrimental effects to the source or will adversely effect the supply available to nearby developed areas that depend on that source for their water supply. If such adverse effects will occur and cannot be mitigated, hydraulic load testing should not be done.

5. Tracer tests

Tracer tests can be used to demonstrate the area proposed to be utilized for an SWAS has adequate aquifer hydraulic characteristics to provide the minimum travel time to the nearest property line, or point of surface breakout in accordance with the pathogen travel time criteria of the Department. Tracer tests can also be used to delineate the zone of influence of the proposed discharge. Demonstration of travel time and zone of influence can be accomplished by sampling and testing of water from selected ground water wells located immediately adjacent to, and downgradient of the point of tracer injection for the presence and concentration of the tracer.

Tracer tests are often conducted in conjunction with hydraulic load tests. The ground water monitoring wells to be sampled can be determined from phreatic surface contour maps generated from ground water monitoring data (water table elevations) developed prior to and during a hydraulic load test. The zone of influence can be determined from the presence of the tracer in the various monitoring wells.

The tracer method of estimating ground water travel times and the zone of influence of a discharge to the ground water has disadvantages. These include the long periods of time normally required for tracers to move significant distances because of low ground water velocities, and the need for numerous monitoring wells and sample analyses. Each monitoring well may have to contain a nest of sampling tubes designed to sample the ground water at different elevations below the water table in order to be reasonably certain of intercepting the tracer plume. Thus, a properly designed tracer test is apt to be time consuming and costly.

As in the case of a hydraulic load test, the results of tracer tests should not be included as part of the submittal for a Discharge Permit without first receiving approval of the Department.

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SECTION VII HYDROGEOLOGIC STUDY AND REPORT

TABLE OF CONTENTS

SECTION VII HYDROGEOLOGIC STUDY AND REPORT

A. Introduction

As part of the information accompanying a Permit application, a hydrogeologic study should be conducted in the area of a proposed OWRS and a report thereon submitted to the Department.

1. General

The study and report should:

a) Describe the regional and local hydrogeologic conditions.
b) Indicate if the discharge from the proposed OWRS will occur within a designated aquifer protection area or public water supply watershed.
c) Define the areal extent and physical properties of the site soils that will assimilate and transmit the discharge.
d) Describe the effects on the water table due to the proposed discharge to the ground water.

The information required in a hydrogeologic study is technical and can be quite complex. Therefore, only persons with significant experience in the fields of hydrogeology and soils should be retained for the study. Preliminary desktop and field studies should be conducted in sufficient detail to determine if there is a reasonable possibility for constructing an OWRS at the project site that will meet the objectives of the Department. These preliminary studies are important to minimize unnecessary effort by the consultant(s) and Department staff.

Where the results of the preliminary studies indicate there is a reasonable possibility for constructing an OWRS at the project site, and prior to beginning a final hydrogeologic study, a plan for the study should be discussed with Department staff for input. Such input will not guarantee approval of the Applicant's hydrogeologic study or permit application, but will help in determining the site-specific requirements for the study and may eliminate the need for multiple and costly iterations of the study. The Department may require the Plan and Hydrogeologic Study and Report to contain some or all of the following elements:

a. Narrative of proposed project.
b. Characterization of site soils in conformance with U.S. Department of Agriculture Natural Resource Conservation Service (NRCS) descriptions.
c. Ground water characterization, including depth from existing ground to seasonal high water table, hydraulic gradient and local direction of ground water flow.
d. Maps - Area Map, Site Plan, Surficial Geology Map, Bedrock Geology Map, Soils Map based on NRCS soil mapping, and Groundwater Contour Map.
e. Description and quantification of proposed discharges to the ground water.
f. Supporting calculations, tables and figures.
g. Conclusions as to project requirements for meeting the Department's criteria for a discharge to the ground waters of the State.

B. Elements

1. Narrative

The narrative portion of the hydrogeologic report should contain the following:

- Purpose and scope of the hydrogeologic study.
- Historical land use of the site.
- Land use in the vicinity of the site.
- Regional and local bedrock and surficial geology.
- Surface waters and drainage patterns in the project area.
- Wetlands and watercourses in the general area of the project, mapped by a Certified Professional Soil Scientist.
- State Water Quality Classifications of surface waters and ground waters in the general area of the project.
- Location of proposed OWRS with respect to any nearby designated public drinking water supply watersheds, aquifer protection areas and other points of concern.
- Discussion of field activities.
- Conclusions relative to the hydrogeologic conditions at the site, including depth from surface to seasonal high water table (SHWT), soil hydraulic capacity, any limiting conditions such as impermeable soils, highly permeable soils, perched water tables, shallow bedrock, fractured bedrock with water table in the bedrock, etc.

2. Characterization of Site Soil Conditions

Site soil conditions shall be characterized by the following field and laboratory activities:

- Soil borings and/or test pits, of sufficient number and depth to characterize site soils that will assimilate and transmit the proposed discharge.
- Soil samples collected by standard soil sampling techniques.
- Particle size distribution, by both sieve and hydrometer.
- Soil horizon classifications under the U.S. Department of Agriculture NRCS soil classification system.
- Sampling and testing for saturated horizontal and vertical hydraulic conductivities, in sufficient numbers to define the hydrogeologic regime on the site. (Discuss method(s) of sampling, testing and statistical analysis of test results).

3. Ground Water Characterization

Ground water conditions beneath the site, characterized as follows:

- Depth to seasonal high water table (SHWT).
- Local aquifer gradients.
- Direction of ground water flow.
- Horizontal velocity of ground water flow.
- Aquifer boundary conditions.

- Location of existing ground water discharges to surface water.
- Ground water mounding calculations.
- Background ground water quality, if there are existing discharges to ground water in close proximity to the proposed discharge site (Discuss need for this information with Department staff).

4. Maps

a. A detailed Area Map, at a scale of one inch equals 500 feet or less, with scale indicated both numerically and graphically, showing:

- Boundaries of the site on which the OWRS is proposed to be located.
- Topography, with existing surficial contours at contour intervals of 10 feet or less,
- Planimetric features.
- All public drinking water supply wells within one-half mile radius of the boundaries of the proposed OWRS and, if available, any associated aquifer protection or wellhead protection boundaries.
- Surface water bodies and watercourses within one-half mile radius of the boundaries of the proposed SWAS.
- Private drinking water wells within 1000 ft of the boundaries of the proposed OWRS.
- Coastal boundaries, if any, as defined by section 22a-94 of the General Statutes as amended to date.
- 10 year and 100 year flood boundaries as defined by FEMA, if any.

b. A Site Plan, drawn to a scale of one inch equals 50 feet or less, with scale indicated both numerically and graphically, showing:

- Property boundaries, as determined by a Licensed Land Surveyor.
- Surficial contours, both existing and proposed, at contour intervals of 2 ft or less, and planimetric features, including but not limited to existing and proposed buildings, as determined by a Licensed Land Surveyor.
- Boundaries of soil map units derived from NRCS Soil Surveys.
- Location of existing or proposed wastewater pretreatment facilities and SWAS.
- All existing and proposed wells on the site.
- All subsurface pipes (water, sewer, gas, drainage including underdrains).
- All surface waters and wetlands located on the site and immediately adjacent thereto, (wetlands to field delineated by a Certified Professional Soil Scientist and mapped by a Licensed Land Surveyor).
- Ground water contours at intervals of 1 foot or less.
- Locations and depths of all water table monitoring wells, soil test pits, borings, slug tests, test wells, pump tests, hydraulic conductivity test pits, etc., located and mapped by a Licensed Land Surveyor.
- Proposed locations and depths of ground water level monitoring wells. [1]
- Proposed locations and depths of ground water quality monitoring wells.[2]

[1] Subject to revision after review by the Department.
[2] Subject to revision after review by the Department.

- Bedrock Geology and Surficial Geology, derived from maps published by the U.S. Geological Survey, showing boundaries of proposed project, covering an area extending to 1000 feet outward from the project boundaries.

5. Calculations

Appendices should include calculations for each of the following:

- Ground water flow direction.
- Ground water flow velocity.
- Hydraulic gradient of the aquifer.
- Hydraulic conductivities.
- Ground water mounding, and depth to SHWT from existing ground after superpositioning of ground water mound on SHWT.
- Site hydraulic capacity.
- Travel Time(s) from SWAS to points of concern

6. Raw Data

The following raw data should be included in each hydrogeologic report (All data logs must show the date the data was obtained. Undated data will not be accepted.):

- Logs of soil borings, test pits, monitoring wells, hydraulic conductivity field tests.
- Potable water well logs within 1000 feet of the discharge, if available.
- Laboratory data sheets for:
  - Soil particle size data.
  - Hydraulic conductivity data.
  - Background ground water quality data, if required by the Department.

7. Locating Soil Borings, Test Pits and Water Table Monitoring Wells

All soil borings, test pits, test wells and ground water monitoring wells shall be located from known and recoverable reference points or benchmarks so that they may be accurately located on the Site Plan.

8. Confirmatory Site Testing By Department Staff

Upon completion of preliminary studies by the Applicant's consultant(s) that conclude there is a reasonable possibility for constructing the proposed OWRS at the project site, and tentative concurrence by the Department with the conclusions thereof, a Department staff member and local health department staff member may perform a site reconnaissance prior to conducting confirmatory site testing. This would allow the Department staff member to get a first hand look at the site, which may affect the site testing to be conducted.

SECTION VIII ELEVATION OF SEASONAL HIGH GROUND WATER TABLE

TABLE OF CONTENTS

TABLES

SECTION VIII ELEVATION OF SEASONAL HIGH GROUND WATER TABLE

A. Introduction

Predicting the elevation of the seasonal high water table (SHWT) is necessary in order to ensure that an adequate depth of unsaturated soil will exist below an SWAS. There are basically two methods for making such predictions: measuring the depth of ground water in a monitoring well during the annual period of seasonally high ground water, and by evaluating redoximorphic features (RMF) in the soil. Neither of these methods is foolproof, and thus prediction of the SHWT is not an exact science.

B. Measurement of Depth to SHWT

Ideally, installing ground water monitoring wells at a proposed SWAS site and determining the depth to the water in such wells during the period of maximum seasonal high ground water would be the method of choice for determining the SHWT. However, this can only be done during a short period of each year, and the likelihood that the maximum SHWT will be encountered at the site being investigated during the period of such investigation is problematic, due to annual variations in the SHWT**.** One method that may resolve this problem in a reasonable manner is discussed below.

Frimpter (1983) developed a method for the estimation of maximum ground water levels at any time of the year in Massachusetts based on statistical analyses of ground water observation well records for different hydrogeologic situations common in Massachusetts. He stated that an estimate of the probable high ground water level at a site could be made on the basis of the assumption that water level fluctuations at the site are directly correlated with water-level fluctuations at a selected observation well. The method developed involves adding an estimated potential rise to the water level currently measured at a site. However, the method developed by Frimpter cannot be currently used in Connecticut because similar statistical analyses of observation well records in Connecticut are not available. The Department may consider an adjustment for on-site water table measurements made using the following relationship:

Depth to SHWT on-site = Current depth to WT on site x (minimum depth to SHWT of record at the Index Well Site / current depth to WT at the Index Well Site).

The data for the Index Well(s) SHWT can be obtained from U.S. Geological Survey (U.S.G.S.) records. The U.S.G.S has maintained and published ground water level records for 71 wells in Connecticut. Records for selected locations in Connecticut were published monthly, beginning in 1967, in "Water-Resources Conditions in Connecticut" until September 1999, after which the data were made available on the Internet (http://waterdata.usgs.gov/ct/nwis/gw). In addition, ground water level data for the period 1935 - 1974 were published as Water-Supply Papers under the title "Ground-water Levels in the United States". Ground water level data are given in feet below land surface.

The published data on ground water levels includes: current month water level and date measured, the water level in the same month of the previous year, the previous month water level, the maximum and minimum water levels in the period of record for the current month, and the median water level for current month. Graphs are given for selected wells depicting the highest, lowest and monthly median water levels of record and the water levels in the current and previous year. Information is also provided on precipitation records maintained for the primary and several secondary weather stations in the State.

More complete ground water data, for all of the USGS ground water monitoring well sites, are available in the annual USGS Water-Data Report publication “Water Resources Data-Connecticut” (U.S.G.S.-Annual Publication). This data includes descriptions of well location, descriptions of the aquifer soil, well characteristics, datum, period of record and extremes for period of record and may prove helpful in evaluating the suitability of using a particular USGS ground water monitoring well as an index well.

The current number of monitoring wells in each county of the State, and the aquifer types represented are shown in the following table.

TABLE 1

U.S. GEOLOGICAL SURVEY GROUND WATER MONITORING WELLS IN CT

AQUIFER TYPE

COUNTY	STRATIFIED DRIFT	TILL	BEDROCK
Fairfield	3	1	3
Hartford	7	6	0
Litchfield	4	1	0
Middlesex	4	5	0
New Haven	8	4	2
New London	3	2	0
Tolland	8	4	0
Windham	5	1	0
Totals:	42	24	5

The index well sites should be carefully chosen so as to be representative of the climate, soil and topographic conditions at the project site. The current SHWT at the project site should be determined from water level measurements made in the on-site water table monitoring wells during the wettest season of the year, which usually occurs from late February through the end of April of each year.

A review of precipitation data should also be made, to determine if the preceding annual total precipitation was at, below, or above the mean annual precipitation for the locality in which the SWAS is proposed to be located. The same determination should be made for the location of the Index Wells. If the preceding annual total precipitation data at the site of the proposed SWAS are not similar to the data at the Index well, the results of the calculation of SHWT at the project site will be suspect.

C. Predicting SHWT Using Redoximorphic Features (RMF)

Predicting the SHWT using RMF, although often used, is not a foolproof method, as is indicated by the following review of published articles.

Soil color criteria (reflecting redoximorphic features) developed by the NCRS can be used to predict the level of the maximum SHWT. Redoximorphic features, (a gray or bluish-gray colored soil matrix), and mottles are formed by the process of reduction, translocation and/or oxidation of iron and manganese oxides. They can be categorized as redox (oxidation/reduction) depletions and redox concentrations (low and high chroma mottles, respectively) and a gleyed (grayish or bluish-gray colored) matrix. When the soil becomes saturated, oxygen is depleted, and organisms reduce soil iron from the ferric form which is red-orange and insoluble to the ferrous form that is soluble. The ferrous iron can leach away leaving a duller colored area behind. Intermittent wet and dry periods lead to mottled duller and brighter areas, and the upper limit of these mottles has been used to predict the seasonal high water table. Soils with black surfaces and no bright colors at all in the profile usually have a very high water table.

The NRCS recognizes soil matrix or mottle colors of chroma 2 or lower, (gray colors), as indicating horizons that are saturated for at least part of the year. However, using this method to determine the SHWT may sometimes lead to serious overestimation or under-estimation of the SHWT (Cogger-1985). Two problems arise when estimating seasonal high water tables from redoximorphic features (RMF). First, it has been shown that color criteria are not always a good indicator of seasonal high water tables, and second, it is not clear what duration of high water tables will lead to inadequate treatment of wastewater (Cogger 1989).

Warkentin and Harward (1978) stated that depth to mottling is a useful tool but not an infallible one, and we need to understand its limitations. Depth to mottling tends to underestimate the maximum height of the water table. Barton (1980), on the other hand, concluded that soil mottling is a highly reliable indication of soil water saturation, and the duration of water saturation can normally be correlated with soil mottling intensity.

Carlile, et al. (1981) stated that soil color proved to be a valuable predictor of seasonal high water table at all sites studied, except where the water table was lowered by large-scale agricultural drainage. In their study, they measured the water tables in the monitoring wells and the seasonal highs (persisting for one month or more) and event highs (short duration after heavy precipitation events) were noted. Using the highest month as a basis, the water table predictions based on mottling were found to be somewhat conservative, with 8 of 16 being accurate and 7 of 16 being high. Using the highest event as a basis, 8 of 14 predictions were too low because these events would be of too short a duration to significantly alter the chemistry of the soil. Similarly, short-term events could not be expected to affect the treatment ability of a drainfield. Although the soil properties failed as predictors of the event highs, these are not as important as the seasonal high water tables in determining septic system operation.

Cogger (1985) also indicated that brief high water tables usually do not have a great effect on septic system operation although occasional temporary effects have been noted. Thus, these infrequent, short-lived high water tables can usually be discounted. Owens, et al. (2001) found their research indicated that any redoximorphic feature (Chroma $\leq$ 2, Chroma $\geq$ 2, and Fe-Mn nodules) should be considered to be an indicator of a seasonal water table, and the frequency and duration of the water table will be related to the intensity of the RMF that is expressed.

Statistical analysis by Williams, et al. (2001.) of daily water table data collected at several wells in eastern North Carolina over a three year period showed that redox depletions (>2%) were significantly correlated to periods of > 21 days saturation (R^2=0.93). In their study, the overall agreement was fair between the estimated water table depth based on morphology and that of the 21-day method for the seven soils investigated, ranging from 2 cm deeper to 28 cm shallower. Veneman (1997) stated that, in Massachusetts, the presence of 5% redoximorphic features is used as an indicator of the mean seasonal high ground water elevation and that this will predict the seasonal high ground water elevation for about 8 out of 10 years.

D. Establishing Depth to Seasonal High Water Table.

It is recommended that water table elevations determined from both on-site water table monitoring wells and redoximorphic features (soil coloration and mottles) be used to estimate the Seasonal High Water Table (SHWT). Where water table elevations from monitoring wells are used, a review of the references listed herein indicates that the SHWT could be defined as the maximum level at which the water table remains over a 21-day period during the wet season in a representative year. It is standard practice to refer to USGS monitoring well data to determine an appropriate ground water monitoring period. When redoximorphic features are used, the Department defines the SHWT as being located at the elevation where the sidewall area of an exposed soil horizon is mottled. The highest water table elevation that is determined through these methods should be used for design of a SWAS.

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SECTION IX BASIC PRETREATMENT

TABLE OF CONTENTS

SECTION IX BASIC PRETREATMENT

A. GENERAL

The basic facilities for pretreatment of domestic wastewater consist of one or more septic tanks, preceded by one or more grease traps in cases where the wastewater contains fats, oils and greases (FOG) in amounts greater than that found in household wastewater.

B. SEPTIC TANKS

A septic tank is a passive device designed for:

- substantial separation of the solid and liquid components of wastewater;
- storage of the separated solids;
- limited digestion of stored organic matter by anaerobic and facultative bacteria;
- discharge of the liquid component of the wastewater to a subsurface wastewater absorption system (SWAS) for further renovation in the subsurface soils.

Separation of the solid and liquid components of the wastewater is brought about by settling of solids having a specific gravity greater than water to a bottom sludge layer, and floatation of FOG and other materials having a specific gravity less than water to a floating scum layer. Rising gas bubbles, resulting from the digestion of the organic matter in the lower reaches of the tank, also aid in carrying FOG with them to the top of the liquid. Factors that affect such separation include tank surface area, length-to-width ratio, depth, detention time, flow baffling provisions, number of tank compartments, the temperature of the wastewater discharged to the tank, and the ambient temperature of the soil surrounding the tank.

It is well established that surface loading rate is a major criterion for effective removal of settleable and floatable solids. Surface loading rate is a function of the effective horizontal area of the tank (inside clear length x clear width) and the rate of flow of wastewater through the tank, and is expressed as gallons per day per square foot of effective horizontal area. Time is also required for these solids to settle down to the lower portion of the tank volume where they will be stored and partially digested. This time factor is reflected in the detention time, which depends upon the liquid volume of the tank (inside clear length x clear width x liquid depth) and the rate of flow of wastewater through the tank. Baffling is required to promote the relatively quiescent conditions needed for effective solids separation and to hinder the escape of solids with the tank effluent. While not all investigators agree, it is generally accepted that separating the tank into two compartments assists in sedimentation of the settleable suspended solids. In fairly recent times, effluent filters have been used as an enhancement to the outlet baffles for more effective retention of solids in the tank.

The temperature of the wastewater in the tank and the ambient soil temperature have a significant effect on the settling rate/floatation rate of the solids and the microbial action in the tank that affects partial digestion of the solids. Lower temperatures aid in separation of the lighter solids by causing the soluble FOG in the wastewater to congeal and float to the higher portion of the tank, but hinder the settlement of the heavier solids, due to the increased liquid viscosity, and reduce the microbial action.

Higher temperatures aid in separation of the heavier solids (by decreasing the liquid viscosity) and increase the microbial action, but hinder the separation of FOG from the wastewater. Normal residential wastewater temperatures are usually not considered a significant factor. However, the higher temperatures of wastewaters discharged from facilities such as food processing and serving establishments can negatively effect the separation and retention of FOG. The ambient temperature of the soil can affect the temperature of the tank contents by heat transfer through the bottom, top and sidewalls of the tank.

The pollutant concentrations in the raw wastewater will be significantly reduced in a properly functioning septic tank. The septic tank will also serve to reduce instantaneous peak wastewater flow rates and pollutant loading rates. It should be noted that due to the large safety factor inherent in sizing of residential septic tanks, they normally provide detention times of several days. However, septic tanks for large-scale on-site systems typically have much shorter detention times and higher flow-through velocities. Therefore the pollutant removals reported for residential septic tanks are often not realized for the large-scale system tanks. This is important to remember when computing the organic and nitrogenous loading in the septic tank effluent.

Important design considerations for septic tanks include:

- Volume
- Surface Area
- Inlet and outlet provisions
- Internal baffling provisions
- Material durability
- Structural integrity
- Safe and controlled access for removal of accumulated scum and sludge and for inspection of inlet and outlet provisions.

C. GREASE TRAPS

The exterior, underground type of grease trap is a passive device designed to collect and retain the fats, oils and greases (FOG) normally found in wastewaters discharged from food processing and serving facilities. Grease traps, sometimes referred to as underground grease interceptors to differentiate them from the smaller automated grease recovery units (AGRU) installed within a building where the FOG originates, are similar in construction to septic tanks but differ in their inlet and outlet and baffling arrangements. The effluent from grease traps is discharged to a septic tank or other pretreatment facility before being discharged to a SWAS.

Grease traps accomplish separation of FOG by taking advantage of the difference in specific gravity between FOG and liquid in the trap and the cooling effect of heat transfer between the trap contents and the surrounding soil. Under quiescent conditions, a significant percentage of the FOG and suspended solids in the wastewater will be removed in the trap. FOG will tend to float to the top of the liquid in the trap, where it is retained in baffled areas, and suspended solids in the wastewater will settle out to the bottom of the trap.

Conditions affecting the efficiency of a grease trap, other than tank geometry and baffling, include wastewater temperature and strength, and types of solvents, detergents and chemical cleaners contained in the wastewater discharged to the trap, and the ambient soil temperature.

These traps are effective in accomplishing a considerable reduction of FOG derived from animal fats if they are properly designed and maintained. However, they can be much less effective in removal of vegetable oils, which are more easily emulsified than animal FOG. The temperature of the wastewater also plays an important part in the ability of FOG to become emulsified, with high temperatures aiding in the emulsification process. Emulsified FOG will escape from the grease trap, may pass through the septic tank and reach the SWAS where, as it cools and congeals, it can cause severe clogging of the infiltrative surfaces.

Important design considerations for exterior grease traps are similar to those previously given for septic tanks.

The volume and surface areas of the trap(s) are important design considerations. The volume must be sufficient to provide adequate detention time of the FOG laden wastewater in the trap to effect the desired separation. A minimum of 24 hours of liquid detention time at the peak rate of discharge into the trap should be provided, and more is desirable. Additional volume should be provided for storage of the FOG that floats on the wastewater and any solids that may collect at the bottom of the trap(s). The storage volume will depend on the frequency of removal of the solidified FOG and the heavier solids that have settled to the bottom of the trap. A minimum storage volume equal to at least 33% of the design liquid detention time should be provided. Thus, the gross working capacity of the grease trap(s), based on inside horizontal dimensions of the trap(s) and the distance from bottom of the trap(s) to the outlet invert(s) should be equal to, or greater than, 133% of the volume necessary to provide the 24-hour liquid detention time.

The surface area in contact with the soil is an important factor in reducing the temperature of the liquid in the trap by heat exchange with the cooler surrounding soil. An important goal in design of a grease trap should be to reduce the temperature of the wastewater in the grease trap to 24°C (75°·F) or less. Mean soil temperatures in Connecticut, at the depths in which underground grease traps are normally installed, range from 9° C to 11°C (48°F to 52°F), while the temperature of FOG laden wastewater may range from 49°C (120°F) to 60 °C (140°F) or more. Thus, there is a significant temperature gradient available to conduct the heat in the wastewater out of the trap. The greater the contact area between the trap and the soil, the more heat transfer will occur.

<u>At least</u> two grease traps, arranged in series flow, should be provided. This will promote better grease separation than a single tank of equivalent liquid capacity because the heat transfer capability, and thus the cooling capacity, will be greater than that of a single trap. Vent piping should be installed between the two tanks to permit venting of the trap to the atmosphere via the building plumbing stack.

For example, a 2000-gal.exterior underground grease trap available in Connecticut might consist of a tank having a length of 12 ft, a width of 5.5 ft and a height of 5.7 ft. A similar 1000-gal. trap might have a length of 8.5 ft, a width of 4.6 ft, and a height of 5.3 ft. The soil contact area of the single 2000-gallon trap would be about 265-sq. ft., exclusive of the top area. If two, 1000-gallon traps were used, in lieu of the single 2000 gal. trap, the total contact area would be about 356-sq. ft. exclusive of the top area. Thus, the contact area of two, 1000-gal. grease traps would be about 34% greater than the single 2000 gal. trap. (If the top areas are included, the contact area of the single 2000 gal. trap would be 331 sq. ft. while the combined contact areas of the two 1000-gal. traps would be 434 sq. ft. and thus the two 1000-gal. tanks would have about 31% greater contact area.) Also note that the sum of the horizontal cross-sectional areas of the two tanks is 18% greater than in the single trap. Larger horizontal surface areas contribute to more efficient floatation of the FOG and settlement of the heavier suspended solids.

The inlet, outlet, and internal baffling of a grease trap are also important design considerations. The inlet should consist of a baffle tee or similar flow control device that extends no closer than 4 inches to the inside top of the trap and to within 12 inches of the bottom of the trap. The outlet of the grease trap should be fitted with a filtering unit designed specifically for use in grease traps and shall extend to within 12 inches of the inside bottom of the trap. A difference in elevation between the inlet and outlet inverts of 3 inches to 6 inches should be provided to ensure flow through the grease trap without backing up of waste in the inlet sewer. (As grease begins to accumulate, the top of the grease layer will begin rising above the normal water level at a distance of approximately one inch for each 9 inches of grease thickness.)

Material durability is important because of the harsh conditions that exist within the trap, including high temperatures and corrosive conditions. Precast, reinforced concrete tanks have been found most suitable for use as exterior underground grease traps.

Grease Traps should only be connected to fixtures that discharge FOG. These may include:

- Pot sinks;
- Pre-rinse sinks;
- Any sink into which fats, oils or grease are likely to be introduced;
- Soup kettles or similar devices;
- WOK stations;
- Sinks into which kettles may be drained;
- Automatic hood wash units
- Any other fixtures or drains that are likely to allow fats, oils and grease to be discharged, and
- Dishwashers without pre-rinse sinks.

The design engineer should carefully evaluate the use of cleaners and sanitizers within the establishment served because of their potential adverse effects on the biological processes used for wastewater treatment.

Easy access to the interior of the trap for removal of accumulated FOG is important, as it will contribute to proper maintenance procedures. Risers should be provided, extending from the top of the tank to the ground surface and should conform to the requirements of the Connecticut Public Health Code. To prevent escape of odors between cleaning, the frames and covers for grease traps should be of the gas-and-watertight-type. The trap should be vented as required by the applicable plumbing codes.

D. FLOOR DRAINS

Emulsifiers, cleaning products that keep fats, oils and grease in suspension, may allow the fats, oils and grease to short-circuit the grease trap and their use should be strongly discouraged. It is also very important that room floor area drainage systems do not discharge to the grease trap. Chemical solutions used for cleaning floors and sanitizing other food processing and serving areas often have an adverse (toxic) effect on the biological activities in grease traps, septic tanks and enhanced pretreatment facilities. Therefore, room floor area drains should not be connected to wastewater plumbing systems that discharge to underground grease traps, septic tanks or other pretreatment facilities. Chemicals used for sanitizing other surfaces (e.g.: countertops, etc.) should be used with caution and kept from the wastewater plumbing systems whenever possible. Where special floor drains are required for receiving discharge of fats, oils and grease from soup kettles or similar devices, the areas in which the special drains are located shall be raised or curbed to prevent chemical solutions used for cleaning the room floor areas from discharging to the special floor drains.

Room floor area drains shall be connected to a separate floor drainage piping system that discharges to a separate holding tank or separate treatment facilities. Where floor drainage holding tanks are utilized, they should be equipped with high water level alarms. The contents of floor drainage holding tanks shall be removed as required to prevent overflow of the tank, and disposed of in conformance with the rules and regulations of the Department and the Connecticut Department of Public Health. Where on-site treatment is provided to eliminate the toxic effects of the contents of the holding tank, provisions can be made for bleeding the treated contents from floor drain holding tanks in a controlled fashion into the wastewater pretreatment facilities. This cannot be done without prior approval of the Department.

E. GREASE TRAP AND SEPTIC TANK MONITORING SYSTEMS.

Equipment is now available that permit continuous monitoring (both locally and remote) of the liquid, scum and sludge levels in grease traps and septic tanks and provide warnings when it is time for pumping of trap or tank contents or replacement of effluent filters. At least one type provides for remote monitoring via the Internet to determine the reason for the alarms received or for periodic monitoring. Such systems provide advanced notice of impending problems and incorporation of such equipment into the grease trap and septic tank facilities serving food processing and serving establishments and are strongly recommended for all new and retrofitted on-site systems serving such establishments.

F. FLOW EQUALIZATION

Where peak daily flows occur sporadically, such as on weekends, flow equalization facilities should be strongly considered. This will reduce the loading on downstream treatment facilities and on the SWAS, resulting in a cost saving normally significantly greater than the cost of the equalization tank and associated facilities. Consideration should be given to equalization of the daily flow during peak days as well as on the basis of average daily vs. peak daily flows. Additional guidance on flow equalization facilities is given in other sections of this document.

TABLE OF CONTENTS

TABLE OF CONTENTS

FIGURES

TABLE OF CONTENTS

TABLES

SECTION X SUBSURFACE WASTEWATER ABSORPTION SYSTEM DESIGN

A. Introduction

This section provides guidance for design of subsurface wastewater absorption systems (SWAS) under various conditions that control such designs, including:

- soil characteristics,
- ground water conditions,
- wastewater flows and characteristics,
- long term acceptance rates,
- effective infiltrative surface areas,
- linear loading rates,
- vertical separating distance to the seasonal high ground water table,
- travel times from the SWAS to a point of concern,
- flow distribution,
- systems in natural soils, and,
- systems constructed in fill materials.

B. Vertical and Horizontal Separating Distances

1. Introduction

The U.S. EPA indicates that over one-half of the waterborne disease outbreaks in the United States are due to the consumption of contaminated ground water. While some of these outbreaks are caused by chemical contamination, the majority are caused by consumption of groundwater that has been contaminated due to the presence of bacteria and viruses in domestic wastewater that has been discharged onto or into the soil.

In particular, in recent times the U.S. EPA and public health agencies have become concerned with viruses. Viruses are of major concern because of their ability to survive for long periods of time in the subsurface and still remain infectious, and the very small number (as little as one virulent particle in some cases) are thought to cause disease. While there are some bacteria and parasites that can cause infection if ingested in small numbers, of greatest concern are the viruses that may find their way into the ground water.

2. Goals for removal/inactivation of Pathogens

Protozoa and helminths are occasionally found in septic tank effluent but are not usually found in groundwater beneath a SWAS. Because of their relatively large size, pathogens such as helminths (parasitic worms, such as roundworms and tapeworms) and protozoa (*Cryptosporidium parvum* and *Giardia lamblia*, and their cysts or oocysts) are generally removed in the biomat that forms at the soil interface of the SWAS and in the underlying unsaturated soils before reaching the water table, although this might not be the case for very coarse soils.

However, bacteria and viruses are much smaller and, when discharged to a SWAS, can move into ground and surface waters, initiate significant health problems, and promote outbreaks of waterborne disease (VA Division of Health-1990). While pathogenic bacteria are of public health concern, studies have shown that viruses travel further and can exist in a viable state for a much longer time than pathogenic bacteria. Therefore, viruses are of significant concern with respect to public health considerations. Where adequate removal/inactivation of viruses is obtained, it is probable that adequate removal of other pathogenic microorganisms has also occurred.

The Department had a detailed review and study of the literature conducted on the fate and transport of pathogens in the subsurface (Jacobson-2002). The results of that study indicated that it is reasonable to establish a goal of at least a 5 $\log_{10}$ (99.999%) removal/inactivation of viruses from domestic wastewater discharged to an OWRS before the commingled wastewater/ground water reaches a sensitive receptor, and that a greater removal/inactivation is preferable.

3. Vertical Separating Distance

Recent detailed studies in Florida, Colorado and Massachusetts have confirmed earlier studies that indicated a three Log_{10} (99.9%) removal/inactivation of viruses can be obtained when domestic wastewater has:

a.) been pretreated in a septic tank and discharged to a properly designed SWAS,

b.) percolated through the biomat that forms at the SWAS-soil interface and,

c.) has moved slowly down through at least three feet of suitable aerobic, unsaturated soil.

Under design flow conditions, additional vertical separating distance may be necessary to provide adequate hydraulic reserve capacity. While the examples contained in this section do not address reserve hydraulic capacity, adequate reserve capacity shall be provided in the system design. This should be discussed with Department staff.

4. Horizontal Separating Distance

While the most significant renovation of septic tank effluent occurs at the biomat that develops at the soil interface with the SWAS and in the unsaturated soil beneath the SWAS, renovation of the percolate from the SWAS continues after it reaches the saturated zone. The effectiveness of renovation in the saturated zone depends on factors such as the type and strain of virus, physical, chemical and biological characteristics of the virus, the physical and chemical characteristics of the soil through which the percolate flows, the temperature of the ground water, and the natural processes that tend to remove or degrade viruses in the subsurface. These natural processes include sorption, ion-exchange, dispersion, and microbial degradation.

Numerous studies have been conducted in an attempt to quantify the rate of virus removal in the ground water. The only factor that has consistently been shown to demonstrate a statistically significant correlation with the decay rate of viruses under saturated flow conditions has been the ground water temperature. Yates et al. (1987) determined from 172 virus experiments conducted at temperatures ranging from 4° to 32°C that the virus inactivation rate could be expressed by the following equation:

$$\text{Inactivation Rate, } Log_{10}\ day^{-1} = (0.018 \times T)-0.0144,$$

where T = ground water temperature, °C. The mean ground water temperature in Connecticut, in the zone affected by seasonal fluctuations, can be assumed to be at least 10°C, except in the extreme northeastern and northwestern corners of the state. Inserting that value in the equation above results in an inactivation rate of 0.036 $\log_{10}$ day^{-1}. This indicates that, in Connecticut, viruses can survive for long periods of time in the ground water. If the goal for virus removal/inactivation is selected to be five (5) $\log_{10}$ for sensitive receptors, and a three (3) $\log_{10}$ removal/inactivation is anticipated before the wastewater reaches the ground water, an additional two (2) $\log_{10}$ inactivation would be required as the viruses travel with the ground water. Based on an inactivation rate of 0.036 $\log_{10}$ per day, a travel time of 56 days is indicated between a SWAS and existing and potential sensitive receptors such as:

a. the outer limit of the cone of depression of a public (community) drinking water supply well,
b. a surface water body used, or intended to be used, as a source of public (community) drinking water supply,
c. a private drinking water supply well serving an individual residence.
d. an impoundment used for aquaculture.

The minimum required travel time to all other points of concern should be not less than 21 days, and a greater travel time is preferable.

It should be noted that some investigators have found that passage of raw wastewater through a septic tank resulted in a reduction of virus concentration in the tank effluent. For example, Higgins et al. (2000) found a 74% (< 1 $\log_{10}$) reduction. On the other hand, other investigators have found little or no such reduction. Thus, while a septic tank may effect some reduction in virus concentration, the amount of reduction is in question.

Therefore, any reduction in virus concentration effected by a septic tank is considered to be a safety factor and any such reduction should not be credited as part of the five (5) $\log_{10}$ reduction goal.

C. Long Term Acceptance Rate (LTAR)

1. General

The Department's criteria for hydraulic design of a subsurface wastewater absorption system (SWAS) are based on consideration of both the hydraulic capacity of the soil in which the system is located, and the long term acceptance rate (LTAR) of pretreated wastewater by the biocrust (biomat) that develops at the soil/SWAS interface (infiltrative surfaces). The determination of the soil hydraulic capacity has been addressed in Section VI- Hydraulic Capacity Analysis. This sub-section addresses the selection of the LTAR of the SWAS infiltrative surfaces.

As indicated in Section II, the thickness and susceptibility of the biocrust to clogging is related to the dissolved and suspended organic matter remaining in the pretreated wastewater (the "organic loading rate"). Excessive organic loading rates will result in conditions leading to a thicker biological/zoogleal layer that severely reduces the rate of flow into the unsaturated soil zone and causes anaerobic conditions to persist.

The LTAR may be defined as the infiltrative surface loading rate at which a SWAS will continuously accept effluent for a long period of time, and is dependent upon the soil characteristics, the biomat, and the wastewater characteristics (Anderson, et al.-1991). Healy and Laak (1974) determined the following relationship between the LTAR of a soil and the soil hydraulic conductivity:

$$LTAR = 5K - [1.2/(Log_{10}K)].$$

In this formula LTAR is in units of gpd/ft^2 and K, saturated hydraulic conductivity, is in units of ft/minute.

Figure LTAR-1 presents this expression in graphical format. For effluent from household septic tanks, the maximum stable LTAR value allowed by the CTDEP is 0.80 gallons per day per square foot of effective leaching area. This corresponds to a K value of ~28 ft/day (0.0197 ft/min. or 0.010 cm/sec).

Siegrist (1987) stated that the rate of discharge from a SWAS to the underlying unsaturated zone should not exceed 3% to 5% of the saturated hydraulic conductivity. He stated that such low discharge rates (hydraulic loading rates) are required in order to maintain adequate soil aeration and the low soil moisture content in the unsaturated zone that will allow intimate contact between the percolate from the SWAS and the soil particles. These conditions are required for removal/attenuation of pathogens and other contaminants in the percolate. The LTAR rates obtained from Figure LTAR-1 satisfy this requirement.

Laak (1970) hypothesized that the service life of a SWAS is related to the sum of the BOD_5 and TSS and that increasing the pretreatment of domestic wastewater prior to discharge to a SWAS would increase the service life of the SWAS. Based on the results of his studies at the University of Toronto (Laak-1966), he suggested an expression for the affect of BOD_5 and TSS in septic tank effluent on the development of the clogging mat at the SWAS-soil interface (Laak-1977). This expression could be used to calculate the increase in infiltrative surface area required for strong wastewater or the decrease in such area where reliable enhanced pretreatment is provided.

An "adjustment factor", based on the Laak expression, can be used to determine the leaching surface application rate to be used for high-strength (or low strength) wastewater. This factor is derived from the mathematical expression shown below (Laak-1977), which relates the five-day Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS) concentrations in such wastewaters, to the average concentrations of BOD_5 and TSS found in the effluent of septic tanks receiving household wastewater:

$$\text{LTAR Adjustment Factor} = [250/(BOD_5 + TSS)]^{1/3}$$

In the preceding mathematical expression, the BOD_5 and TSS are expressed in milligrams per liter, and represent the values of these constituents in the pretreated wastewater discharged to the SWAS.

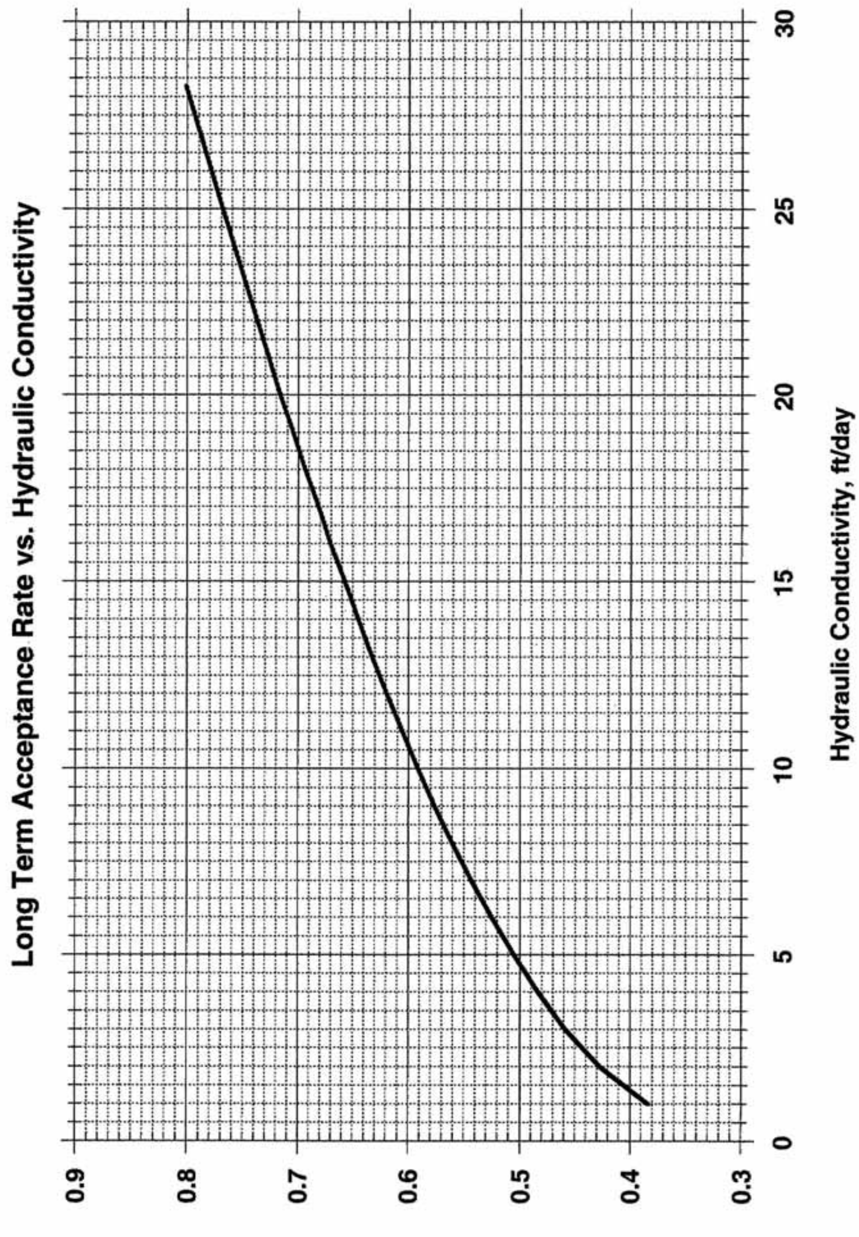

Figure LTAR-1

Thus, for wastewaters having BOD_5 and TSS values higher (stronger) than normal domestic wastewater, the LTAR value is decreased, and for wastewaters having lower (weaker) values, the LTAR value is increased. Where the septic tank effluent does not receive additional treatment prior to discharge to a SWAS, the maximum LTAR recommended = 0.8 gpd/sf of effective infiltrative area (ELA). Where additional treatment is provided, the maximum LTAR value recommended is 1.2 gallons per day per square foot of effective bottom area only. This limiting value is used to ensure the unsaturated soil conditions necessary in the soil beneath the SWAS for effective removal/inactivation of bacteria and viruses.

2. Results of Additional Research

Considerable research has been conducted since the current method for determining LTAR was developed. (Anderson, et al-1981; Otis, R.J.-1984; Siegrist, et al-1984a&b; Siegrist-1987a & b; Tyler and Converse-1989; Jensen and Siegrist-1991; Tyler and Converse-1994; Loudon, et al-1998; Loudon-1999; Matejcek, et al-2000; Erlsten and Bloomquist-2001; Tyler-2001). Of considerable interest with respect to long term acceptance rates for wastewater strengths considerably higher than household wastewater are the very recent studies by Matejcek, et al-2000 and Erlsten and Bloomquist-2001.

Matejcek et al (2000) conducted a thorough and well-documented study on long term acceptance rates for restaurant wastewater. Wastewater physical and chemical characteristics were determined for 133 samples of septic tank effluent from fifteen randomly chosen restaurants in Florida.

Failure occurred primarily in the lysimeters with two feet of unsaturated soil that were dosed with medium and high strength wastewater. Twenty-four lysimeters failed during the 112-day study with 20 failures occurring between 20 and 47 days. No failures were recorded in lysimeters dosed with low strength wastewaters, which received a daily mass loading (BOD_5 and TSS) of 0.0015 $lb/ft^2/day$. In addition, the cumulative mass loaded on the low strength columns exceeded the cumulative mass loading of the failed columns dosed with medium strength wastewater.

Conclusions reached by Matejcek et al. (ibid.) with respect to long term acceptance rates for restaurant wastewater were as follows:

1. Hydraulic loading alone does not cause drainfields to fail. Effluent concentration and hydraulic loading both contribute to clogging and formation of biomat, resulting in failure.

2. Fine sand soil columns receiving less than 0.0015 $lb/ft^2/day$ of contaminant mass (sum of BOD and TSS) did not fail. Similar columns receiving 0.0043 $lb/ft^2/day$ did fail. Therefore, there is a possible threshold at which drainfields fail due to daily mass loading. In this case, it appears to be between 0.0015 and 0.0043 $lb/ft^2/day$ for the fine sand soil.

A similar case can be made for all four soil types. Below the thresholds, drainfields appear to be able to adequately treat the daily load and are poised for the next application with no apparent permanent failure.

Recommendations made by Matejcek et al. (ibid.) with respect to long term acceptance rates included:

1. Limits should be established for restaurant effluent with concentrations to be in the low wastewater strength category (similar concentrations to those of wastes from domestic systems).

2. Drainfield sizing should include mass loading rates and hydraulic loading rates based on soil properties. Mass loading rates should not exceed 0.0015 lb/ft^2/day, but this value may need to be reduced based on soil properties.

However, Erlsten and Bloomquist (2001) reported on subsequent phases of the University of Florida's Onsite Sewage Treatment and Disposal Systems and Long Term Acceptance Rate study. In phase 2, the mass loading threshold was shown to lie between 0.0015 and 0.0024 lb/ft^2/day of combined $CBOD_5$ (carbonaceous BOD_5) and TSS loading. The purpose of the phase 3 study was to further refine the apparent threshold above which lysimeter failure occurred consistently. The results obtained from the phase 3 study confirmed the upper limit established in the phase 2 study.

3. Calculating LTAR

The data on which Healy and Laak based their LTAR expression was obtained from residential sites discharging to stone filled trenches and were adjusted to a one foot ponding depth. If the infiltrative surface area hydraulic loading rates determined from the Healy and Laak LTAR expression are to be used for design of large scale on-site systems receiving a higher organic strength wastewater, the organic loading rates should be adjusted to that of household septic tank effluent. If it is assumed that the "strength" of household septic tank effluent (concentrations of BOD_5 + TSS) = 250 mg/L, the equivalent "strength" loading, at 1 gpd/ft^2, = 91 lbs./acre/day or 0.0021 lbs/ft^2/day. At the maximum allowable LTAR (hydraulic loading rate) of 0.8 gpd/ft^2, this equivalent loading rate becomes 72.6 lbs/acre/day, or 0.0017 lbs/ft^2/day. This falls within the mass loading threshold range of 0.0015-0.0024 lbs/ft^2/day found by Erlsten and Bloomquist (2001). The upper end of that range (0.0024 lbs/ft^2/day) would be representative of a wastewater strength of about 360 mg/L. The mid-point of that range is 0.0020 lbs/ft^2/day.

The 250 mg/L value for the sum of household septic tank BOD_5 + TSS came from Laak (1977) and apparently was based on household wastewater characteristics determined several decades ago. Additional data that has become available since that time appears to indicate that this value may be a little low. This may be partially due to the reduced flow fixtures that have been on the market for almost two decades, including both the 3.5 gallon per flush toilet and the newer 1.6 gallon per flush toilet, plus reduced flow lavatory and shower head fixtures. This reduction in flow can be expected to result in a corresponding increase in the septic tank effluent pollutant concentrations. However, a decrease in flow should show an increase in septic tank efficiency, and thus the effects of decreased flow may cancel each other.

A method has been developed for adjusting the LTAR by using the Laak formula with the values obtained therefrom truncated when they exceed a mass loading of 0.0020 lbs./sf/day. A graph entitled "Adjustment of LTAR based on Wastewater Strength" is shown in Figure LTAR-2.

The adjusted LTAR determined from Figure LTAR-2 is then further adjusted on the basis of the concentration of TN anticipated to be found in the pretreated wastewater discharged to the SWAS. This will account for the increased oxygen demand (nitrogenous oxygen demand) exerted by the bacteria that oxidize the TN to nitrates where the TN concentration exceeds the TN concentration found in household wastewater.

Thus, where the TN concentration in the pretreated wastewater is greater than 56 mg/L, the adjusted LTAR based on wastewater strength is multiplied by the following factor:

$$\text{TN adjustment factor} = \frac{\text{56 mg/L[typical septic tank effluent]}}{\text{Pretreated Wastewater TN concentration, mg/L.}}$$

[The 56 mg/L is based on an upper limit of TN for raw residential wastewater of 70 mg/L and a removal rate of 20% in the septic tank. (70 mg/L *(1-0.20) = 56 mg/L)]

The procedures discussed above provide a means for determining the infiltrative surface loading rates based both on hydraulic and organic loading rates.

ADJUSTMENT OF LTAR BASED ON WASTEWATER STRENGTH

[Wastewater Strength = Σ (BOD_5 and TSS), mg/L]

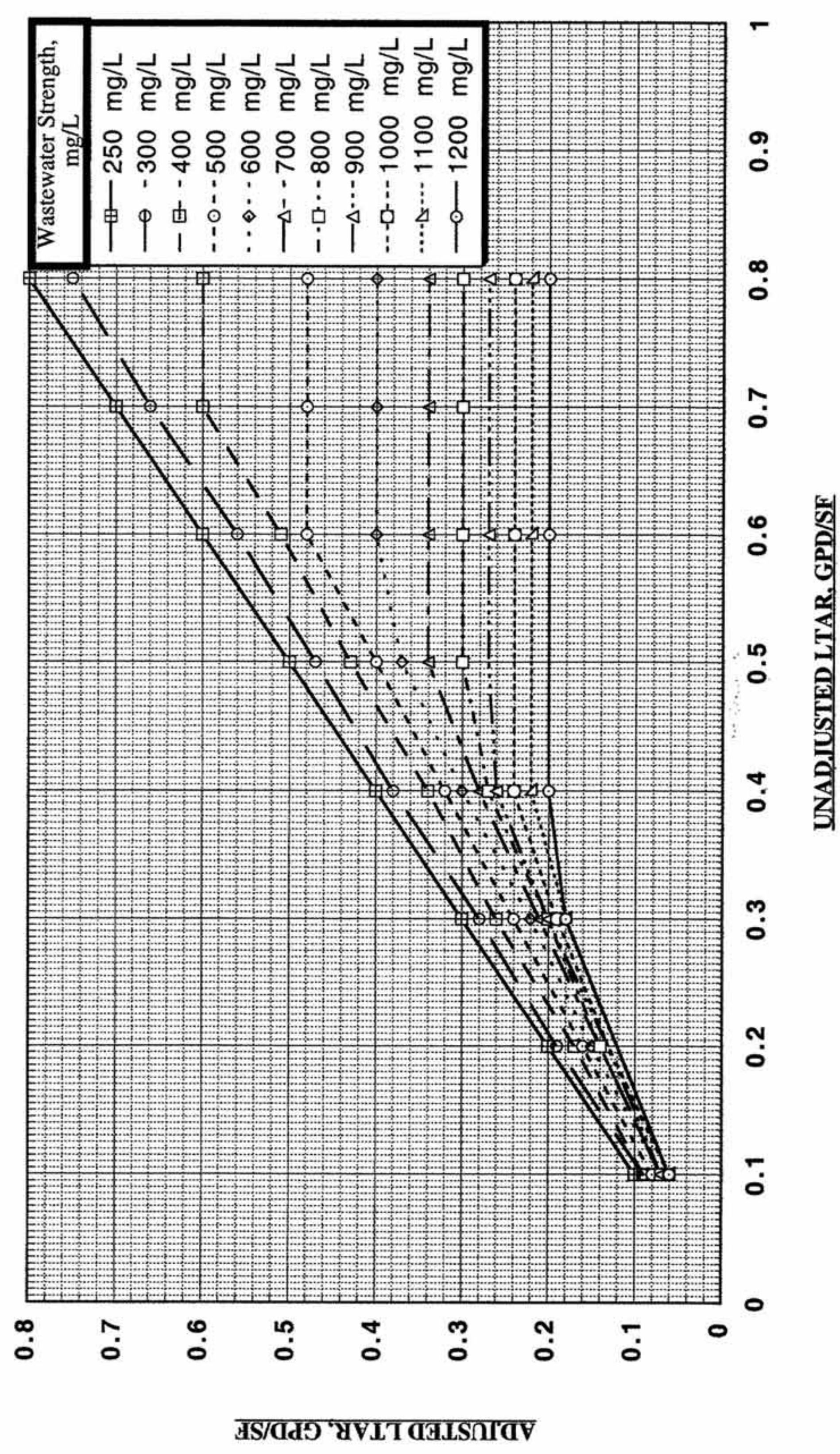

FIGURE LTAR-2

D. Effective Leaching Surface Area

1. General

The effective leaching (infiltrative) surface area (ELA) of a SWAS is the interface area between the soil and the facilities used for applying the pretreated wastewater to the soil. The wastewater application facilities, commonly referred to as leaching systems, may consist of:

1) flow distribution piping embedded in a coarse aggregate (commonly referred to as stone, broken stone or "gravel") filled trench,
2) a row, or rows, of precast concrete gallery units or plastic chamber units with open bottom and coarse aggregate placed along the sides of the units and flow distribution piping installed within the units,
3) flow distribution piping embedded in a coarse aggregate leaching bed, but only where enhanced pretreatment is provided, or
4) other wastewater leaching units that are approved by the Department.

As previously discussed under subsection C, the Healy and Laak expression for LTAR was based on a stone-filled trench ponded to a depth of one foot. Thus, the unit value (per linear ft. of trench) for ELA was the trench bottom contact area plus the sidewall contact area (one ft of height on each side of the trench).

Several investigators have determined that, where gallery or chamber units are installed without coarse aggregate placed between the units and the soil interface (so called "gravel-less leaching systems"), infiltration of the pretreated wastewater into the soil is considerably more efficient. They attribute this increased efficiency to the lack of the "masking (shadowing) effect" of the broken stone or natural gravel. The masking effect on the infiltrative surface area by stone or gravel has been discussed for many years (Bouma and Magdoff -1974; Siegrist - 1987; Tyler, Converse and Milter-1991, Siegrist and Van Cuyk. 2001.). Recent studies have indicated that gravel-less leaching systems can be loaded at rates equal to 1.7 to 2.0 times the loading rate of systems using gravel. (Hoxie and Frick-1984; Tyler, Converse and Milter, ibid; Siegrist and Van Cuyk, ibid).

On the other hand, while White and West (2003) agreed that gravel-less systems are more efficient in permitting the infiltration of wastewater through the biomat, they disagreed with the masking concept. The results of their studies indicated that it is the fines associated with the "gravel" aggregate that eventually slough off the aggregate and accumulate at the infiltrative surface that cause a reduction in leaching capacity. Their premise was later refuted by Siegrist, et al. (2004).

Amerson and others (1991) stated "the presence of fines is the predominant factor in infiltration rate reductions. One to four percent of gravel fines by weight resulted in a significant reduction in infiltration rates by 35 to 65 percent."

While most state regulatory agencies require that the "gravel" be washed prior to use, in practice washed gravel is not always used. Even after washing, gravel used in constructing a SWAS still contains a small amount of fines (typically 3 to 5 percent), ranging in size from 2 mm to less than 20 μm. Over a short period of time, the fines wash from the gravel and settle at the bottom of the trench. Fines are a significant problem as they significantly reduce flow rates (White and West-ibid.).

Regardless of whether it is the lack of fines, the absence of the "masking effect", or both, that results in the observed increase in infiltrative efficiency of gravel-less systems, the increase appears to have been validated by several detailed studies. Therefore, the Department has determined that "gravel-less" systems can be allowed a higher ELA than that allowed for a leaching system where gravel is used and has adopted a factor of 1.5 for computing the unit value for ELA for gravel-less leaching systems.

2. Calculation of Effective Leaching Area (ELA)

The following formula should be used to calculate the unit value for ELA/lf. The formula takes into account both masked and unmasked infiltrative surface areas, the hydraulic head on the infiltrative surfaces and an allowance for reserve storage area.

ELA/lf = [1.5 x inside clear (unmasked) bottom area of leaching unit + 1.0 x effective stone-masked bottom area] + [1.0 x effective height of stone-masked sidewall areas of leaching units*]

Where:

Leaching Unit = stone-filled trench, concrete gallery unit, plastic chamber unit, or other type of unit approved by the Department

Effective Sidewall Height = from Leaching Unit bottom to wastewater inlet invert, in ft, but not more than one foot (30 cm).

* For gallery and plastic chamber units, inclusion of sidewall height in calculating the ELA will only be permitted if the wastewater can flow into the sidewall areas through openings in the sidewalls that are less than one foot from the bottom of the unit.

Stone-masked Sidewall ELA, sf/lf = 2 x Effective Sidewall Height, in ft.

Stone-masked Bottom ELA, sf/lf = Bottom contact area of stone placed beneath or on sides of Leaching Unit (1 ft. maximum either side of Leaching Unit), in ft. (Maximum allowable width of Leaching Unit plus sidewall stone = 6 ft)

Where a stone-bottomed leaching bed is used in a Lateral Sand Filter, the entire bed bottom area should be considered stone masked.

Unmasked Bottom Area, sf/lf = average inside clear bottom area of Leaching Unit/lf.

[It is acknowledged that additional sidewall height will provide additional ELA when the depth of ponding above the bottom of a Leaching Unit exceeds one ft (30 cm); however this is considered to be a safety factor and is not used in computing the unit value for ELA.]

E. Linear Loading Rates

1. General

As discussed in Section VI, the rate of flow of ground water is proportional to the hydraulic conductivity of the soil, the hydraulic gradient, and the available effective area of flow perpendicular to the hydraulic gradient. The available effective area of flow is directly proportional to the available depth of soil through which the percolate from the SWAS will flow. Thus it is necessary to orient the SWAS perpendicular to the hydraulic gradient and distribute the flow to the SWAS in such a manner that it will be contained within the available depth of soil.

2. Sloping Sites

On a sloping site, it is assumed that the percolate from the SWAS flows downgradient in the soil zone above the seasonal high ground water table (SHWT) with a hydraulic gradient equal to the local natural hydraulic gradient of the water table. This assumption is reasonable as long as the natural gradient is not influenced by an induced gradient (e.g.: from a well or underdrain). It is possible, and in some instances probable, that lateral flow will also occur in an unsaturated soil zone. However, the procedure adopted by the Department for determining linear loading rates is applied only to the soil zone above the seasonal high ground water table (above the phreatic surface) that will become saturated due to the introduction of the SWAS percolate.

Where a SWAS is installed completely in natural soils, the amount by which the water table can be raised (mounded) on a sloping site by the introduction of the SWAS percolate is limited to the depth D of soil above the SHWT - d_u (where d_u = depth of cover over leaching units + depth of leaching units + required vertical separating distance between the bottom of the leaching units and the seasonally high ground water table).

The amount of SWAS percolate that can be accommodated per linear foot of SWAS (in the direction perpendicular to the hydraulic gradient) can be calculated from Darcy's law in the following manner.

$q_{lf} = K_{sat} \times i \times A$, where q_{lf} = flow per linear ft. of SWAS measured perpendicular to the hydraulic gradient, in ft^3/d, i = the local hydraulic gradient of the water table, in ft/ft, and A = the soil area perpendicular to the hydraulic gradient through which the percolate will flow, in ft^2. The soil area per linear foot of SWAS in the direction perpendicular to the hydraulic gradient = 1 linear ft x ($D-d_u$), the depth of soil in ft available to transmit the percolate down gradient from the SWAS.

$$q_{lf} = K_{sat} \times i \times 1 \text{ linear ft} \times (D-d_u), \ = \text{cubic ft/day/linear ft.}$$ [1]

The required lateral extent of the SWAS, in the direction perpendicular to the local hydraulic gradient = Q_t / q_{lf} where Q_t is the design daily flow for which the SWAS is being designed.

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[1] If it is desired to have q_{lf} expressed in gallons/day/linear ft., the result of this calculation should be multiplied by a factor of 7.48 gal/cu. ft.)

The total effective leaching surface area required per linear foot of SWAS = q_{lf}/LTAR (with LTAR adjusted as may be required by wastewater strength). The number of rows of leaching units required for the SWAS then depends upon the effective leaching surface area of the selected leaching units or trenches.

The procedure for determining the required lateral extent of a SWAS is illustrated in the following example, using U.S. units. Refer to Figure LL-1.

<u>EXAMPLE</u>:

A SWAS needs to be designed for a facility that will generate a maximum day flow (Q) of 6000 gpd. The available site has the following characteristics:

The width of the site perpendicular to the local hydraulic gradient = 280 ft.
The depth to the seasonal high ground water table (SHWT) = 10 ft.
The local natural hydraulic gradient = 0.09 ft/ft.

It is proposed to use leaching galleries having a height of 1.5 ft and to provide 1 ft. of cover over the top of the leaching galleries.

In this example, the required vertical height of unsaturated soil below the bottom of the leaching galleries and the mounded seasonal high ground water table = 3 ft.

d_u = 1.0 ft. + 1.5 ft + 3.0 ft. = 5.5 ft

D-d_u = 10 ft - 5.5 ft = 4.5 ft. (Note that this is the maximum allowable height of mounding above the seasonal high ground water table due to discharge of the percolate from the SWAS).

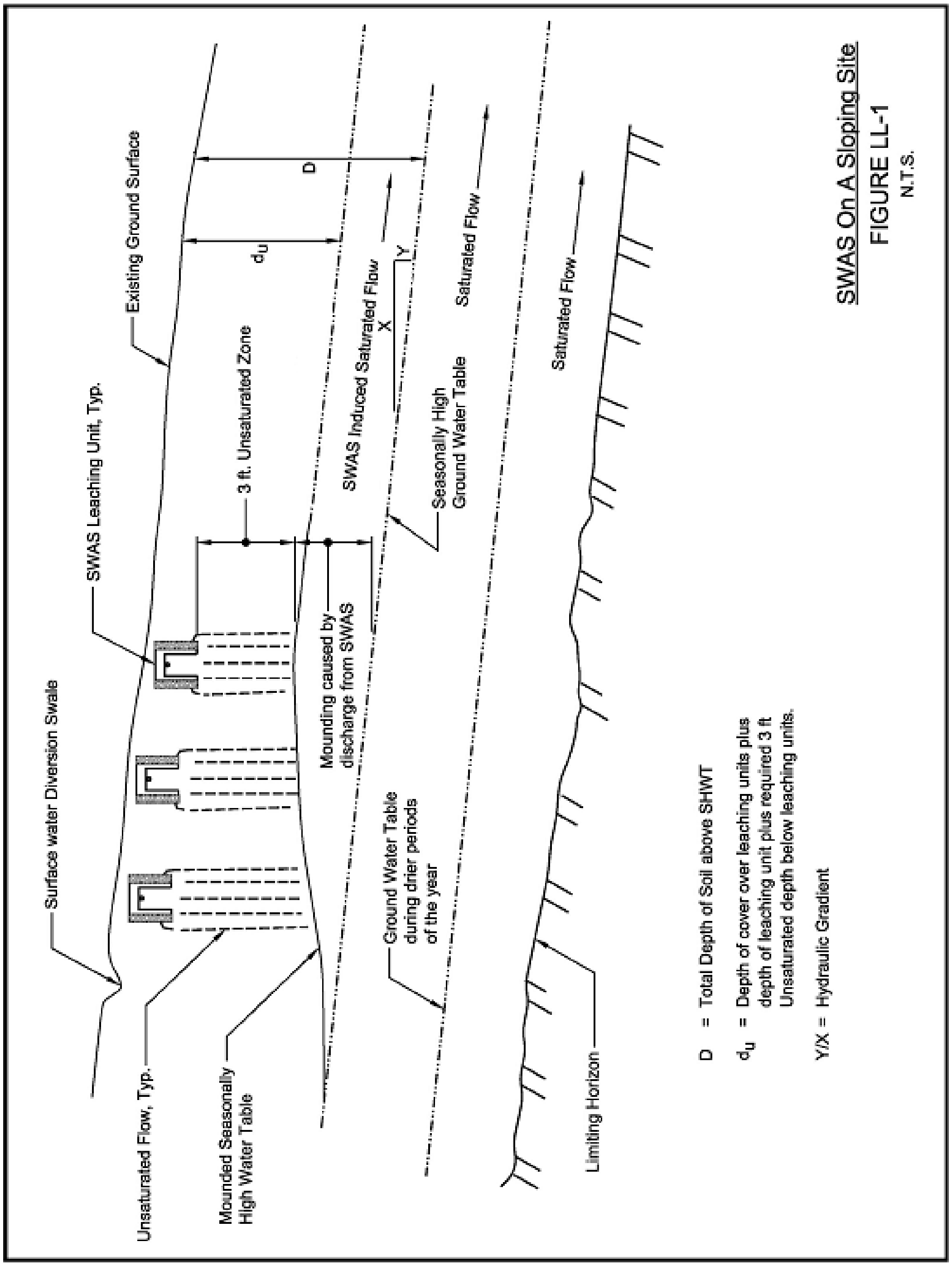
The design K_{sat} of the existing unsaturated soil, from 5.5 ft. below the ground surface to the SHWT was determined to be 8.0 ft/day.

The allowable linear loading rate, q_{lf} = 8.0 ft/day x 0.09 ft/ft x 1 lf x 4.5 ft. = 3.24 cu. ft./day/lf. x 7.48 g/cu. ft. = 24.2 gpd/lf

The required width of SWAS perpendicular to the local hydraulic gradient = Q/q_{lf} = 6000 gpd/24.2 gpd/lf = 248 lf.

Since the lot width is 280 ft, there appears to be ample room to install the SWAS, absent any local requirements for property line setbacks that may restrict the width of the SWAS. (Of course, the ability to use this site will also depend upon having sufficient distance between the SWAS and the closest point of concern to meet the travel time requirements, the phosphorus attenuation capabilities of the soils, the ability to meet the TN requirements at the closest point of concern and the ability to provide adequate hydraulic and infiltrative reserve.)

Examples where the existing natural soils do not have sufficient capability to conduct the percolate away from the SWAS are given in the sub-section on Fill Systems.



SWAS On A Sloping Site

FIGURE LL-1

N.T.S.

3. Sites With Very Low Hydraulic Gradients

On a site where the local hydraulic gradient is very low, a 3-Dimensional Hydraulic Capacity analysis is required, as discussed in Section VI.

The approach to determining ground water mounding under such conditions is different from that used where there is a significant slope to the hydraulic gradient. In the low hydraulic gradient situation, a configuration of the SWAS must be assumed and the resulting mound height calculated to determine if there will be at least 3 ft. of unsaturated soil beneath the bottom of the SWAS and the SHWT. This may involve several iterations before the final configuration of the SWAS is selected. The following example, using U.S. units, indicates how the ground water mounding under such conditions may be calculated.

EXAMPLE: (Refer to Figure LL-2.)

A SWAS needs to be designed for a facility that will generate a maximum day flow (Q) of 6000 gpd. The available site has the following characteristics:

The lot dimensions of the proposed site of the SWAS = 400 ft wide perpendicular to the hydraulic gradient and 600 ft long in the direction of the hydraulic gradient.

The depth from ground surface to the seasonal high ground water table (SHWT) = 8 ft.

The depth from the SHWT to the limiting horizon = 12 ft.

The local natural hydraulic gradient = 0.001 ft/ft. and is considered to be negligible for the purposes of this example.

The soils beneath the site consist of sands that extend from about 2 ft. below ground level down to the limiting horizon, which is bedrock. A water table exists above the bedrock at all times of the year.

It is proposed to use 2.5 ft. high by 4.0 ft wide x 8.0 ft. long precast concrete gallery units with one foot of broken stone alongside the gallery sides. The effective leaching area for these units is given as 9.25 sq. ft./lf. The tops of the galleries will be one ft. below existing grade.

The required vertical height of unsaturated soil below the bottom of the leaching galleries and the mounded seasonal high ground water table = 3 ft.

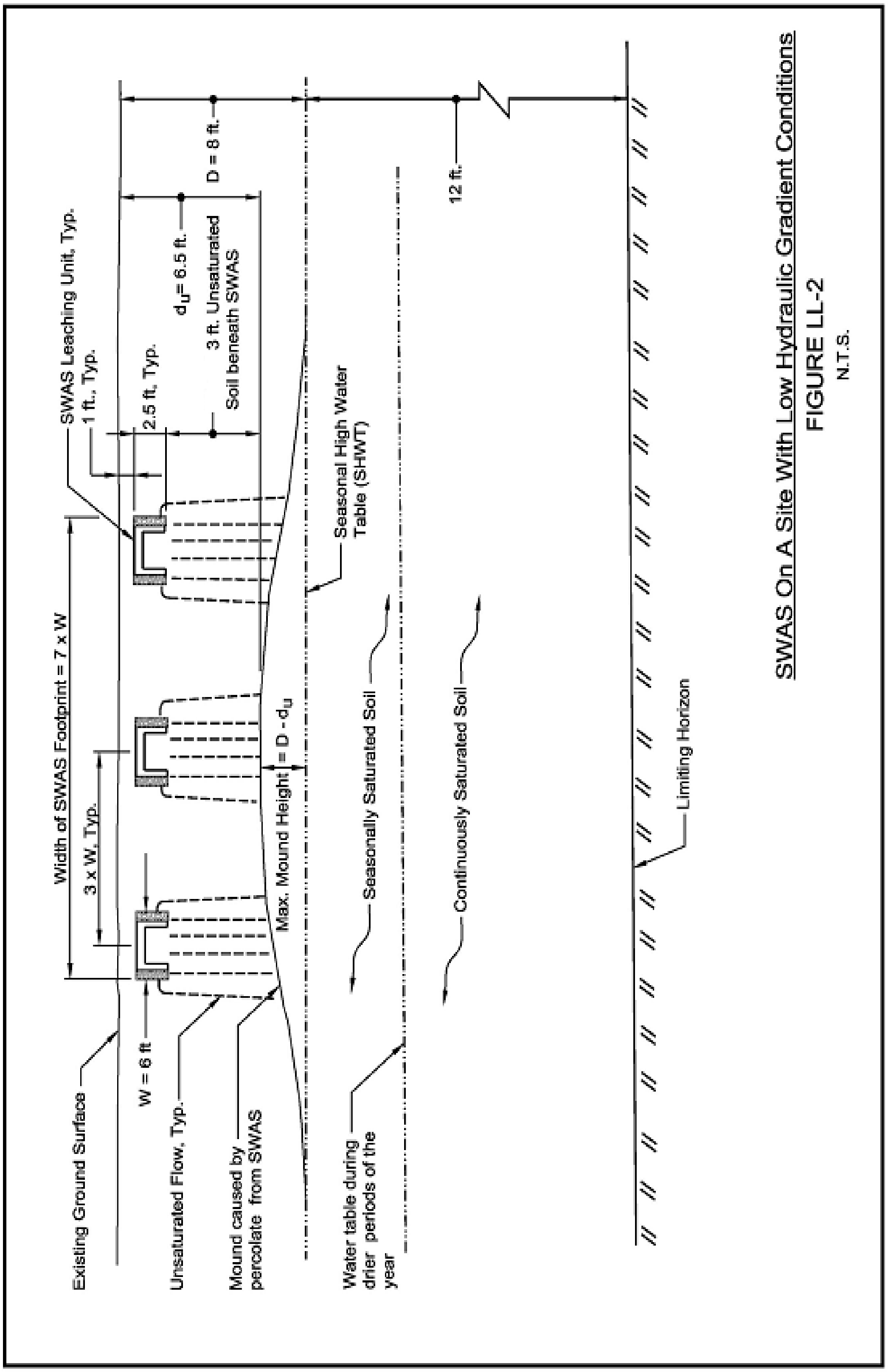
d_u = 1.0 ft. + 2.5 ft. + 3.0 ft. = 6.5 ft.

D-d_u = 8 ft (the depth from ground surface to the SHWT) - 6.5 ft = 1.5 ft. (Note that this is the maximum allowable height of mounding above the seasonal high ground water table due to discharge of the percolate from the SWAS).

The design value selected for K_{sat} of the existing sandy soil, from 3.5 ft. below the ground surface to the bedrock = 25 ft/day = 0.0174 ft./min. From Figure LTAR-1, the long-term acceptance rate = 0.77 gpd/sf of effective leaching area. The pretreated effluent is estimated to have concentrations of BOD_5 and TSS typical of effluent from a domestic septic tank, and therefore no adjustment for the LTAR is required.

The total effective leaching area required = 6,000 gpd/0.77 gpd/sf = 7792 sq. ft.

The total linear feet of leaching galleries required = 7792 sf/9.25 sf/lf = 840 lf.



SWAS On A Site With Low Hydraulic Gradient Conditions

FIGURE LL-2

N.T.S.

An initial trial layout of the SWAS assumes 3 rows of leaching galleries spaced at 18-ft c.c. The leaching galleries are fabricated in 8-ft sections. Thus, each row will consist of 35 sections, yielding a total of 105 sections with a total of 840 lf of leaching galleries.

The center to center spacing between rows of leaching galleries = 18 ft, the width of each gallery row will be 6 ft (including a one ft. width of stone along the sidewalls) and the length of each row will be 35 sections x 8 ft./section = 280 ft. Thus, the overall footprint of the SWAS will be (2 x 18 ft + 6 ft.) x 280 ft = 42 ft x 280 ft. = 11,760 sq. ft.

A computer program for an analytical model of ground water mounding beneath a ground water recharge basin ("Flow From Wells and Recharge Pits" - Ref. Section VI, Subsection H.3.) is used to assess the hydraulic capacity of the proposed site. The SWAS footprint is assumed to be equivalent to a ground water recharge basin of the same dimensions. The analytical model requires the following input:

- Transmissivity of the aquifer (saturated soil) material. (This is equivalent to the hydraulic conductivity, (K, ft/d) x the depth of the aquifer (ft).) In this case, the transmissivity is calculated to be 25 ft/day x 12 ft = 300 sq. ft./day. In this example the aquifer is assumed to be homogeneous, isotrophic, and of infinite areal extent.

- The dimensions of the equivalent recharge basin (width and length). In this case, these dimensions are 42 ft. and 280 ft. respectively.

- The hydraulic loading rate of the basin. In this case, the equivalent unit hydraulic loading on the footprint area of the basin = 6,000 gpd/(42 x 280) sq. ft. = 0.51 gpd/sf = 0.068 cu. ft/day/sq. ft., or 0.068 ft/day.

- The duration the basin will be loaded, in days. In this case, loading periods equal to 10 years and 20 years were selected.

- The incremental distance from the center of the basin along the mound profile in the X and Y directions for which the mound height will be calculated. In this case, an incremental distance of 20 ft. was selected. (Note; this information is useful for computing travel times to the closest points of concern, as the slope of the mounded water table (the hydraulic gradient) will vary with distance from the center of the SWAS.)

- The depth from ground surface to the (seasonally high) water table. In this case, 8 ft.

- This data, when entered into the analytical model indicated a mound height of 1.8 ft would develop beneath the SWAS during the first ten years of loading and a mound height of 2.0 ft would occur after 20 years of loading. This indicates the mound height will probably not significantly exceed 2.0 ft. over a very long time period.

Since the calculated maximum height of the mound is 2.0 ft, which is greater than $D\text{-}d_u$ (1.5 ft), the design is unsatisfactory with respect to the required 3 ft of vertical separating distance as the mounded water table will rise to 2.5 ft below the SWAS.

It should be noted that additional assumptions of the SWAS footprint, type of leaching unit, etc. will allow iterations of the computer model to optimize the design with respect to SWAS dimensions, length and number of gallery rows, etc.

A comparison of the results obtained from the analytical computer model was made with the results obtained from the simple well formula given in Section VI. This formula is:

$$Q = (\pi\, k\, (H^2 - h^2))/\ln (R/r) = (\pi\, k\, (H^2 - h^2))/2.3\mathrm{Log}_{10}\,(R/r)$$

Where :

- k = saturated hydraulic conductivity (ft/d),
- H = height of the ground water mound above an impermeable lower boundary at a distance r from the center of the recharge basin (ft),
- h = the original saturated thickness of the aquifer (ft),
- R = the radial distance (ft) from the center of the recharge basin to an aquifer boundary or an assumed outer limit of the mound (where H ~ h), and,
- r = the radial distance (ft) from the center of the recharge basin to a point on the mound for which H is calculated.

The bottom area of the SWAS was calculated to be 11,760 sq. ft. The radius of a circular area having the same bottom area = $(11{,}760\,/\pi)^{0.5}$ = 61 ft. In order to compare the results of the simple well formula with the analytical computer model, the value of R must be large enough to simulate an aquifer of infinite lateral extent[2]. Therefore, R has arbitrarily been assumed to be 100 x the equivalent radius of the recharge basin; i.e. 100 x 61 ft = 6100 ft. The value of r has been assumed as five feet; that is, the value of H will be computed at a distance of 5 feet from the center of the equivalent circular recharge basin.

$2.3 \times \log_{10} (R/r) = 2.3 \times \log_{10} (6100/5) = 7.1$. From Figure LL-2, h = 12 ft.

$$[H^2 - h^2] = \frac{Q \times 2.3 \times \mathrm{Log}_{10} R/r}{\pi K} = \frac{802 \text{ ft/day} \times 7.1}{\pi \times 25 \text{ ft/day}} = 72.5 \text{ ft}^2$$

$H^2 = 72.5 + (12)^2 = 216.5\ \text{ft}^2$. H = 14.7 ft. and the mound height = H-h = 14.7-12 = 2.7 ft.

Thus, the simple well formula predicts that the mounded SHWT will rise to 8 ft. - 2.7 ft. = 5.3 ft below the ground surface. This will only be 1.8 ft below the bottom of the SWAS, which is not acceptable. On the other hand, the analytical computer program predicts that the mounded SHWT will rise to 6.0 ft (8.0 ft.-2.0 ft.) below the ground surface, which is also unacceptable, as there will only be 2.5 ft of unsaturated soil beneath the SWAS. Thus, in both cases, fill would be required to provide the required 3 ft vertical separating distance.

In this example, the results of the two methods of mounding analysis are similar; that is, the site does not have sufficient hydraulic capacity to provide the required 3-ft. vertical separating distance. In other cases the results might be different. For example, had the depth from ground surface to the SHWT been 9.0 ft instead of 8.0 ft., the results from the

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[2] It should also be noted that the results from the simple well formula are somewhat sensitive to values of R assumed, except where it is known that H~h at the assumed value of R (i.e.: where R extends to a surface water body, open drainage ditch or ground water interceptor drain).

analytical computer program would have indicated that the site had sufficient hydraulic capacity (i.e.: mounded SHWT at 3.5 ft below the bottom of the SWAS). However, the simple well formula would have indicated that the design was not suitable, as the mounded SHWT would have risen to 2.8 ft below the bottom of the SWAS. This illustrates the need to carefully consider the method to be used in estimating the height of the ground water mound beneath an SWAS.

F. Fill Systems

1. General

The principles set forth in sections II, III, and IV are also applicable to the design of fill systems. However, the design and construction of fill systems will require significantly more effort and the cost to design and construct such systems are likely to be very much greater than for systems installed in natural soils. There are also regulatory constraints on the use of fill, as discussed further in subsection F.7.

2. Types of Fill Systems

Fill systems constructed to supplement natural soils are generally proposed where the existing soil is suitable with respect to hydraulic conductivity, wastewater renovative capacity, and depth to bedrock or other hydraulically restrictive layer, but a high ground water table will not permit the SWAS to have the required vertical separating distance above the mounded seasonal high ground water table that will exist during system operation. In this case, the soil downgradient of the SWAS has adequate hydraulic capacity to conduct the percolate from the SWAS for a sufficient distance downgradient to meet travel time requirements, but fill is needed to elevate the area in which the SWAS will be installed.

Fill systems constructed to replace natural soils are designated by the Department as Lateral Sand Filters, and are generally proposed in the following cases.

Case a.1. The existing soil is suitable with respect to hydraulic conductivity and wastewater renovation, but there is an insufficient depth of such soil above bedrock or other hydraulically restrictive layer (i.e. insufficient hydraulic capacity and insufficient unsaturated vertical separating distance).

Case a.2. An existing system has failed because the existing soil has inadequate hydraulic capacity or wastewater renovative capacity (or both), or there is insufficient separating distance above the seasonally high mounded ground water table, or the ground water table is at or below the surface of the bedrock.

In the cases of a.1. and a.2, the soils below and downgradient of the SWAS have inadequate hydraulic and renovative capacity. Therefore, sufficient fill must be placed to provide the required three feet of unsaturated soil of suitable renovative capacity below the bottom of the SWAS, and to provide the additional hydraulic capacity to conduct the percolate from the SWAS for a sufficient distance downgradient to meet the travel time requirements. It will also be necessary to provide a hydraulic barrier between the bottom of the fill and the soil on which the fill is placed to ensure that the percolate from the SWAS does not reach a ground water table located at or below the surface of the fractured bedrock before it is sufficiently renovated. The reason for the barrier is addressed below.

In the absence of definitive and conclusive evidence to the contrary, the Department will assume that all bedrock is fractured (the predominant condition in Connecticut) or contains large solution voids or channels such as exist in the karstic bedrock areas in northwestern Connecticut.

Where the existing soils have adequate hydraulic capacity but a high water table requires the use of fill to provide the required vertical separating distance, it is assumed that the percolate will flow downward through the unsaturated zone until it reaches the water table[3] in the soil above the bedrock and then will flow laterally in the direction of ground water flow in response to the hydraulic gradient. This assumption is considered reasonable because vertical mixing of the percolate with the ground water is usually slow and limited to several feet. Therefore, vertical movement of the contaminants remaining in the percolate into the bedrock aquifer can be ignored.

However, there is a need to be very careful where the soils are shallow to bedrock and where the seasonal ground water table is in the bedrock. This situation is represented by cases a.1. and a.2. and is often found on the crest of hills and ridges. The soil mantle at these locations often consists of well-drained or excessively well-drained soils or has inclusions of such soils that allow infiltrating water to rapidly reach the bedrock aquifer. Under these conditions, it is probable that any pathogens and pollutants remaining in the percolate from the SWAS after it flows through the unsaturated zone could easily reach the bedrock aquifer.

In such cases, it is difficult to determine the travel time of the percolate and a conservative assumption is made that it can travel quite rapidly through the bedrock fractures to a point of concern due to uncertainty in fracture distribution and orientation. In addition, the percolate in such cases will not receive the additional renovation normally provided by horizontal travel through a suitable soil aquifer for a sufficient period of time. Thus, the percolate must be prevented from entering the fractured bedrock until it has met the prescribed travel time requirement of the Department. Where it is necessary to locate an OWRS in such an area, a hydraulic barrier beneath the entire fill system may be required.

Another situation that needs careful attention is when the seasonal high water table is above the bedrock, but the water table recedes into the bedrock during the drier portions of the year. In this case, the bedrock aquifer can become contaminated in the same manner as described above. Where initial subsurface investigations indicate the absence of a water table in the soils or unconsolidated substratum, further investigations should be

[3] However, some investigators (e.g. Crosby, et al -1968; Pask -1988, 1994; and Mooers and Waller-1996) have shown that under certain conditions lateral flow will also occur in the unsaturated zone.

conducted during the driest period of the year.[4] (However, such investigations might be problematic if significant rainfall events occur in the normally driest portion of the year.) If these further investigations confirm that the water table may recede into the bedrock any time during the year, the Department may require that an approved hydraulic barrier be provided beneath the entire fill system.

Where a hydraulic barrier is required, the hydraulic conductivity and thickness of the barrier soil must be such that the vertical travel time through the barrier and any existing soil or substratum materials below the barrier will be equivalent to the travel times prescribed by the Department elsewhere in this document.

3. Requirements for Fill Material

Where fill is required only to provide vertical separation between the bottom of a SWAS and the seasonal high ground water table (leaching fill), the required vertical saturated hydraulic conductivity ($K_{sat.}$) of the fill material, after placement and compaction, is based on the unit hydraulic loading rate selected for design (e. g,: if a hydraulic loading rate of 0.8 gpd/sf, the maximum LTAR permitted, is selected for design of a SWAS, a $K_{sat.}$ value ≥ 29 ft/day is required). Coarse sand, as defined in Appendix B, should not be used for leaching fill.

It is also important that the vertical hydraulic conductivity of the fill should not be significantly lower than that of any soil horizon on which it is placed. If this should occur, it is possible for the fill with the lower K value to become saturated with the percolate from the SWAS before the percolate will flow downward through the fine soil-coarse soil interface. This situation can occur due to the soil moisture tension (matrix potential) being greater than the gravitational potential. In such cases, water will not cross the boundary between the upper fine soil and the lower coarser soil until the voids (capillaries) in the fine soil are filled. In such circumstances, it is likely that the requirement for unsaturated soil conditions would not be met in the fill. Thus, the use of soils having predominantly small particle sizes (e.g. fine sands, loamy sands and sandy loams) for fill material placed above coarser textured soils becomes problematic, and should be avoided.

Where fill is required to provide the saturated hydraulic capacity to conduct the flow laterally from the bottom of the unsaturated zone for a distance sufficient to meet the travel time requirement of the Department, the hydraulic conductivity required will be based on the linear loading rate, the slope of the mounded seasonal high ground water table, the depth of the fill and the SWAS configuration.

In this case, the designer can adjust any or all of these parameters to obtain a cost-effective system, although the adjustments are usually constrained by site features such as existing ground slope, the width and length of the site, and the characteristics of available fill materials. By making several trial analyses, the designer can determine the required hydraulic conductivity of the fill.

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[4] The presence of a permanent water table above bedrock may be indicated by the presence of a soil horizon with a gleyed (gray to bluish hue, chroma color ≤ 2) matrix. However, some low chroma colors may occur in unsaturated materials that contain little to no oxidized iron; this may more often be the case in sandy soils (fine-grained soils usually contain some iron). In such cases, the soil may or may not be permanently saturated. Therefore, it is important that saturation be confirmed by other means than soil color before assuming that the presence of gleyed soil indicates continual soil saturation.

Where fill is required to meet vertical separation and lateral travel time requirements, it is possible that two types of fill material might be required. One type of fill would be required for the unsaturated zone beneath the SWAS and another for the fill in the saturated zone beneath and down-gradient of the SWAS.

Ideally, suitable fill material would consist of a medium sand with a small amount of silt and clay particles having a sufficient reactive mineral content to which the contaminants in the percolate that are not removed by filtration can be sorbed as discussed in Section II of this document. It is this sorption that plays a large role in the ability of the soil to remove or attenuate pathogenic, organic and inorganic contaminants. However, in practice, medium sand with any significant amount of fine particles would have a significant reduction in hydraulic conductivity when the fill is placed and compacted (Ref: Subsection D of Section VI of this document). Thus the selection of fill material almost always involves a compromise between adequate hydraulic conductivity and adequate renovative capacity.

It has been found that, to avoid significant reduction of hydraulic conductivity due to compaction of the fill, the percentage (by weight) of fine particles passing the 100 and 200 mesh sieves should be limited to 6% (≤ 4% is preferable) and 3% respectively. In addition, the fill should not be too well graded from coarse to fine, as such material will tend to pack tightly when compacted, resulting in low values of K_{sat}. Thus, the use of fill materials with a uniformity coefficient (d_{60}/d_{10}) > 4 should be avoided.

It should be noted that in most cases, fill meeting the above requirements is obtained from commercial pits that extend a considerable distance below the soil solum (A and B horizons). In many cases, the fill is obtained near the bottom of the pits, where the deposition of colloids, soluble salts, and mineral particles by the process of illuviation has not occurred to a significant extent. In such cases, the sand may not have the phosphorus removal properties of the soils that lie closer to the ground surface. Therefore it will be necessary to evaluate the ability of such fill materials to sorb phosphorus, rather than use P sorption capacity values contained in the literature or given in Section II of this document, which are normally based on soils obtained from the soil solum. To determine phosphorus removal capabilities of proposed fill material, laboratory batch equilibrium experiments should be conducted to generate phosphorus adsorption isotherms on which the phosphorus sorption capacity (mg P sorbed/100 g of soil) can be based.

The particle size distribution of the fill material should be determined by sieve and hydrometer analyses (e.g. ASTM Standard Test Method D 422) for a number of proposed fill samples. It is recommended that the nest of sieves used should include those U.S. Standard Sieve sizes that will permit soil classification in accordance with the U. S. Department of Agriculture Natural Resources and Conservation Service (NRCS-formerly the SCS) soil texture classification method. This method classifies soils by particle size as shown in the following table.

TABLE FS-1

NRCS SOIL CLASSIFICATION BY PARTICLE SIZE RANGES

Classification	Particle Size Range, mm
gravel	2.0-75.0
very coarse sand	1.0-2.0
coarse sand	0.5-1.0
medium sand	0.25-0.50
fine sand	0.10-0.25
very fine sand	0.05-0.10
silt	0.002-0.05
clay	<0.002

A nest of sieves that includes the 3”, 3/4”, #4, #10, #18, #35, #60, #100, and #200 standard sieves will provide data for a plot of a particle size distribution curve. This data will aid in classification of the soil in conformance with the NRCS soil texture method for all but the silt and clay fractions. The hydrometer analysis will provide information on the relative amount of silt and clay present. The information obtained from sieve analyses and hydrometer analyses can also be used to define the soil texture, using the soil textural classification system adopted by the NRCS (See Appendix B).

The fill material must be placed in layers (lifts) not to exceed 12 inches in loose depth and compacted to a specified density. If this is not done, the hydraulic conductivity of the fill eventually will be significantly reduced due to settlement caused by gravitational affects and the affect of infiltrated precipitation.

If each layer of fill is compacted to at least 90% of maximum density[5], it is unlikely that any further compaction will occur. (N.B. However, where fill is placed in areas subject to heavy wheel loadings, it should be compacted to 95% of maximum density.) The hydraulic conductivity of each layer of fill must therefore be determined after it has been placed and compacted.

Initial testing of representative fill material samples should be conducted by a soil testing laboratory by compacting the material to at least 90% of maximum modified Proctor density in order to determine the hydraulic conductivity of the material after compaction. After the samples have been adequately compacted, tube samples of the compacted material should be taken and the hydraulic conductivity determined as discussed in Section VI of this document. Because of the inherent variation in soil properties and test results, there will be a range of K values obtained from such tests. Therefore, the K values specified for the fill material should span a range that can reasonably be attained in the field.

4. Hydraulic Analysis

Where fill is placed to increase the hydraulic capacity of a site, it will be necessary to carry out a hydraulic analysis to determine the thickness and lateral dimensions of the fill required. These hydraulic analyses will differ in detail and complexity depending upon the nature of the fill system.

[5] As determined by the modified Proctor compaction test performed in conformance with ASTM D1557, Method D

Case a. Fill required for adequate vertical separation distance above seasonal high ground water table (Leaching Fill).

In this case, the flow from the SWAS will basically be vertically downward through the fill (leaching fill) and existing subsoils until it reaches the water table. The depth of leaching fill that will be required will depend upon the calculated increase in the height of the seasonal high water table. For sloping sites, where the ground water table slope is similar to the surface slope, such calculations are similar to those shown for basic site hydraulic capacity analyses in Section VI, Subsection F 3 of this document. For cases where the existing ground water table is essentially horizontal, a three dimensional analysis will be required, as discussed in Section VI, Subsection G of this document.

The lateral extent (width of the leaching fill) will depend upon the maximum lateral extent of the SWAS (parallel to the surface contours). The leaching fill should extend for a distance of at least five feet beyond the entire perimeter of the SWAS facilities to provide for some lateral dispersion of the percolate. The finished grade over the SWAS should have a slope of at least 2% to permit precipitation to flow off of the filled area.

The depth of the leaching fill and any existing soil that will remain unsaturated must be such as to provide sufficient phosphorus sorption capacity as discussed in subsection G.4 of this section.

A berm of compacted inorganic material (glacial till) having a relatively low hydraulic conductivity compared to the leaching fill material should be placed completely around the perimeter of the leaching fill material and should be toed into the existing surface for a depth of at least one foot. This berm should have a top width of at least five feet and of such additional width that may be necessary to accomodate compaction equipment.

An example of such a system is shown in Figure FS-1.

Case b. Fill Required for Providing Adequate Vertical Separation Distance Above Selected Soil Horizons

With respect to hydraulic capacity computations, Case b is a combination of Cases a and c and the methods described herein for those cases can be used for Case b. It should be noted, however, that there would be at least two soil horizons involved in conducting the SWAS percolate away from the SWAS. These would include the fill horizon and at least one existing soil horizon, and thus a different K value should be used to determine the hydraulic capacity of each horizon.

Case c. Lateral Sand Filters.

A fill system used to increase the hydraulic capacity of a site by constructing part or all of the system above existing grade has been designated as a Lateral Sand Filter (LSF) by the Department. Several lateral sand filters have been approved and constructed in Connecticut.

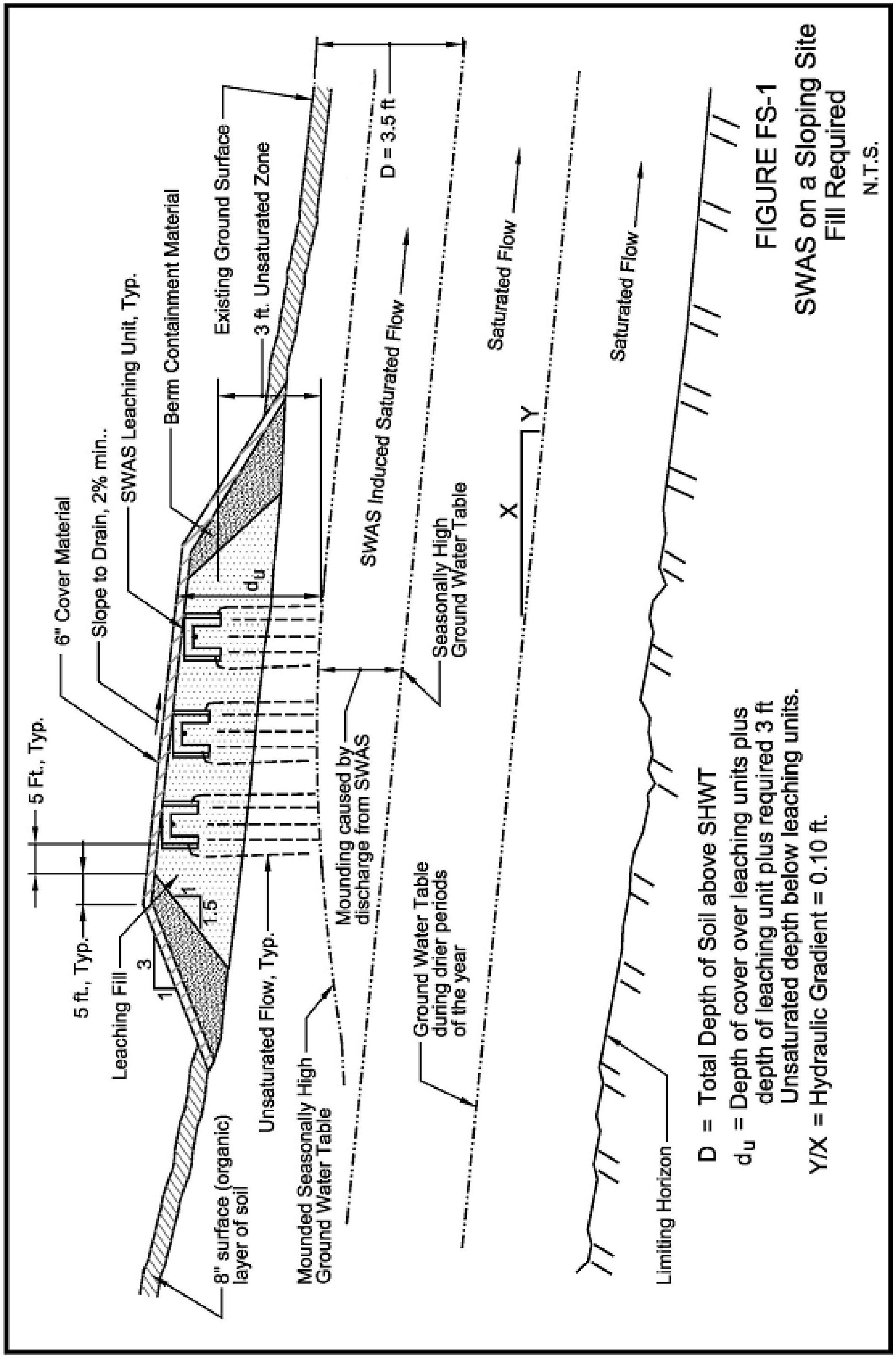


FIGURE FS-1
SWAS on a Sloping Site
Fill Required
N.T.S.

Hydraulic analyses for determining the thickness and lateral dimensions of the fill will differ in detail and complexity depending upon the nature and configuration of the system. A number of variables have to be considered in the design of a LSF, including:

- length, width, height, and cross-section geometry,
- the existing local hydraulic gradient, or the established hydraulic gradient,
- the LSF site area, geometry and topographic constraints
- the amount of existing soil above the bedrock on the site,
- the position of the water table with respect to the bedrock,
- the hydraulic capacity of the existing soil,
- the landscape position of the LSF on the site,
- the soil materials readily available, and the range of soil hydraulic conductivities,
- the design flow and organic loading,
- the configuration of the leaching system,
- the required travel time of the SWAS percolate,
- points of concern within and adjacent to the proposed site, and
- Applicant's ownership or control of zone of influence extending to a wetland or surface water body.

Many of these variables are interrelated; and as the values of one variable or more are changed, they will have an affect on the configuration of the LSF; (e.g., LSF length, width, depth and cross-section geometry and the hydraulic gradient).

The LSF basically consists of the sand used to provide renovation of pretreated wastewater contained within a three-sided U-shaped berm, constructed of low permeability soil materials, that directs the flow of liquid in the LSF down-gradient to the open end of the system. The renovated wastewater then seeps from the toe of the sand fill as a non-point source discharge.

The designer of a LSF has the ability to specify and control certain design parameters that could not be specified or controlled for a system constructed in native soils. For example, soil materials can be selected (and thus the hydraulic conductivity (K) values, within a reasonably close range) and, within site boundary and topographic constraints, the hydraulic gradient can be selected and the system configuration can be varied without concern for natural soil conditions, boundary conditions and topographic constraints. However, this latitude in design comes at a very significant cost. The cost to design and construct a LSF can be at least an order of magnitude greater than for similarly sized systems constructed in natural soils that have adequate hydraulic and renovative capacities. This is due to the fact that design of an LSF system is more complicated, and the construction and quality control testing is more difficult, as compared to what would be experienced for systems of equal capacity constructed in natural soils.

Major cost factors are the importation of select fill materials, careful placement and compaction of the fill, the associated extensive laboratory and field testing that is required to obtain dependable data for design and during construction for quality control, and the extensive construction inspection that must be employed. The design of an LSF is more involved because the latitude in selecting design values may require a number of iterations of the design in order to arrive at one that is cost-effective and meets the water quality goals of the Department.

In practice, however, there are other materials and components of the LSF that are required to insure the integrity of the system. These include, but are not necessarily limited to:

- a low-permeability layer of soil beneath the sand. in certain instances, to confine the pretreated wastewater to the sand fill,
- pervious toe drains (usually constructed of geotextile fabric, broken stone and riprap) at the outside toes of the berms and at the toe of slope at the downslope end of the sand fill to prevent slope failure due to excess pore water pressure,
- materials to stabilize the downslope end of the sand fill to prevent sloughing and erosion of the fill,
- vegetated topsoil for cover over the berms and top of the sand fill, and,
- the materials required for the SWAS

Figure FS-2 shows typical sections through a LSF. This figure depicts many of the conditions that may be encountered when the use of a LSF is being considered.

Suggested steps for preliminary design of an LSF are given below. Unless otherwise stated, the hydraulic conductivity, K, is the horizontal saturated hydraulic conductivity of the soil materials.

A. Design the portion of the LSF located down-gradient of the leaching fill area. The objective here is to compute the required cross-sectional area of the saturated sand fill through which the design wastewater daily flow (Q_{df}), will flow to the down-gradient end of the LSF. Where a liner is required below the LSF, the design flow must also include the precipitation that infiltrates through the top of the LSF (Q_{pi}).

    1. Compute the total design flow, Q_t.

        This includes the design wastewater daily flow (Q_{df}) and, where a low permeability liner is required below the LSF, the precipitation that will fall on and infiltrate the LSF (Q_{pi}). (A reasonable precipitation value might be based on the average daily precipitation during the maximum monthly precipitation that occurs late in the year, when the ground has not yet frozen but evapotranspiration is negligible. A conservative approach, with respect to the hydraulic calculations for the LSF, would be to assume that all of this precipitation infiltrates the top surface of the LSF).

        Q_t is actually a variable because the amount of the infiltrated precipitation added to Q_{df}, as one proceeds from the up-gradient end to the downgradient end of the LSF, increases with the incremental increase of surface area as the flow proceeds downgradient. However, the total precipitation infiltrated through the top surface of the LSF is a relatively small part of Q_t and therefore the entire amount of the precipitation infiltrating through all of the top surfaces of the LSF, (Q_{pi}), can be added to Q_{df}. This will provide a small factor of safety when computing the required depth of the saturated sand fill.

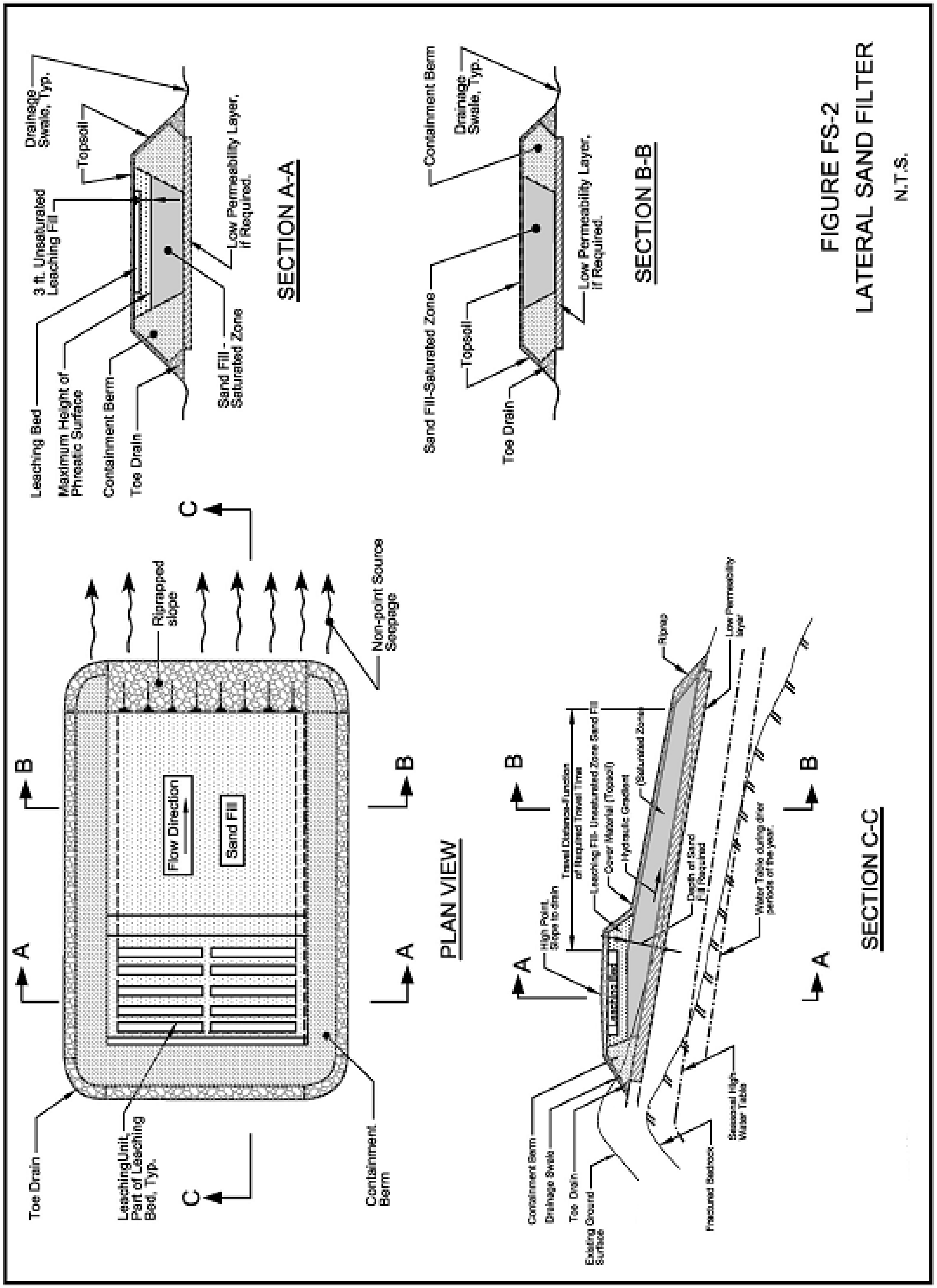


FIGURE FS-2
LATERAL SAND FILTER
N.T.S.

2. Compute the total required effective leaching area (ELA) based on the leaching units selected.

3. Assume a layout of leaching units that will provide the required ELA and determine the width of the required leaching area perpendicular to the hydraulic gradient and the length of the leaching area in the direction of the hydraulic gradient.

4. Assume a value of K for the compacted sand fill. (Ref: Appendix C.)

5. Assume a value for the hydraulic gradient.

   While the designer has some latitude in selecting the hydraulic gradient, in most instances, the hydraulic gradient should be configured as closely as possible to the pre-existing topography to minimize construction costs. However, in some cases the distance from the leaching fill area to the downgradient property line, or other point of concern, may be constrained. It may then be necessary to reduce the hydraulic gradient and thus reduce the required travel distance since the travel distance is inversely related to the hydraulic gradient when K is held constant.

   Another means of reducing the travel distance is to select a fill material with a lower value for K for computing travel time. However, this will have an affect on the required flow area perpendicular to the hydraulic gradient.

6. Calculate the required flow area cross-section perpendicular to the hydraulic gradient. The required flow area is computed from the Darcy's law $Q_t = K\ i\ A$. Thus, $A = Q_t /(K\ i)$. (To get A in square feet, Q_t must be in units of cubic ft/day and K must be in units of ft/day.)

7. Determine the geometry of the flow area cross-section, which will normally be in the form of a trapezoid, with a bottom width smaller than the top width, due to the shape of the containment berms.

   The side slopes of the trapezoid will be the same as the inside slope of the containment berms. A reasonable first assumption for these slopes is 2 horizontal to 1 vertical. The bottom width of the trapezoid (W_b) will equal the width of the required leaching area perpendicular to the hydraulic gradient as calculated in step No. 3.

   The area of the trapezoid, $A = W_b\ H + 2H^2$, where H is the required depth of sand. Also, from Step No. 6 above, $A = Qt/(Ki)$; therefore, $Qt /(K\ i) = W_b\ H + 2\ H^2$. All terms in this equation except H are known and H can be determined by solving the resulting quadratic equation.

8. The length of the LSF, of cross-section A, down-gradient of the leaching fill area will depend upon the values of K used to calculate travel time, the slope of the phreatic surface, or hydraulic gradient, i, and the required travel distance.

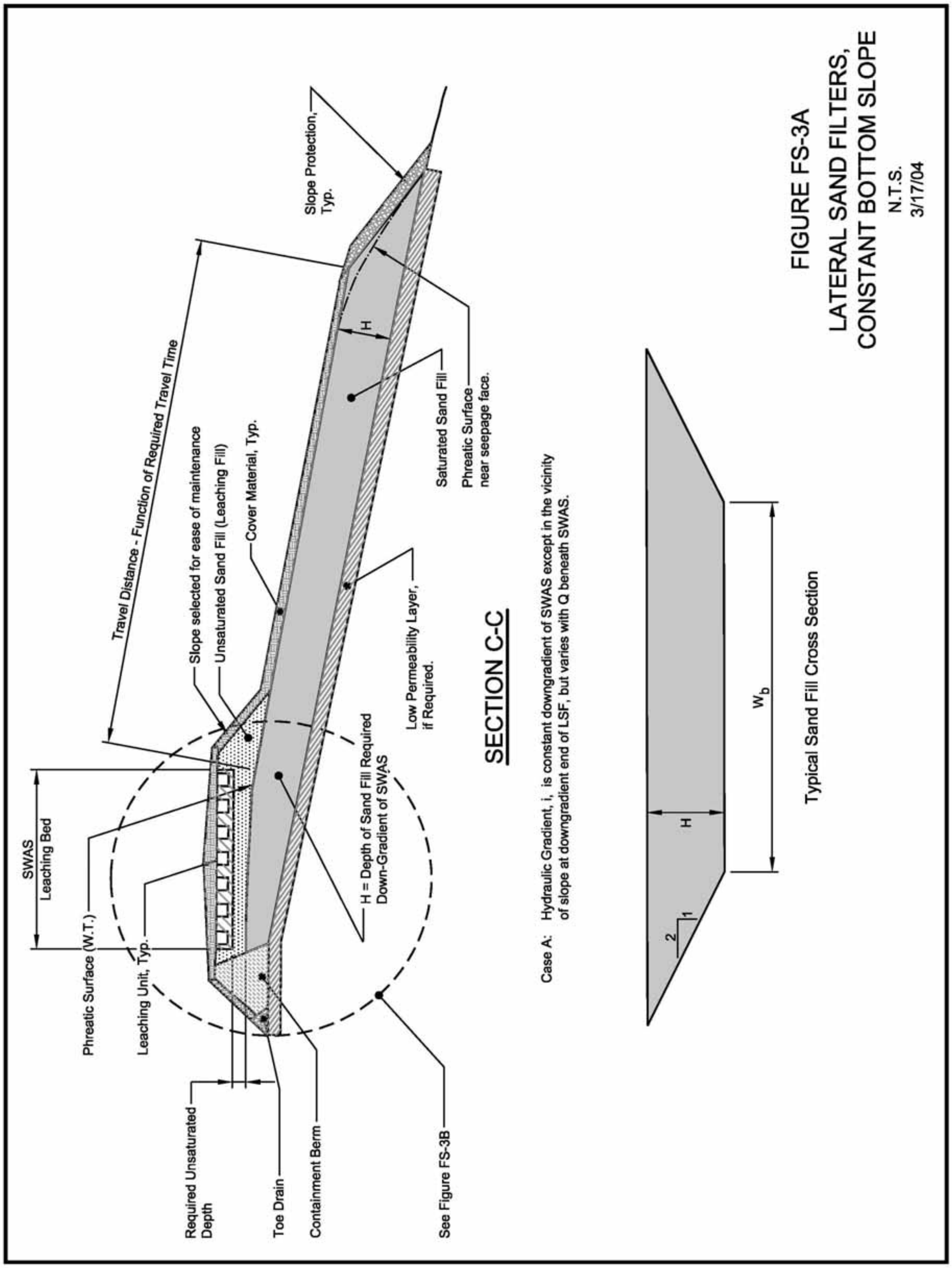
The required travel distance in the LSF downgradient of the leaching fill area (travel distance) = travel time required, in days, x (K i)/n. For most of this distance, the phreatic surface will have a constant gradient (i) and depth (H). The design values of K and n (porosity) used to compute travel distance are normally considered to be constant for the entire LSF. However, from a point near the crest of the sloping seepage face at the down-gradient end of the LSF to the point where the phreatic surface will intersect the seepage face, the phreatic surface will become steeper. The velocity in this portion of the LSF will be significantly higher than in the full fill section because of the increase in the slope of the phreatic surface as the flow approaches the seepage face.

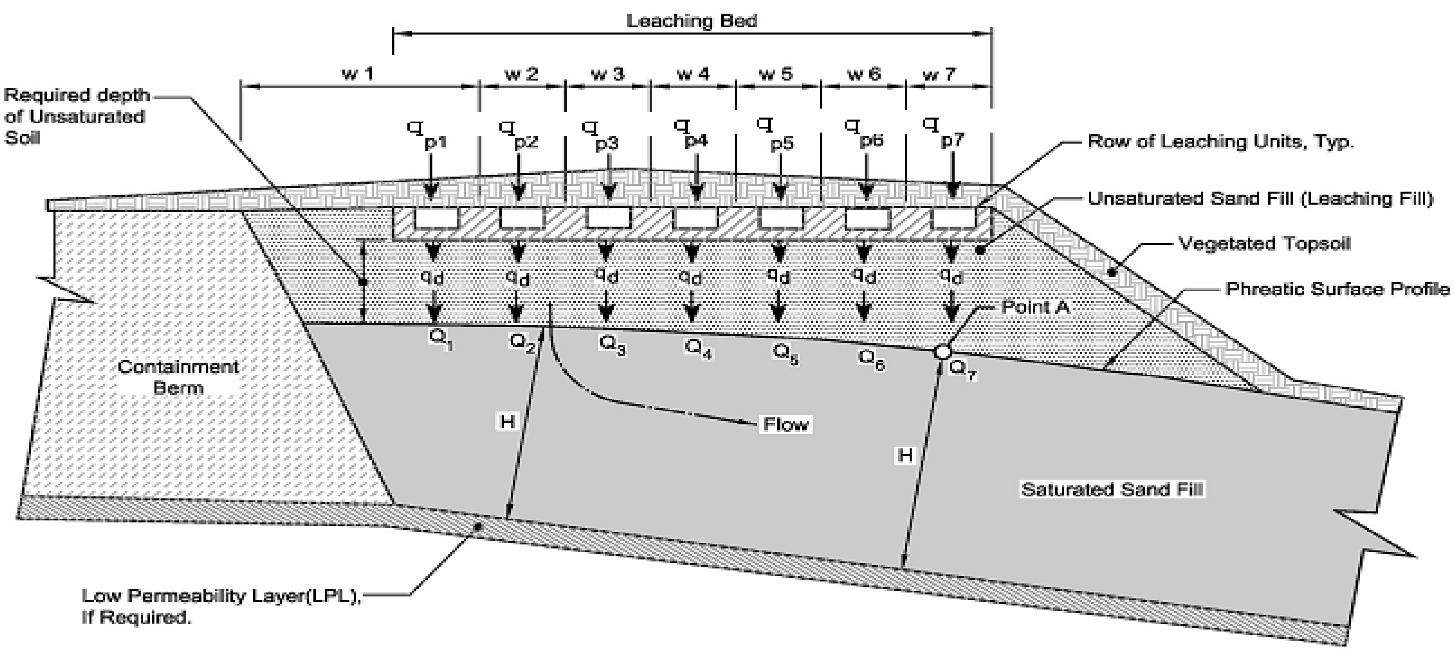
A flow net analysis in this portion of the LSF can be made that will define the steeper phreatic surface. This will enable calculation of the travel time from the crest of the slope to the point of seepage breakout at the face of the slope as a function of the increasing hydraulic gradient. However, a simpler method of defining the phreatic surface in this portion of the LSF can be used without significant error. This method assumes the phreatic surface to be a straight line drawn from a point on the phreatic surface beneath the crest of the slope to the toe of the sand fill. The slope of this line can be taken as the hydraulic gradient for computing the travel time increment in this portion of the LSF. The required length of the LSF, from the downgradient end of the SWAS to the crest of the slope at the downgradient end of the LSF, can then be computed using the required travel time less the aforesaid time increment and the procedure for computing travel distance given above.

B. Design the saturated portion of the LSF beneath the leaching fill area. The objective here is to compute the required cross-sectional area and depth of the saturated sand fill through which the design wastewater daily flow (Q_{df}), plus precipitation that infiltrates through the leaching fill (Q_{pi}) where a liner is used, will flow in a horizontal direction beneath the unsaturated portion of the leaching fill. There are several methods that can be used to compute the depth of the saturated sand fill.

1. One method of computing the depth of the saturated sand fill is to assume a constant bottom slope for the saturated fill, as shown in Figure FS-3 A. Having established the depth of flow in the lateral sand filter downgradient of the leaching fill area, the depth of the leaching fill below the SWAS can be determined by calculation of the phreatic surface below the SWAS. Refer to Figure FS-3B, which is an enlargement of Section C-C shown in Figure FS-3A.

   The phreatic surface profile below the SWAS leaching units is determined by the following form of Darcy's Law: $I = Q_t/KA$, where Q_t = total design flow at any selected point on the profile, A = the cross-sectional area of flow at Q_t, and K is the hydraulic conductivity for the sand fill. The phreatic surface profile is determined by summation of the q_d and q_{pi} values for each sub-area shown in Figure FS-3B and treating the flow as a line source at the centerline of each row of leaching units.


Travel Distance - Function of Required Travel Time
SWAS
Leaching Bed
Phreatic Surface (W.T.)
Leaching Unit, Typ.
Slope selected for ease of maintenance
Unsaturated Sand Fill (Leaching Fill)
Cover Material, Typ.
Slope Protection, Typ.
H = Depth of Sand Fill Required Down-Gradient of SWAS
Low Permeability Layer, if Required.
Saturated Sand Fill
Phreatic Surface near seepage face.
H
Required Unsaturated Depth
Toe Drain
Containment Berm
See Figure FS-3B
SECTION C-C
Case A: Hydraulic Gradient, i, is constant downgradient of SWAS except in the vicinity of slope at downgradient end of LSF, but varies with Q beneath SWAS.
2
1
H
W_b
Typical Sand Fill Cross Section
FIGURE FS-3A
LATERAL SAND FILTERS,
CONSTANT BOTTOM SLOPE
N.T.S.
3/17/04



COMPUTATION OF PHREATIC SURFACE PROFILE ON THE BASIS OF A CONSTANT BOTTOM SLOPE

q_d = Design flow from one row of leaching units
q_{p1} = Precipitation infiltrating an area with dimensions = W1 × top width of LSF at SWAS. (Where LPL is required)
$Q_1 = q_{p1} + q_{p2} + q_{pn}$, and so on.
When H is held constant, the saturated cross-section of flow is constant = A. Thus, the hydraulic gradient at any point on the phreatic surface profile, i = ? Q/(KA).
The elevation at Point A will have been established on the basis of the site restraints and the constant hydraulic gradient downgradient of Point A. The elevation of the phreatic surface profile beneath the centerline of each row of leaching units can then be calculated based on the calculated hydraulic gradients, the distance between the rows and the elevation of Point A.

FIGURE FS-3B
LATERAL SAND FILTERS
PHREATIC SURFACE PROFILE
N.T.S.

Computation of the phreatic surface profile by the above equation is accomplished by assuming a constant bottom slope of the entire LSF fill, including that portion of the fill beneath the SWAS. Therefore, the hydraulic gradient required to conduct flow through the saturated sand cross-section varies linearly as a function of Q_t. The resulting phreatic surface profile has a hydraulic gradient equal to that of the sand fill downgradient of the SWAS and the hydraulic gradient at the up-gradient end of the LSF approaches zero. The generalized procedure for making these computations is shown in Figure FS-3B.

2. Another method to compute the depth of the saturated sand fill is to assume a constant hydraulic gradient for the full length of the LSF, as shown in Figure FS-3C. The generalized procedure for making these computations is shown in Figure FS-3D. In this method, the bottom slope of the fill beneath the SWAS will vary.

3. A third method is to vary both the depth of the saturated fill and hydraulic gradient beneath the SWAS. This is a more involved procedure, requiring several iterations because of the two unknown variables, H and i, and may be of limited practical use.

C. Compute the depth of low permeability layer required beneath the LSF.

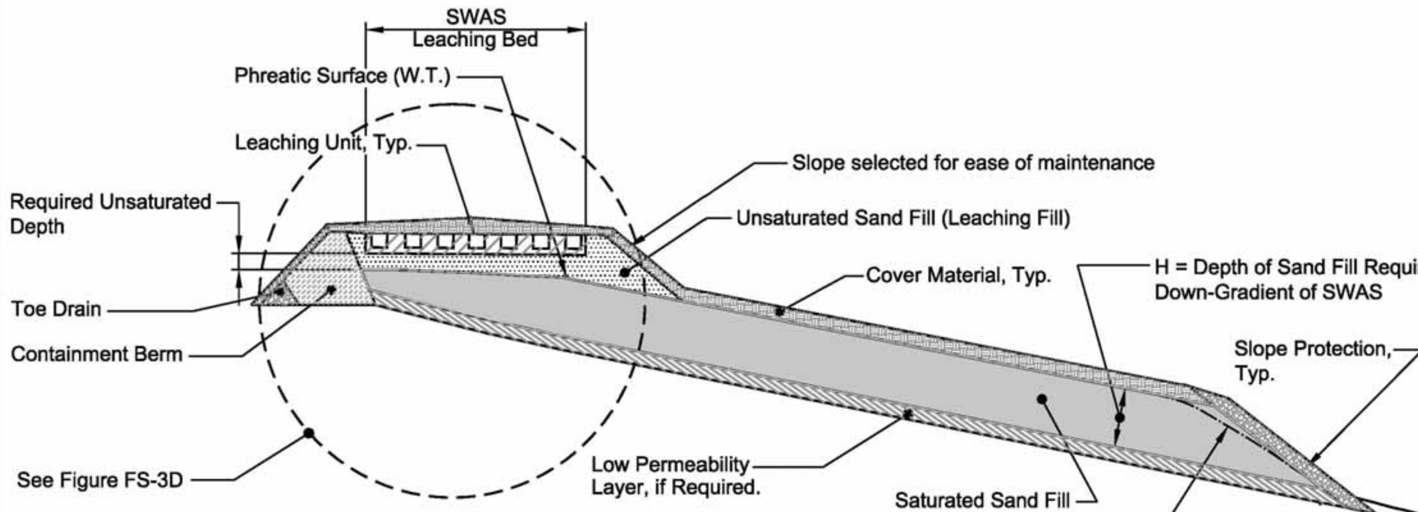
To calculate the required depth of the low permeability layer (LPL), if such a layer is required, the hydraulic gradient through the LPL and the vertical K of the layer must be known. The hydraulic gradient = H/L, where H is the height of the water table in the sand above the LPL and L is the thickness of the LPL. Therefore, the height of the water table, (H), above the LPL must first be determined.

With H known from step A.7, the required minimum vertical K for the LPL based on an assumed value for L, and value of n can be calculated, or the required value for L can be calculated for an assumed known value of vertical K and n.

Assuming a value of L for the LPL, which is also the travel distance corresponding to the required travel time, and having previously determined H, the hydraulic gradient through the LPL is known and the corresponding value for the vertical K may be calculated.

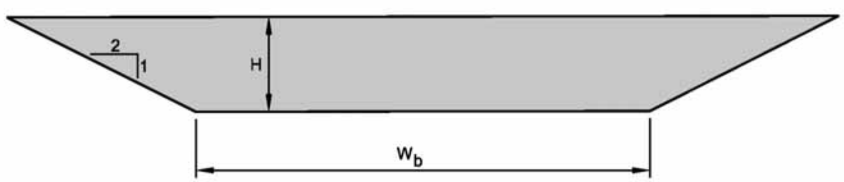
The travel time through the LPL = V x T= [(K x H/L)/n] x T, where V = the linear velocity of flow through the LPL, T = the required travel time, and n = the porosity of the LPL. V x T also equals the required value L. Thus, L/T = (K x H/L)/n, and L = [T/n] x [K x (H/L)]. This equation for L cannot be solved directly, because it contains three unknowns, H, K and n. However, after determining the value of H from step 7, assuming a value for maximum allowable K and n will permit the calculation of L, or assuming a value for L and n, the maximum value for K can be determined. For example:

H for a particular LSF has been calculated to be ten feet. Assume the thickness of the LPL =2 ft =L. Then H/L = 10/2 = 5. Assume a reasonable porosity for glacial till is 0.3. Assume the required travel time is 56 days.



SECTION C-C

Case B: Hydraulic Gradient, i, assumed to be constant for entire length of Saturated Sand Fill, except in vicinity of slope at downgradient end of LSF.
H varies with Q beneath SWAS, is constant downgradient of SWAS.



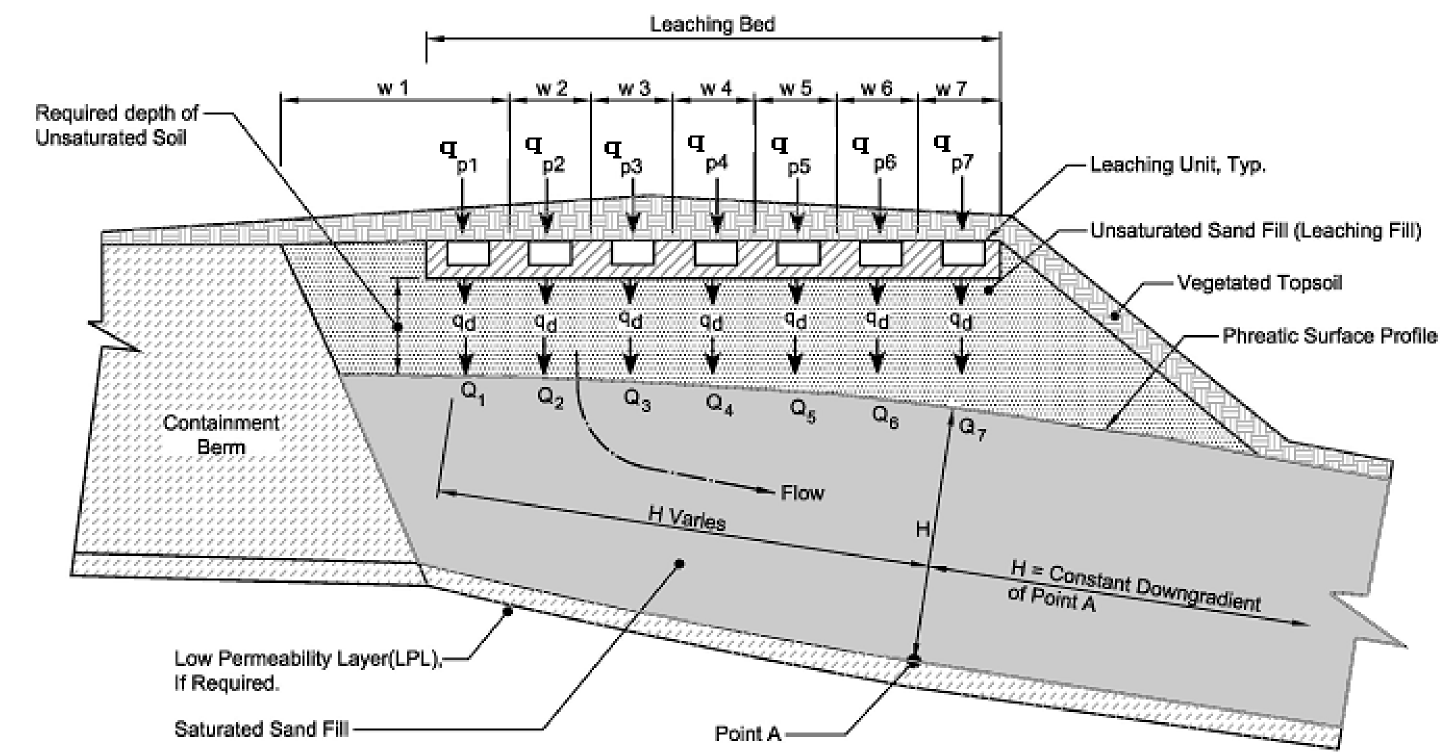
Typical Sand Fill Cross Section

FIGURE FS-3 C

LATERAL SAND FILTER
CONSTANT HYDRAULIC GRADIENT

N.T.S.

Rev.3/17/04



COMPUTATION OF BOTTOM SURFACE PROFILE ON THE BASIS OF A CONSTANT HYDRAULIC GRADIENT

q_d = Design flow from one row of leaching units
q_{p1} = Precipitation infiltrating an area with dimensions = W1 x top width of Leaching Fill, if LPL is required.
$Q_1 = q_{p1} + q_d$; $Q_2 = q_{p1} + q_{p2} + 2q_d$; and so on.

The cross-sectional area required at any point upstream of Point A will be a function of the total flow (Q_t) at that point and the constant hydraulic gradient, I (e.g. $A=Q_t/(Ki)$. For a trapezoidal cross-section with a constant bottom width W and constant side slopes of 2 horizontal to one vertical, A also = $WH + 2H^2$ and this equation can be solved for H by equating the two equations for A. Assuming that the elevation at Point A will have been established on the basis of the site restraints, the constant hydraulic gradient and total flow downgradient of that point, the bottom elevation of any point upgradient of Point A can be determined by subtracting the value of H calculated for that point from the elevation of the hydraulic gradient at that point.

FIGURE FS-3D
LATERAL SAND FILTERS
CONSTANT HYDRAULIC GRADIENT
BOTTOM SURFACE PROFILE

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Travel velocity, V = (K x H/L)/n; thus K = (V x n) /(H/L)= (V x 0.3)/5. V also = 2 ft/56 days = 0.036 ft/day. Thus, the required K ≤ (0.036 x 0.3)/5 ≤ 2.1 x 10^{-3} ft/day.

As shown in Table 4 of Section VI, remolded glacial till[6] can have a K =1.5 x 10^{-3} ft./day, (particularly if the till has a significant amount of silt and clay). Various publications show K values for glacial till ranging from 2.8 x 10^{-7} to 5.7 x 10^{-2} ft/day (Walton-1991; Freeze and Cherry-1979; Domenico and Schwartz-1997). Clays have values of K one or more orders of magnitude lower than glacial till. However, clays are difficult to obtain, are more costly than glacial till soils in Connecticut, and are more difficult to place and compact. In addition, clays may not provide a stable base for the overlying fill, depending upon the slope of the LPL. This may also be true for the various manufactured low permeability liner materials available.

Therefore, poorly graded glacial tills (gravel/sand/silt/clay mixtures) that have a K ≤ 1.5 x 10^{-3} ft./day (0.53 x 10^{-6} cm/s) after placement and compaction to 90% maximum density are usually suitable for constructing a LPL of reasonable thickness. Glacial tills with similar values of K can also be used for the construction of the containment berms.

D. Determine the preliminary configuration of the LSF in the direction of the hydraulic gradient.

1. Compute height of LSF above the LPL at the up-gradient end of the LSF.

   The height will be based on the value of H calculated in step 7, the required depth of unsaturated leaching fill beneath the SWAS (3 ft. min.), the height (thickness) of the SWAS, and the depth of cover material.

2. Verify the selected values for top width and side slopes of the containment berms

   An initial trial section for the containment berms can be assumed using side slopes of 2 horizontal to 1 vertical and a top width sufficient to allow placement and compaction of the low permeability berm material. A top width of at least eight feet is a reasonable first assumption, as this will permit ease of construction. The berm must be stable against the earth pressure exerted by the sand fill and the pore water pressure. The outer slope of the berm will be affected by seepage through the berm near its toe, to a height of roughly one-third H (U.S. Department of the Interior -1977). Therefore a toe drain will be required to protect against slope failure of the berm due to seepage. However, because of the earth pressures and seepage forces involved, it may be prudent to have the stability of the berms, based on assumed dimensions and slopes and the type and configuration of the toe drains, checked by a geotechnical engineer with knowledge of design of earth embankments to contain saturated soils.

[6] Note: It is assumed that glacial till, when placed and compacted to 90% of maximum density, can be considered as having been remolded.

The horizontal travel time (T) through the side of the berm to the toe drain must also be checked. This travel time is a function of the highest value for H previously determined, the 95 percentile value for K for the compacted berm material, the hydraulic gradient through the berm and the net bottom width of the berm, (W_b), exclusive of the width occupied by the toe drain. The hydraulic gradient at the bottom of the berm can be taken as H/W_b. Thus, the travel time = V x T where V = (K x H/W_b)/n.

3. Methods and materials for stabilizing the downslope end of the LSF must also be selected. Such stabilization can consist of the installation of geotextile fabric and riprap, or by other suitable means.

4. After the configuration of the LSF has been determined, the stability of all elements of the LSF should be checked. Calculations should be made to determine if the LSF will be a stable earth structure during and after construction and after being placed into operation and subjected to steady-state seepage (pore) pressures. Such calculations should be made by a person qualified to perform stability analyses of earth structures. Factors of safety against failure should be at least 1.3 during and after construction and 1.5 after the LSF is placed into operation.

It is evident from the above discussion that, for a given design flow, configuration of the LSF will be effected by a number of factors including: a.) the range of K values for the sand fill and low permeability materials, b.) the selected hydraulic gradient, c.) the required travel time, d.) the cross-section selected for the containment berms, e.) the configuration of the SWAS and f.) the earth slope on which the LSF will be constructed. Thus it may take several iterations of the design process to arrive at a cost-effective design for the LSF that will make use of suitable available materials while meeting site and boundary constraints.

5. Construction Quality Control

After initial testing of the fill materials has been satisfactorily completed as discussed below, the materials can be delivered to the project site. These materials should be subject to further testing during construction, including grain size analysis, density of soil in place after compaction, modified Proctor density tests, and hydraulic conductivity. The number of samples to be taken and tests to be made for density of soil in place, and hydraulic conductivity, can be determined from Figure QC-1 based on the sample coefficient of variation (C_v) for each previous lift. For the first lift, a reasonable estimate of the expected C_v should be made, based on the results obtained from the initial laboratory testing. Experience has indicated that where suitable fill material is used, the value for C_v may range between 0.2 and 0.3.

All samples should be taken at random locations within the filled area; however the samples should be representative of the fill placed throughout the area. Therefore, the fill area should be divided by a grid pattern, with individual rectangles with the grid pattern having an area not greater than 2,500 sq. ft. The samples for each fill lift should be taken at random locations within each grid rectangle.

Any layer of fill material that does not meet the compaction and hydraulic conductivity requirements should be removed and replaced with material that, after compaction, will meet those requirements.

The moisture content of the fill should be controlled as required to meet the compaction requirements and can be estimated from the results of the initial moisture-density compaction tests and the subsequent tests for each layer of fill placed. Excessive precipitation, or inadequate watering, can cause problems with compaction of the fill materials. Subfreezing temperatures require frost protection of the emplaced fill. Methods for providing such protection can include placement of loose, uncompacted fill material over the compacted fill or covering the compacted fill with thermal blankets or a thick layer of hay or straw.

Failure to attain the required hydraulic conductivity may require further compaction and re-testing if the resulting K values are too high, or scarification of the fill layer and re-compaction of the layer to a lower field density if the K values are too low. In some cases, removal of the fill material and placement of new fill material may be required. In other cases where the K values are too low, it may be cost-effective to allow that layer to remain in place and add an additional layer to the top of the fill to provide the additional hydraulic capacity required. It is also possible that, upon completion of the number of layers as designed, it may be found that the total design hydraulic capacity has been attained. This possibility can be checked by summing up the hydraulic capacity of the individual layers. However, the Department should be consulted before a decision is reached concerning the need of an additional layer under these circumstances.

Both laboratory and field testing of all fill materials should be conducted by a commercial laboratory approved by the Department. Tests should be conducted in conformance with the following standards:

a. ANSI/ASTM D422 - Particle Size Analysis of Soils (Washed Method)

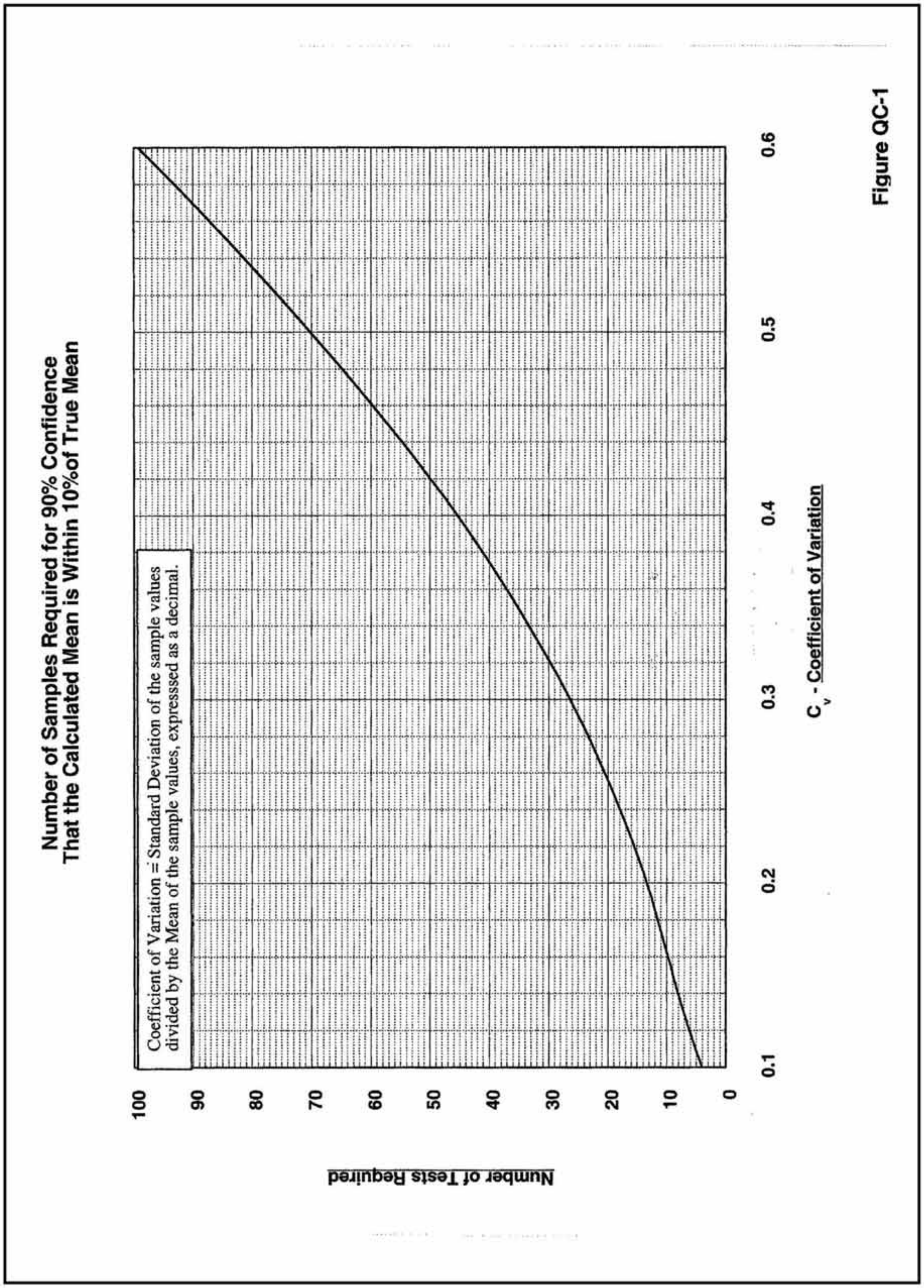
b. ANSI/ASTM D1556 - Test Method for Density of Soil in Place by the Sand-Cone Method, or

c. ANSI/ASTM D2167 - Test Method for Density of Soil in Place by the Rubber Balloon method, or

d. ANSI/ASTM D 2922 - Standard Test Methods for Density of Soil and Soil-Aggregate in Place by Nuclear Methods (Shallow Depth).

e. ANSI/ASTM D1557 - Moisture-Density Relations of Soils and Soil Aggregate Mixtures Using 10 lb. Rammer and 18 Inch Drop.

f. Hydraulic conductivity testing should be performed in conformance with Section VI of this document.


Number of Samples Required for 90% Confidence
That the Calculated Mean is Within 10%of True Mean
Coefficient of Variation = Standard Deviation of the sample values divided by the Mean of the sample values, expresssed as a decimal.
Number of Tests Required
100
90
80
70
60
50
40
30
20
10
0
0.1
0.2
0.3
0.4
0.5
0.6
C_v - Coefficient of Variation
Figure QC-1

6. Placement of Fill

The results of all field inspections should be reported to the Department by a licensed Professional Engineer who has conducted or witnessed such inspections, or who verifies the results of such inspections by a member of his staff who has the proper qualifications to conduct such inspections.

Prior to placement of fill, all boulders, rocks, trees, tree stumps, other existing vegetation and organic matter and topsoil should be removed from the area(s) in which the fill is to be placed. (It is important that, for normal fill systems, any organic soil horizon be removed from beneath the entire fill area. Placement and compaction of the fill over organic soil horizons would cause these horizons to be compressed, resulting in a mat of low hydraulic conductivity that would restrict downward flow of the effluent. This is normally not required when constructing a LSF.) The prepared surface should be left in a scarified, unsmeared and uncompacted condition.

The fill material should then be placed in layer (lifts) not exceeding one foot in compacted thickness. The fill should be placed by dumping on the edge of the fill area, keeping rubber tired vehicles and equipment off the area.

A crawler tractor (bulldozer) should be used to move the fill material into final position. Each layer of fill should be compacted to at least 90% of its maximum modified Proctor density as determined by compaction testing. After each layer of fill material has been satisfactorily compacted (as determined from soil density tests), tube samples should be taken to confirm that the desired hydraulic conductivity has been attained. Samples should also be obtained for particle size (sieve) analyses.

After placement of the fill has been satisfactorily completed, it should be covered with topsoil, limed and fertilized as required, and then seeded and mulched. The seed mixture specified for the vegetated cover surfaces over the sand fill should be carefully chosen to withstand the relatively harsh conditions that may exist because of the lack of a B soil horizon. Despite the fact that the LSF contains a significant depth of saturated soil, the actual phreatic surface (water table) will most likely be well below the topsoil layer due to use of conservative design flows and design assumptions and the relatively coarse nature of the sand fill materials. Consequently, seed mixtures tolerant of droughty conditions should be specified.

7. Regulatory Constraints on the Use of Fill

Filling for subsurface wastewater renovation systems is an area where pure engineering analysis comes into conflict with construction reality and regulatory requirements. The basic goal of the Department is that in-place measurable, testable natural soil formations provide the treatment prior to a broad non-point source discharge. The ultimate use of fill is described in this section under “Lateral Sand Filters”, where the fill provides the treatment normally provided by natural in-place soils and renovated wastewater seeps out of the toe of the fill embankment. Any proposal for use of a lateral sand filter will receive a very stringent review due to the following realities:

1. Wastewater renovation analysis is never very exact.

2. Exact specified materials are difficult to acquire and may be drastically altered by placement methods.

3. The cost of such an installation, particularly engineering inspection and testing, is very high. If an error is made, the cost of correction may become prohibitive.

G. Nutrient Reduction (Nitrogen and Phosphorus)

1. General

A discussion on the importance of reduction of the amount nitrogen and phosphorus discharged to the environment via an OWRS is given in Section II. In the following, where computations of nitrogen dilution or phosphorus immobilization in the soil are made, the wastewater flow used in such computations should be the design average daily flow, rather than the design maximum day flow.

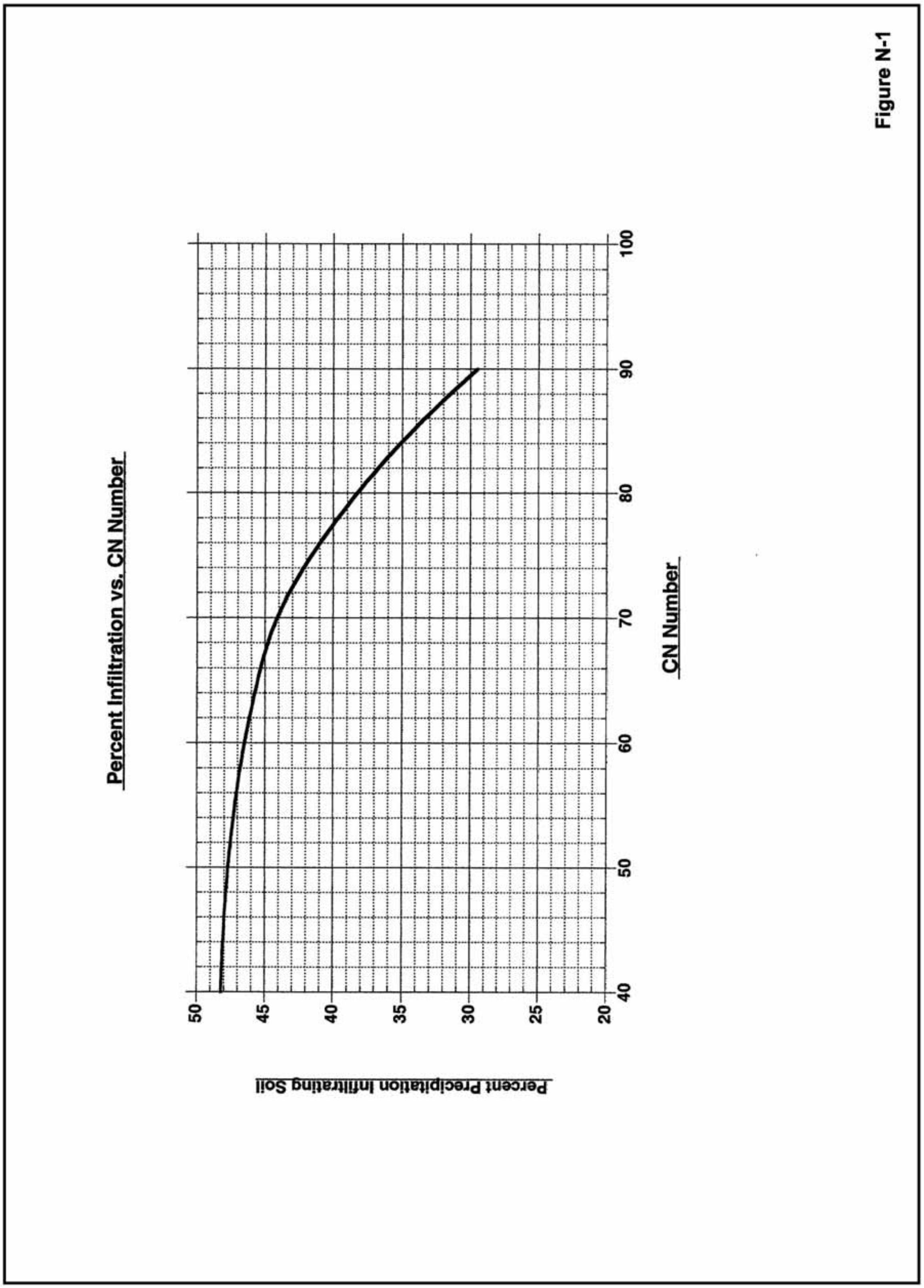
2. Nitrogen Dilution by Infiltrated Precipitation

The model used by the Department for nitrogen dilution by infiltrated precipitation, as presented in Healy and May (1982, rev. 1997) is retained in this document. However the methodologies for determining the amount of rainfall that infiltrates to the ground water, and the effective infiltration area, have been revised.

A study of available publications on water resources in Connecticut and rainfall-runoff relationships lead to adoption of a method for defining the percent of precipitation that infiltrates to the ground water under various soil conditions (Jacobson-2001). The results, given in graphical form in Figure No. N-1, permits determination of the percentage of infiltration based on the Runoff Curve Number (CN) method developed by the US S.C.S.(U.S.D.A.-1986).

The curve shown in Figure No. N-1 is intended to be used with a composite CN value computed for that portion of a project site that can logically be assumed to contribute infiltration for dilution of nitrogen discharged from a SWAS. The soil types and Hydrologic Soil Group classifications for soils at a project site can be obtained from maps and tables contained in the S.C.S. Soil Surveys for the various counties in Connecticut. The corresponding CN values can be obtained from Tables 2a-2c in the S.C.S./N.R.C.S. publication TR-55 (U.S.D.A.-1986). The procedures for computing a composite CN value for a project site are explained in TR-55, are familiar to most consulting engineers, and need not be given here.

Using the total lot area as the effective infiltration area, where the SWAS occupies only a small portion of the lot width, results in overestimating the affect of nitrogen dilution by infiltrated precipitation. After wastewater percolates downward from a SWAS to the ground water table, it generally flows as a plume in the local direction of ground water flow and gradually spreads transverse to the direction of the local ground water flow. The spreading of the nitrogen plume depends on the characteristics of the aquifer. When the lot width is substantially greater than the width of the SWAS, the spread may not be such that the plume covers the entire lot area, and therefore the total lot area should not be used as the effective infiltration area.


Percent Infiltration vs. CN Number
Percent Precipitation Infiltrating Soil
50
45
40
35
30
25
20
40
50
60
70
80
90
100
CN Number
Figure N-1

Mass transport processes determine the extent of plume spread and the geometric character of the contaminant concentration distribution (Domenico and Schwartz-1990). The principal processes responsible for the mass transport of chemicals dissolved in the ground water include advection, dispersion, and retardation. For non-reactive (conservative) chemicals, only the advection and dispersion processes are of concern. Based on the information previously discussed concerning the fate and transport of nitrogen (specifically, nitrates) in ground water, it can be considered as a non-reactive contaminant.

An elementary approach for modeling the effective dilution area was developed based on the concepts of hydrodynamic dispersion discussed in Freeze and Cherry (1979) and of contaminant transport in Domenico and Schwartz (1990). Domenico and Schwartz (1990) provide an analytical equation developed by Domenico and Robbins (1985) for advective and dispersive mass transport of a contaminant from a continuous finite planar source. A two-dimensional solution (vertical dispersion assumed negligible) was deemed reasonable for delineating the horizontal extent(boundary) of a nitrogen plume.

Therefore, the Domenico and Robbins equation was adjusted for a two-dimensional plume analysis (horizontal x and y directions) by eliminating the term for dispersion in the vertical direction as suggested in Domenico and Schwartz (1990). The analytical equation was solved for values of the horizontal perpendicular offset (y) from the plume centerline to the point on the plume boundary where the N concentration in the ground water is reduced from the initial concentration (C_o) in the percolating wastewater to a concentration (C)=10 mg/l (Jacobson-2001). Thus, within the plume boundary, the N concentrations vary from the initial concentration C_0 to a concentration of 10 mg/L, while outside of the plume boundary the concentration of N is less than 10 mg/L.

Tables were prepared to provide values of y, at various distances (designated as x) down-gradient from the SWAS, for various values of the initial concentration (C_o) of N in the wastewater percolating downward from a SWAS and for various lateral dimensions of the SWAS. Separate tables are provided for glacial till (Table No. N-1A) and stratified drift aquifers (Table No. N-1B). These tables can be used to determine the lateral extent of the effective infiltration area.

Figure No. N-2 presents an idealized view of the lateral extent of the plume concentration contour of 10 mg/l at a distance of x meters down-gradient of a SWAS, and indicates how the information obtained from Tables N-1A and N-1B can be used to determine the effective infiltration area.

It should be noted that, when the horizontal perpendicular offset (y) from the plume centerline to the point on the plume boundary where the N concentration is 10 mg/l, (for a given value of x from the SWAS to the Applicant's downgradient property line), indicates the plume boundary extends beyond a side boundary of the Applicant's property, it will be necessary to enter either Table N-1A or Table N-1B with the value of the shortest horizontal perpendicular offset (y) from the plume centerline to the nearest side boundary and solve for a revised distance x. It is this revised distance that should be used, together with the values for X_{swas} and X_u to determine the length of the effective infiltration area (See Figure N-2 for depiction of (y), (x), X_{swas} and X_u).

TABLE N-1A

Lateral Extent of 10 mg/L Nitrogen Plume in Glacial Till

y=Distance perpendicular to direction of ground water flow, from centerline of plume to plume C = 10 mg/L												
Co, mg/L	Y=100 Ft.						Y=200 Ft.					
	x=0	x=100	x=200	x=300	x=400	x=500	x=0	x=100	x=200	x=300	x=400	x=500
	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=
24	50	54	58	58	58	58	100	104	108	113	116	117
30	50	59	67	73	73	73	100	109	117	126	134	141
36	50	62	74	83	87	87	100	112	124	135	147	158
42	50	64	78	91	99	102	100	114	129	143	159	170
48	50	66	82	97	109	115	100	116	133	149	165	180
54	50	68	86	103	118	126	100	118	136	154	172	189
60	50	69	89	107	123	134	100	119	139	158	177	196
66	50	71	91	111	128	142	100	121	141	162	182	203
72	50	72	93	114	133	148	100	122	143	165	187	208

Co, mg/L	Y=300 Ft.						Y=400 Ft.					
	x=0	x=100	x=200	x=300	x=400	x=500	x=0	x=100	x=200	x=300	x=400	x=500
	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=
24	150	154	158	163	167	171	200	204	208	213	217	221
30	150	159	167	176	185	193	200	209	217	226	236	243
36	150	162	174	185	197	209	200	212	224	236	247	259
42	150	164	179	193	207	227	200	214	229	243	257	271
48	150	166	183	199	215	231	200	216	233	249	265	281
54	150	168	186	204	222	240	200	218	236	254	272	290
60	150	169	189	208	227	247	200	219	23	258	277	297
66	150	171	191	212	232	253	200	221	241	262	282	303
72	150	172	193	215	237	259	200	222	243	265	287	309

Co, mg/L	Y=500 Ft.						Y=600 Ft.					
	x=0	x=100	x=200	x=300	x=400	x=500	x=0	x=100	x=200	x=300	x=400	x=500
	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=
24	250	254	258	262	267	271	300	304	308	313	317	321
30	250	258	267	276	285	293	300	308	317	326	335	343
36	250	262	274	285	297	309	300	312	324	335	347	359
42	250	264	279	293	307	321	300	315	329	343	357	371
48	250	267	283	299	315	331	300	317	333	349	365	381
54	250	268	286	308	322	340	300	318	336	354	372	390
60	250	269	289	308	327	347	300	319	339	358	377	397
66	250	270	291	312	332	353	300	320	341	362	382	403
72	250	271	293	315	337	359	300	321	343	365	387	409

Notes:

1. C_o = Nitrogen concentration in discharge from SWAS.
2. x = longitudinal horizontal distance from SWAS to point of concern, measured parallel to the local direction of ground water flow.
3. Y = horizontal dimension of SWAS measured perpendicular to the local direction of ground water flow.
4. For intermediate values of C_o, Y and y, interpolate from tables.
5. Refer to Figure N-2 for depiction of x, Y, and y.

TABLE N -1B

Lateral Extent of 10 mg/L Nitrogen Plume in Stratified Drift

y=Distance perpendicular to direction of ground water flow, from centerline of plume to plume C = 10 mg/L												
Co,	Y=100 Ft.						Y=200 Ft.					
mg/L	x=0	x=100	x=200	x=300	x=400	x=500	x=0	x=100	x=200	x=300	x=400	x=500
	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=
24	50	52	54	56	58	59	100	102	104	106	108	111
30	50	54	59	63	67	70	100	104	109	113	117	122
36	50	56	62	68	74	79	100	106	112	118	124	130
42	50	57	64	71	78	85	100	107	114	121	129	136
48	50	58	66	74	82	90	100	108	116	124	133	141
54	50	59	68	77	86	94	100	109	118	127	136	145
60	50	60	69	79	89	98	100	110	119	129	139	148
66	50	60	71	81	91	101	100	110	121	131	141	152
72	50	61	72	83	93	104	100	111	122	133	143	154

Co,	Y=300 Ft.						Y=400 Ft.					
mg/L	x=0	x=100	x=200	x=300	x=400	x=500	x=0	x=100	x=200	x=300	x=400	x=500
	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=
24	150	152	154	156	158	161	200	202	204	206	208	211
30	150	154	159	163	167	172	200	204	209	213	217	222
36	150	156	162	168	174	180	200	206	212	218	224	230
42	150	157	164	171	179	186	200	207	214	221	229	236
48	150	158	166	174	183	191	200	208	216	224	233	241
54	150	159	168	177	186	195	200	209	218	227	236	245
60	150	160	169	179	189	198	200	210	219	229	239	248
66	150	160	171	181	191	202	200	210	221	231	241	252
72	150	161	172	183	193	204	200	211	222	233	243	254

Co,	Y=500 Ft.						Y=600 Ft.					
mg/L	x=0	x=100	x=200	x=300	x=400	x=500	x=0	x=100	x=200	x=300	x=400	x=500
	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=	y=
24	250	252	254	256	258	261	300	302	304	306	308	311
30	250	254	259	263	267	272	300	304	309	313	317	322
36	250	256	262	268	274	280	300	306	312	318	324	330
42	250	257	264	271	279	286	300	307	314	321	329	336
48	250	258	266	274	283	291	300	308	316	324	333	341
54	250	259	268	277	286	295	300	309	318	327	336	345
60	250	260	269	279	289	298	300	310	319	329	339	348
66	250	260	271	281	291	302	300	311	321	331	341	351
72	250	261	272	283	293	304	300	311	322	333	343	354

Notes:

1. C_o = Nitrogen Concentration in discharge from SWAS.
2. x = longitudinal horizontal distance from SWAS to point of concern, measured parallel to local direction of ground water flow.
3. Y = horizontal dimension of SWAS measured perpendicular to the local direction of ground water flow.
4. For intermediate values of Co, Y and y, interpolate from tables.
5. Refer to Figure N-2 for depiction of x, Y, and y.

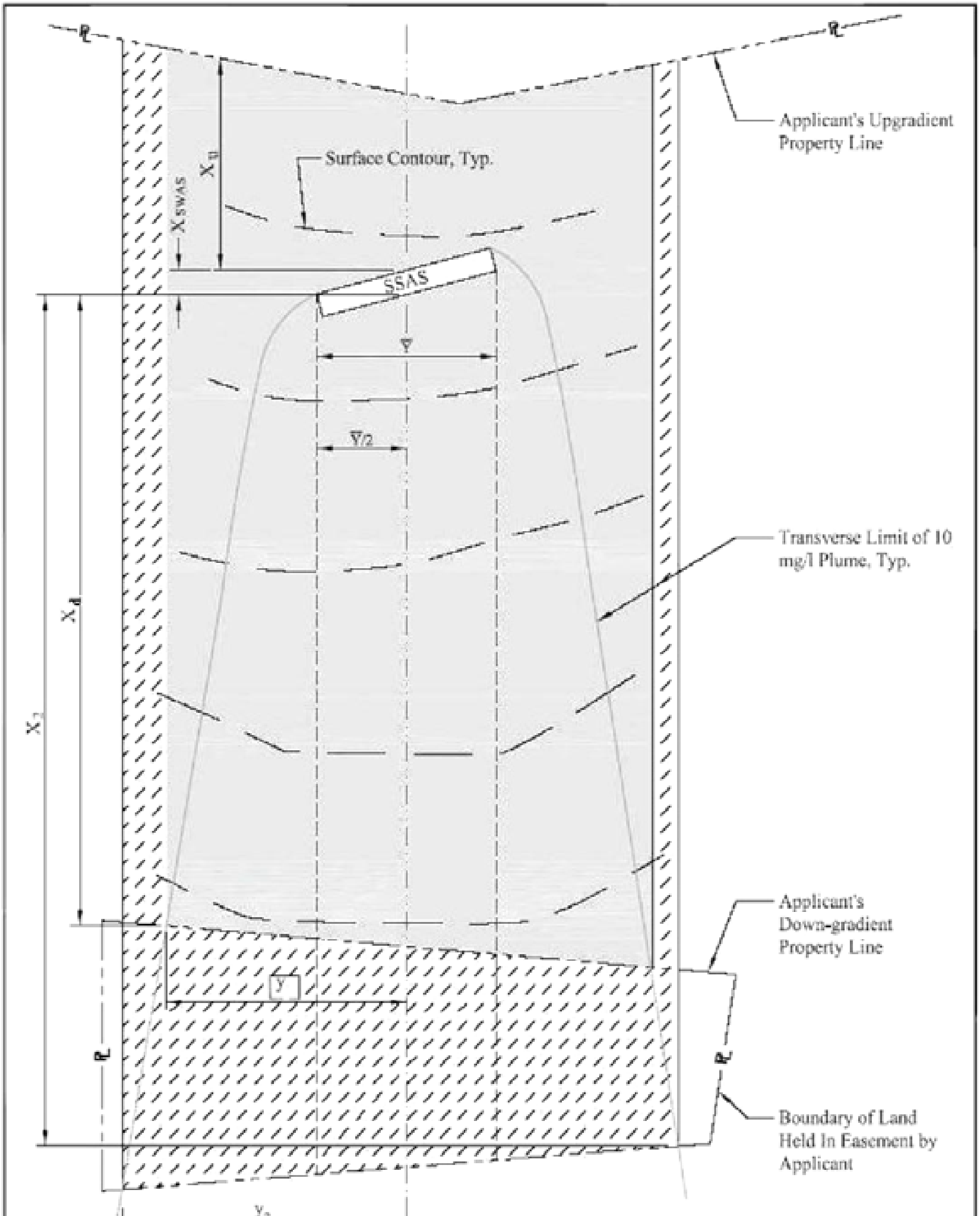


FIGURE N-2
IDEALIZED SKETCH OF EFFECTIVE DILUTION AREA FOR NITROGEN PLUMES

Nitrogen Dilution Model

The mathematical expression of the nitrogen dilution model used by the Department is as follows:

$N_{gw} = [(Q_{ww} \times N_{ww}) /(Q_{ww} + Q_{ip})]$,

where:

N_{gw} = nitrogen concentration in ground water at the point of concern, [M/V]
Q_{ww} = daily design volume of wastewater, $[L^3]$
N_{ww} = nitrogen concentration in the wastewater reaching the ground water,
= 60% of the raw wastewater total nitrogen concentration, [M/V]
Q_{ip} = daily volume of infiltrated precipitation, $[L^3]$

Also, Qip = %I x A_e /100 where %I = percent infiltration, from Figure N-1, and A_e= effective infiltration area, = $(X_d + X_u + X_{SWAS})(2y)$, $[L^2]$

As shown on Figure N-2,

X_d = longitudinal horizontal distance from the downgradient side of the SWAS to the down gradient point of concern, measured parallel to the local direction of ground water flow [L]

X_u = longitudinal horizontal distance from the up-gradient side of the SWAS to the up gradient property line, measured parallel to the local direction of ground water flow [L]

X_{SWAS} = horizontal width of SWAS, measured parallel to the local direction of ground water flow [L]

y = horizontal transverse distance from the point of concern on the longitudinal centerline of nitrogen plume to the plume concentration contour = 10 mg/l nitrogen, measured perpendicular to direction of local ground water flow, obtained from Tables No. N-1A or Table N-1B (by interpolation if necessary) [L]

Y = horizontal transverse width of SWAS, measured perpendicular to direction of local ground water flow [L]

An example of the use of the model equation follows.

A design average daily flow of 5,000 gallons of wastewater discharged from a school is to be discharged from a SWAS to a glacial till aquifer. The raw wastewater has a total nitrogen concentration of 80 mg/l. There is sufficient depth of unsaturated soil to permit installation of the SWAS in the existing soil while still maintaining the required separating distance between the bottom of the SWAS and the mounded ground water.

The width of the SWAS measured perpendicular to the direction of the local ground water gradient = 256 ft and the SWAS is located 164 ft from the applicant’s up-gradient property line. The dimension of the SWAS parallel to the direction of the local ground water gradient = 46 ft The distance from the SWAS to the closest down gradient point of concern, measured parallel to the direction of the local ground water gradient, = 400 ft The composite SCS Curve Number (CN) for the soil in the area of the proposed SWAS = 72. Annual average precipitation = 48 inches (equivalent to 0.13 inches/day).

a. From Figure No. N-1, for a CN value of 72, the percent of precipitation infiltrating to the ground water = 43%. (Stated another way, the decimal fraction of total precipitation infiltrating to the ground water = 0.43)

b. The total nitrogen concentration in the wastewater discharged from the SWAS (Co), Nww, = 0.6 x 80 mg/l = 48 mg/L.

c. From Table No. N-1A (for glacial till aquifers), for Co = 48 mg/l, Y = 256 ft. and x = 400 ft, y = 193 ft (by interpolation between Y= 200 ft. and 300 ft. Therefore, Ae, the effective infiltration area, = (2 x 193) x (164+ 46 + 400) ft = 235,400 sq. ft, or 21,870 sq. meters.

d. Q_{ip}, the annual daily volume of infiltrated precipitation, = 0.43 x 0.13 in/day x 2.54 cm/inch x (1 m/100 cm) x 21,870 sq. meters = 0.43 x 0.003 meters/d x 21,870 sq. meters =31.1 cu. meters x 1000 liters/cu. meter = 31,100 liters/d.

e. Q_{ww} = 5,000 gal/d x 3.785 liters/gal = 18,925 liters/d.

f. Q_{ww} x N_{ww} = [18,925 liters/d x 48 mg/l] = 908,400 mg/d

g. Q_{ww} + Q_{ip} = [18,925 liters/d +31,100 liters]/d = 50,025 liters/d.

N_{gw} = [(Q_{ww} x N_{ww}) / (Q_{ww} + Q_{ip})] = 908,400 mg/d/50,025 l/d = 18.2 mg/l. Since this concentration > 10 mg/l, additional pretreatment will be necessary as the nitrate nitrogen will not be sufficiently diluted by infiltrated precipitation. As alternatives, the width of the SWAS could be increased to increase the nitrogen dilution area; or, if that was not possible, additional land that would contribute to nitrogen dilution could be acquired by purchase or easement.

The nitrogen dilution model equation can also be re-arranged to solve for the reduction in N_{ww} required to be obtained by additional pretreatment in order to meet the requirement that $N_{gw} \leq 10$ mg/l. In this case, the equation takes the following form:

$$\text{Maximum allowable } N_{ww} = 10[(Q_{ww}+Q_{ip})/Q_{ww}].$$

In the example just given, the maximum allowable N_{ww}=10 x [(18,925 liters/d + 31,100 liters/d) /18,925 liters/d] = 26.4 mg/l. Thus, additional pretreatment would be required to reduce the total nitrogen in the wastewater discharged to the SSDS from 48 mg/l to 26.4 mg/l.

3. Additional Pretreatment for Nitrogen Removal

Physical/chemical processes and biological processes can be used for nitrogen removal. However, physical/chemical processes are not considered to be suitable for on-site wastewater renovation systems because of the cost of such processes, the operational problems inherent in such processes, and the need for highly skilled operation. In fact, while physical/chemical processes were once considered to be attractive for nitrogen removal at municipal wastewater treatment facilities, they have largely been abandoned in favor of biological processes.

Biological nitrogen removal is a two-step process involving nitrification and de-nitrification. As previously discussed in Section II of this document, nitrification is the biological oxidation of ammonium (NH4+) to nitrate (NO3-), and de-nitrification is the biological reduction of NO3- to nitrogen gas.

There are two basic types of wastewater treatment systems used in the biological nitrogen removal process. One type consists of the suspended growth system, in which the microorganisms that remove the impurities from the wastewater are maintained in suspension in intimate contact with the wastewater to be treated. The other consists of the fixed film system, in which the microorganisms are attached to some type of media, with the wastewater either passing through the media or the media passing through the wastewater. There are also hybrid systems that combine both suspended growth and fixed film processes.

The Department has approved several types of facilities that employ either suspended growth or fixed film processes, or hybrid processes, for pretreatment. Further discussion on enhanced pretreatment for nitrogen removal, including requirements for design, construction, operation, and maintenance, is given in Enhanced Pretreatment, Section XI of this document.

4. Phosphorus Removal

The model used by the Department for removal of phosphorus (P) in the percolate from a SWAS assumes that 30% of the P is removed in the septic tank and in the biomat that forms at the SWAS-soil interface. The remainder must be removed in the soil beneath the SWAS.

Studies have indicated that very limited P transport to ground water occurs in aerobic, water-unsaturated soils of suitable texture and chemical characteristics. In most soils in which Fe, Al and Ca are present in reactive form, aerobic conditions exist, and flow rates are minimal, P movement is minimal and pollution of ground and surface waters from P applied in a SWAS is considered unlikely. In recent extensive field studies, the evidence suggested that P removal in the subsurface is influenced by mineral precipitation reactions in the unsaturated zone which tend to be irreversible

On the other hand, while some P may be removed in the saturated (ground water) zone beneath and down-gradient of the SWAS there is potential for the migration of P in the saturated zone under certain conditions. P removal in the ground water zone appears to be dominated by sorption reactions that are readily reversible (Robertson and Harman-1999). P has been detected above background levels in ground water adjacent to and down-gradient of subsurface wastewater absorption systems under conditions of saturated flow, high water tables, or high hydraulic loading rates (Reneau-1979).

Therefore, absent any enhanced pretreatment for P removal, it should be demonstrated that the P in the percolate from a SWAS will be removed in the unsaturated soil zone beneath the SWAS.

The Department model assumes that P removed in the unsaturated zone is initially sorbed onto active soil particles, but that over a 6 month period, the sorbed P will combine with Fe, Al or Ca in the soil to form less soluble precipitates. As the precipitates form, the original sorption sites are regenerated. It should be demonstrated that the unsaturated soil beneath a SWAS has the capacity to sorb at least 6 months of the P in the percolate from the SWAS. Therefore, it is necessary to determine the P sorption capacity of the unsaturated soils below the SWAS area and the total mass of soil that the percolate from a SWAS will contact as it moves downward through the unsaturated zone.

Test procedures are available to conservatively estimate the P sorption capacity. Such tests should be conducted for existing relatively coarse textured soils (e.g.: sands and gravelly sands) and for soils proposed to be used in fill systems in which a SWAS is proposed to be constructed. P-Sorption tests are recommended because reliance on published data on P sorption may prove to be problematic and require unanticipated future retrofitting of an OWRS with enhanced pretreatment for P removal should the P sorption capacity be exhausted and travel of P in the subsurface become significant.

The phosphorus sorption test should conform to the procedure “Phosphorus Sorption Isotherm Determination” (Graetz, and Nair - 2000) included in the Appendices (Appendix F). unless otherwise approved by the Department.

Where the existing unsaturated soils beneath a SWAS have a low P sorption capacity, it will be necessary to limit the rate at which P is applied to the soil. Limiting the P application rate involves adjusting the infiltrative surface P loading rate to that of the long-term P sorption rate of the soil. This can be accomplished by adjusting the infiltrative surface hydraulic loading rate. If this is not feasible, there are two options that can be considered for the selected site. Suitable fill can be placed in the SWAS area above the existing unsaturated zone to provide additional thickness (mass) of unsaturated soil in order to meet the 6-month P sorption requirement. If the capability of the soils for long-term immobilization of P is problematic and placing additional suitable fill is not feasible, enhanced pretreatment for the removal of P from the wastewater can be provided. Further discussion on such pretreatment is given in Section XI (Enhanced Pretreatment) of this document.

The following assumptions are made in estimating the P removal capabilities of the unsaturated soil beneath a SWAS.

1) The effective horizontal area through which the percolate from the SWAS flows is equal to the bottom width and the effective sidewall heights of the leaching units. This assumes that the flow through the effective sidewall area is dispersed over a horizontal area, located in the same horizontal plane as the bottom area, equal to the unfolded effective sidewall area, and that there is no dispersion of the percolate beyond the effective horizontal area.

2) The percolate from the SWAS flows vertically downward through the finer soil pores, and in thin films over some of the soil particle surfaces in the larger soil pores due to the affinity of water to the surface of soil solids. It is assumed that this results in approximately 50% of the soil in the unsaturated zone being wetted.

3) The P-sorption capacity of a soil, in milligrams P per 100 grams (dry weight) of soil, has been determined on the basis of P-sorption tests on representative samples of the soil beneath the SWAS and comparison with published capacity values of similar soils.

4) The dry unit weight of the soil beneath the SWAS is known.

5) The unsaturated soil beneath the effective horizontal area of the SWAS must be such as to adsorb at least 6 months of the P in the percolate from the SWAS.

An example of calculations used to determine the suitability of a site for P removal is given below.

The design average daily flow is 6,000 gpd. This flow will be discharged via a low-pressure distribution system to a leaching system consisting of 866 lf of leaching gallery units. The leaching gallery units will have a bottom width of 6 ft., including one ft. of broken stone on each side of the gallery units. The effective sidewall height = 1 ft. Thus, the total equivalent horizontal area = 866 lf x (6+2) ft/lf. = 6928 sq. ft. The depth of unsaturated soil is 3 ft. The effluent P concentration in the SWAS percolate is estimated to be 9 mg/L, based on sampling of septic tank effluent from similar facilities. Based on the results of P sorption tests of the soil beneath the SWAS and a review of relevant literature, a P sorption value of 8 mg P /100 grams of soil has been selected.

The total PO_4-P discharged to the unsaturated leaching material each day = 6,000 gpd x 3.785 liters/gal x 9 mg/L = 204,390 mg. Thus, the total P discharged over a one month period = 30.4 days per average month x 204,390 mg /day = 6.2×10^6 mg P. The unsaturated leaching material has an average dry unit weight of 105 lb./cu. ft at 90% of maximum density. 105 lb./cu. ft. x 454 gm/lb. = 47,700 gm./cu. ft. The mass of soil over which the P-laden water will flow = 0.5 x 6928 sq. ft x 3 ft of depth x 47,700 gm./cu. ft. = 4.95×10^8 grams.

The total sorption capacity of this soil = 8 mg /100 grams soil x 4.95×10^8 grams = 3.96×10^7 mg. P.

Thus, the unsaturated leaching fill can sorb 3.96×10^7 mg P/6.2×10^6 mg P/month = 6.4 months of P in the percolate from the SWAS. The site appears to be satisfactory with respect to P removal.

H. Flow Equalization

Where water use varies widely on a daily basis, it is often cost-effective to design large scale on-site wastewater renovation systems on the basis of a uniform flow rate and provide some equalization storage to even out the daily variations in flow rate. The cost of providing equalization storage is often a small fraction of the additional cost for providing a subsurface soil absorption system to accommodate higher flow rates. There is a secondary benefit to using equalization storage in that it tends to dampen the variations in wastewater constituent concentrations. Thus, flow equalization allows downstream processes to operate at more uniform flow rates and contaminant loadings, and this is beneficial to the operating stability and efficiency of these processes.

Where enhanced pretreatment facilities are needed, flow equalization will allow designing these facilities for the equalized rather than peak flows, thus reducing the size and cost of these facilities.

Flow equalization facilities can be designed to function on either an in-stream or off-stream basis. In the in-stream case, all flow passes through the equalization basin. In the off-stream case, only the flow that exceeds the desired uniform flow rate is diverted to the equalization facilities. The in-stream case is more beneficial because it is more effective in dampening the variations in wastewater constituents, and should normally be used.

While flow equalization can be used to equalize hourly flows, and this approach is sometimes used for large wastewater treatment facilities, a reasonably conservative approach for on-site facilities is to use the average daily water use during the maximum month as the uniform flow rate.

Determination of the equalization storage volume required for a selected uniform flow rate can be made by analyzing mass curves of the actual daily wastewater flows to be received by the on-site facilities over selected periods of time. In many cases, information will not be available for wastewater flows, and thus water use data as determined from daily water meter readings must be used. In the latter case, care should be exercised to avoid use of data that includes water used outdoors and for other consumptive uses that will not be discharged to the on-site system.

A mass curve of daily wastewater flows is simply a plot of the cumulative daily flows over a selected period of time. In order to assess the yearly variations in daily flows, an initial analysis of cumulative flows over a two to three year period should be performed. This is accomplished by tabulating values of the accumulated flows and elapsed days and then preparing a graphical mass curve by plotting the accumulated flows on the vertical axis vs. elapsed days for the period being investigated on the horizontal axis.

A sloping line is then plotted on the graph with the slope of the line being equal to the desired uniform daily flow rate. Lines parallel to the uniform flow rate line are then plotted so as to be tangent to the upper and lower extremities of the mass flow curve. The required equalization volume is the maximum vertical distance between these two lines.

The scale of the mass curve graph is often such that an accurate value of the maximum vertical distance is not possible. Therefore, the cumulative flow data for that portion of the mass flow curve in the area of the maximum vertical distance can be analyzed mathematically to validate the required equalization storage volume determined graphically.

Use of the mass curve and mathematical methods of determining flow equalization storage are given in the following example. While this example is for a design wastewater flow rate less than 5,000 gpd (the threshold for a large-scale OWRS), it will serve to indicate the procedure for determining flow equalization storage. In this example, no wastewater flow data were available and thus water meter readings were utilized, after determining that essentially all water used would be used for sanitary purposes and would be discharged to the on-site system.

Inspection of the tabulated water meter readings indicated that the period from June 2000 to May 2001 represented the highest annual water use for over a 2-year period. The average daily water use, maximum daily water use, and the maximum 7-Consecutive Day Moving Average (7CDMA) of water use for that period were determined, as shown in Table 1.

<u>TABLE EQ -1</u>

WATER USE DATA

Month	Average Daily Water Use, Gals.	Maximum Daily Water Use, Gals.	Max. Value 7CDMA, Gals.
June 2000	1200	2800	1943
July 2000	1952	2700	2157
Aug. 2000	2252	4700	2871
Sept. 2000	1103	3800	1829
Oct. 2000	564	1100	90
Nov. 2000	147	367	328
Dec. 2000	127	700	300
Jan. 2001	295	2900	724
Feb. 2001	200	1134	657
Mar. 2001	206	600	343
April 2001	307	1000	529
` May 2001	948	2700	1162

The information shown in Table EQ-1 indicated that highest period of water use occurred during the period from June through September 2000, the maximum monthly water use occurred in August 2000, and the average daily flow during that month was 2,252 gpd. Table EQ -1 also indicated that the maximum value in the 7CDMA also occurred in August 2000. Accordingly, a mass curve spanning the period from July through September was prepared and is shown in Figure EQ-1. The slope of the lines of tangency = 2,252 gpd, the average daily water use during the maximum month. From this curve, it was determined that the maximum vertical distance between the lines of tangency was found to be equivalent to 6,500± gallons. A storage volume of 6750 gallons was assumed for a detailed analysis. A tabulation of the water use data from August 13, 2000 through September 14, 2000 was then prepared (Table EQ-2).

Starting on a date when the mass curve was tangent to the lower line of tangency (no equalization storage required) the average daily flow during the maximum month (2,252 gpd) was subtracted from the daily water use values in Table EQ-2 to arrive at the volume to be stored. For each day, the volume to be stored was subtracted from the total equalization storage volume determined from analysis of the mass curve. The results indicated that on August 13th the equalization storage tank was empty and by August 25th, the tank was essentially full. However, on August 26th, the tank began to empty and by September 4th it was again empty and after September 8th it remained empty during the remainder of the period of interest. The maximum required storage volume was computed to be 6,750 gal - 122 gal = 6628 gal. Thus, the approximate equalization storage volume determined graphically was validated mathematically.

The wastewater in the flow equalization tank must be pumped to the downstream facilities at a daily rate of 2252 gals. Thus, the pumping capacity must be equal to 2,252 gallons divided by the pump running time. Assuming the active pump would deliver four doses per day to the pressure distribution system, the average dose would be 2,252/4 = 563 gallons. Assuming a volume of 50 gallons drains back from the flow distribution system when the pump stops, the total volume per dose would be 613 gallons.

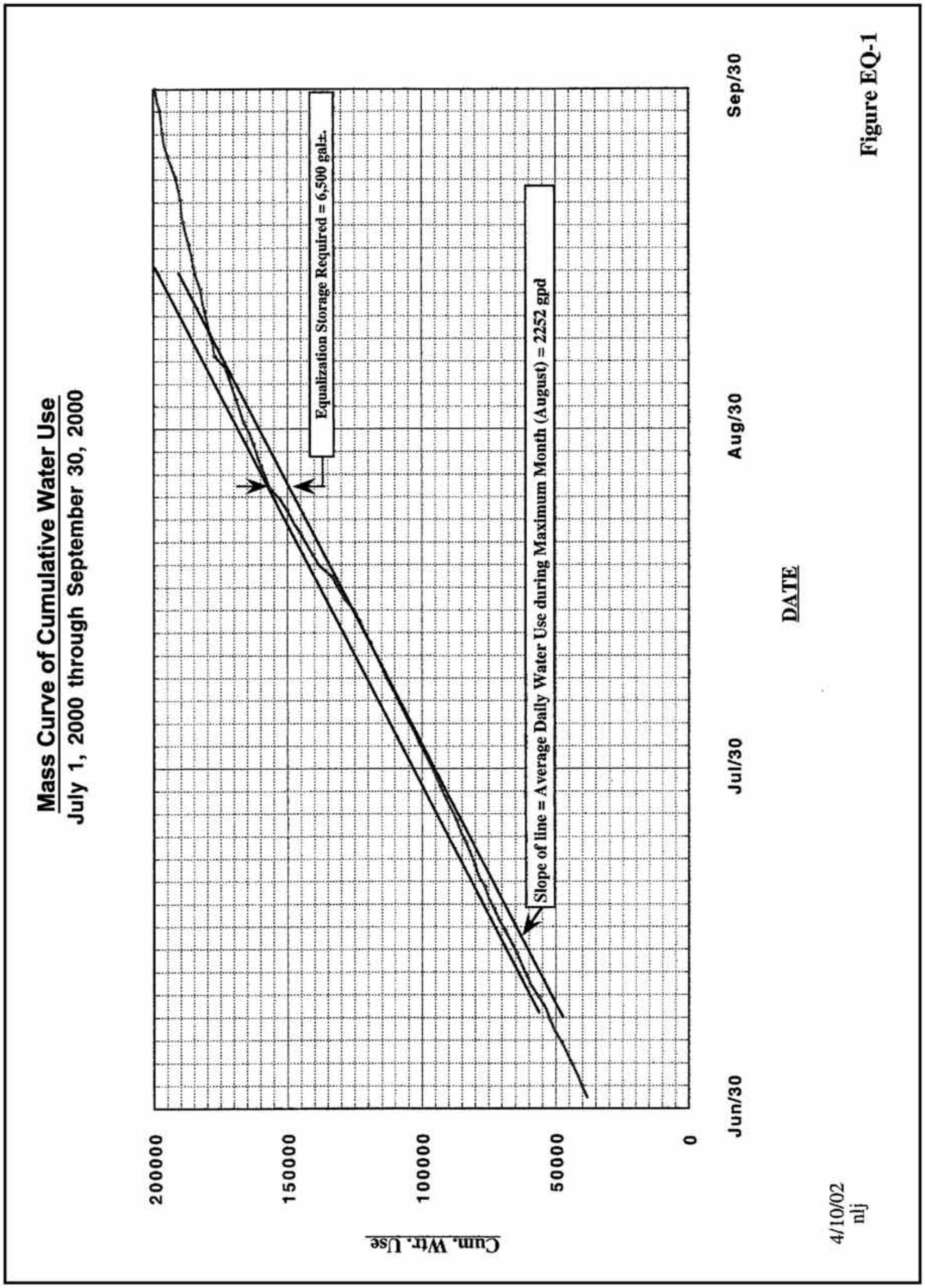

Mass Curve of Cumulative Water Use
July 1, 2000 through September 30, 2000
Equalization Storage Required = 6,500 gal±.
Slope of line = Average Daily Water Use during Maximum Month (August) = 2252 gpd
Cum. Wtr. Use
200000
150000
100000
50000
0
Jun/30
Jul/30
Aug/30
Sep/30
DATE
4/10/02
nlj

Figure EQ-1

TABLE EQ-2

Calculations to Validate Volume of Equalization Tankage Determined by Graphical Method (6750 Gal.)

Date	Water Use GPD	Cum. Wtr. Use, Gal.	7CDMA GPD	Vol. To be Stored, Gal.	Storage Remaining, Gal	
8/13/00					6,750	Tank Empty
8/14/00	2,033	125100	2,014	0	6,750	
8/15/00	3,500	128600	2,186	1,248	5,502	
8/16/00	2,500	131100	2,286	248	5,254	
8/17/00	2,200	133300	2,329	-52	5,306	
8/18/00	4,700	138000	2,714	2,448	2,858	
8/19/00	2,450	140450	2,774	198	2,660	
8/20/00	2,450	142900	2,833	198	2,462	
8/21/00	2,300	145200	2,871	48	2,414	
8/22/00	2,600	147800	2,743	348	2,066	
8/23/00	2,200	150000	2,700	-52	2,118	
8/24/00	2,700	152700	2,771	448	1,670	
8/25/00	3,800	156500	2,643	1,548	122	Tank Essentially Full
8/26/00	1,600	158100	2,521	-652	774	
8/27/00	1,600	159700	2,400	-652	1,426	
8/28/00	1,600	161300	2,300	-652	2,078	
8/29/00	1,400	162700	2,129	-852	2,930	
8/30/00	1,500	164200	2,029	-752	3,682	
8/31/00	2,100	166300	1,943	-152	3,834	
9/1/00	1,200	167500	1,571	-1,052	4,886	
9/2/00	1,425	168925	1,546	-827	5,713	
9/3/00	1,425	170350	1,521	-827	6,540	
9/4/00	1,425	171775	1,496	-827	6,750	Tank Empty
9/5/00	1,425	173200	1,500	-827	6,750	
9/6/00	3,800	177000	1,829	1,548	5,202	
9/7/00	800	177800	1,643	-1,452	6,654	
9/8/00	1,100	178900	1,629	-1,152	6,750	Tank Empty
9/9/00	967	179867	1,563	-1,285	6,750	
9/10/00	967	180834	1,498	-1,285	6,750	
9/11/00	966	181800	1,432	-1,286	6,750	
9/12/00	600	182400	1,314	-1,652	6,750	
9/13/00	1,600	184000	1,000	-652	6,750	
9/14/00	1,100	185100	1,043	-1,152	6,750	

The pump discharge rate would be such as to maintain a flow velocity of at least 2 ft/sec in the force main and pressure distribution manifold. For example, for a 4-inch dia. manifold, the nominal cross-sectional area would be 0.087 sq. ft. At 2 ft/sec, the flow would be 0.174 cu ft/sec, which is equivalent to ~ 78 gal/min. and the dosing time would be 613 gal/78 gal/min = ~ 8 min.

The final liquid capacity of the flow equalization tank must be greater than the required equalization volume. The additional capacity is required for the drain-back volume, the volume required to submerge the pump volute when the liquid level reaches the lowest level in the tank, and some ventilation space between the high liquid level and the inside top of the equalization tank. Additional capacity is also needed to provide emergency storage should events occur that would not permit normal operations. Additional discussion on these requirements is covered under Pump Chambers elsewhere in this document.

I. Flow Distribution

1. General

The basic objective of flow distribution is to uniformly distribute septic tank effluent to the infiltrative surfaces of the leaching system to permit full utilization of the renovative capacity of the soil. There is considerable debate as to whether the distribution should be by means of gravity flow to the various units of the leaching system, or by means of a pressure distribution system (PDS). In the latter case, this would require delivery of septic tank effluent under pressure to the PDS.

The proponents of gravity flow distribution postulate that once a mature biomat is developed at the infiltrative surfaces of the leaching system, the flow-restricting nature of the biomat will cause ponding within the system and thus even distribution will occur naturally. Another reasonable argument is to “keep it simple”. Pumping stations (or dosing siphons where permitted) used for pressure distribution can and do malfunction, require periodic inspection and maintenance efforts above that required by a gravity flow system, and, until overt failure of the system occurs, these requirements may be largely ignored by the owners of small systems. Thus, the additional construction and maintenance costs for a PDS cannot be justified for small systems serving individual residences and other facilities with similar wastewater flows.

The proponents of pressure distribution counter with the following arguments. Given the hydraulic conditions that exist in the usual gravity flow distribution system, it is probable that the septic tank effluent will not be uniformly distributed to the various leaching units that constitute the leaching system, let alone be evenly distributed within each individual unit. They also point out that it can take a considerable period of time (measured in months) for a mature biomat to develop. In the rows of trenches, galleries, or chambers, until a mature biomat develops, gravity flow distribution will result in the septic tank effluent being discharged to the soil within a short distance of the inlet end of the leaching system units.

Thus, the loading rate on the infiltrative surface in this localized area will be substantially greater than the design leaching surface application rate, resulting in overtaxing of the soil's renovative capacity and contamination of the ground water.

Further, a much heavier biomat will develop, beginning at the entrance to a leaching trench, gallery, or chamber. As this heavier mat develops, the flow through the infiltrative surface at the entrance will be severely restricted, and the flow will then be distributed to another localized area, with the same result, and so on, until the distal (far) end of the leaching facility is reached. This is sometimes referred to as "creeping failure" if the heavy biomat causes severe ponding, resulting in backups in the sanitary waste plumbing facilities or surfacing of the septic tank effluent (overt failure).

Most proponents of pressure distribution systems will acknowledge that completely uniform flow distribution is not obtained by such systems. This is because the distribution piping contains holes usually spaced several feet apart, and the areas between the holes do not get evenly dosed until a mature biomat develops in such systems. However, a PDS approaches a reasonable uniform distribution and is reputed to mitigate the development of an excessively heavy biomat.

2. Gravity Flow Distribution

There are devices available (e.g. tipping buckets, flow control orifices and weirs, etc.) to aid in equalizing the flow to the various pipes that make up a gravity flow type of SWAS. However, while these may assist in equalizing the flow to each gravity flow distribution pipe, they do little to distribute the flow uniformly along the length of each pipe, particularly for long lengths of pipe that are usually found in large scale on-site wastewater renovation systems.

3. Automatic Dosing Siphons

Automatic dosing siphons have had a long history of use for distributing pretreated wastewater to subsurface wastewater absorption systems. Burks and Minnis (1994) state: "Siphons are used because they require no energy and, in theory, work indefinitely if they are properly installed and maintained. In practice, however, siphons may fail because they leak or become plugged. Pumps provide a more reliable dosing method." Siphons may be useful for intermittent dosing of a SWAS that is designed for gravity flow, but are not suitable for low pressure distribution systems because of the low discharge head (generally not greater than three ft.) developed by a siphon. In certain instances, where the siphon chamber is located at an elevation significantly higher than the SWAS, sufficient elevation head may be available.

While siphon chambers have been used in the past where the elevation difference between the siphon chamber and the pressure distribution piping system was sufficient, they cannot match the performance that a modern pumping system can deliver.

4. Low Pressure Distribution

Low-pressure distribution of wastewater to a SWAS is desirable for all on-site systems subject to the jurisdiction of the Department. It is recommended that serious consideration be given to the use of low-pressure distribution where the SWAS will be installed in sands and when the design flow is greater than 1,500 gpd. (Recall that all on-site systems located on property owned by the applicant that in the aggregate have a total design flow > 5,000 gpd fall under the jurisdiction of the Department.).

Low-pressure distribution of wastewater to a SWAS should be used in cases where:

- The SWAS is situated in coarse grained soils (K classified as very rapid by the NCRS).
- Systems include enhanced pretreatment.
- Systems are predominantly used on a seasonal basis (i.e. during a particular time of the year, rather than on a continuous year-round basis).

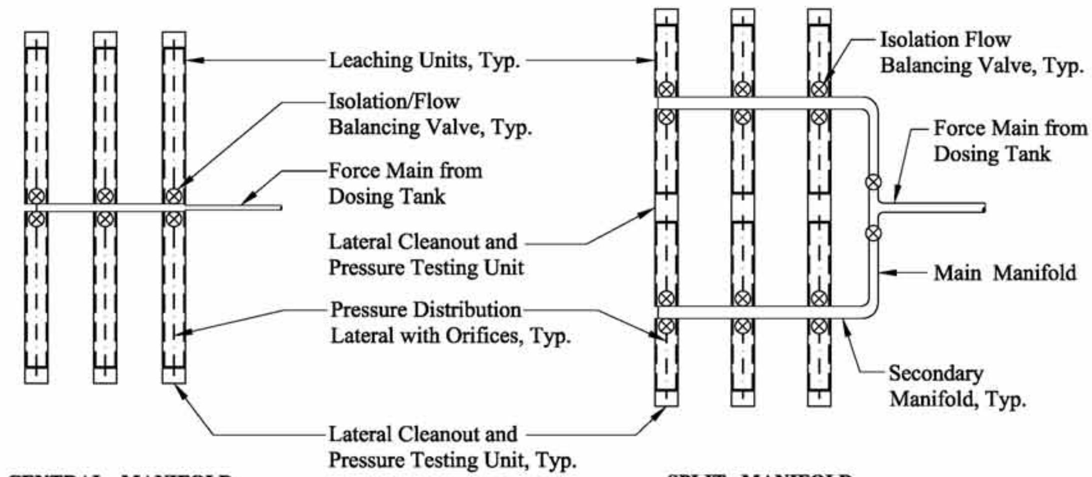
Low pressure distribution is necessary in such cases to reasonably equalize distribution of the effluent over the entire infiltrative surfaces of the SWAS so as to maximize the use of ability of the soil to renovate the pretreated effluent. It will also assist in maintaining aerobic conditions in the unsaturated zone beneath the SWAS. Uneven distribution of the pretreated effluent can result in localized overloading of the soil, leading to anaerobic conditions, short-circuiting through the soil, and localized reduction or elimination of the unsaturated soil zone between the bottom of the SWAS and the seasonal high ground water table. This soil must remain unsaturated and aerobic in order to maximize renovation of the wastewater.

Uniform distribution is achieved using a pressure distribution system (PDS) that consists of a dosing tank equipped with centrifugal pumps and associated controls, a force main, a distribution manifold, and pressure distribution laterals (PDLs). The force main delivers the dosed flow from the dosing tank to the manifold, which is designed to provide essentially equal distribution of flow to the PDLs that are connected to the manifold. The PDLs in turn are designed to provide essentially equal distribution of flow along their lengths via orifice holes drilled in the laterals. There are several configurations that can be used for a PDS, as shown in Figure PDS-1. The configuration to be used will depend on local site conditions, the size of the system and the preference of the designer.

Design and construction of pressure distribution systems should conform to the following criteria and requirements.

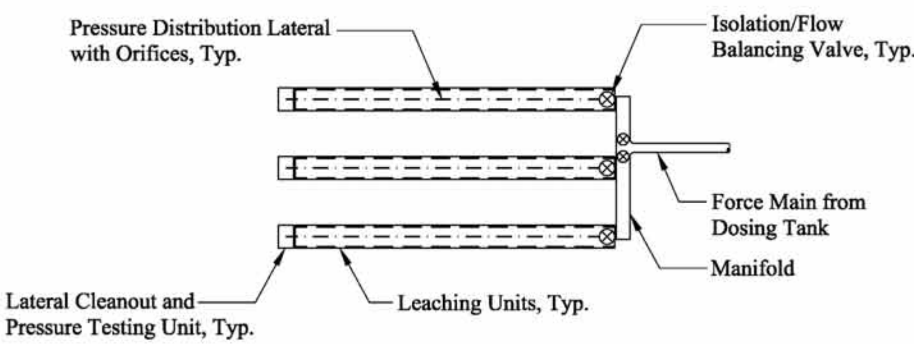
1) PDLs may be installed in plastic chambers, precast concrete galleries, stone filled trenches or stone leaching beds. However, stone leaching beds should only be used where enhanced pretreatment is provided.

2) Where septic tank effluent is being distributed, consideration should be given to using plastic chambers or rock-filled trenches, rather than precast concrete galleries, whenever design conditions permit. Plastic chambers or rock-filled trenches are preferred because of their greater resistance to the corrosive effect of hydrogen sulfide gases released by the spraying of the effluent from the orifices.



CENTRAL MANIFOLD
N.T.S.

SPLIT MANIFOLD
N.T.S.



END MANIFOLD
N.T.S.

Notes:

1. All three types of Pressure Distribution Systems would have the same total length of laterals for the same size of subsurface soil absorption system.

2. All force mains, manifolds, and sub-manifolds installed at least 4 ft. below finished grade, to prevent freezing.

3. All pressure distribution laterals are set at elevations above the elevation of the force mains and manifolds to maintain the mains and manifolds full of water between doses.

FIGURE PDS-1

TYPICAL LAYOUTS OF PRESSURE DISTRIBUTION SYSTEMS

Not To Scale

3) PDLs should be placed along the longitudinal centerline of the precast concrete galleries, plastic chambers or stone filled trenches. PDLs in stone leaching beds should be installed in parallel, at a center-to-center distance not greater than six (6) ft.

4) The design of a pressure distribution network should provide essentially equal distribution, as described below, throughout the SWAS.

5) There should be a maximum of 10% difference in discharge rate between any two orifices in a PDL connected to the same manifold.

6) The maximum length of a PDL, for a given orifice diameter and spacing, should be that at which the difference between the rates of discharge between any two orifices in the same PDL does not exceed 10%. PDLs should not be telescoped in size. [N.B. Telescoping of PDLs would make cleanout and unclogging of orifices difficult to accomplish.]

7) The maximum length of a pressure distribution manifold for a given total discharge rate (sum of the discharges from all orifices in all PDLs) should be that length at which the variation in flow between any two PDLs in a PDS does not exceed 10%. To minimize friction losses and assure even flow distribution to the distribution laterals, manifolds should be as short as possible. This will also enable optimization of the manifold diameter. Manifolds may be telescoped in size.

8) The force main from pump station to the pressure distribution manifold, and the manifold(s), should be sized to provide a minimum velocity of two feet (0.6 m) per second.

9) The PDS should be designed to maintain a minimum pressure of at least 3 ft of head (0.9 m) at the distal end of each PDL.

10) The minimum dose should be at least five times the volume of liquid contained in the PDS under pipe-full conditions, plus the quantity of wastewater that drains back to the dosing tank between doses (the drain-back volume, as discussed elsewhere in this document).

    Note: For medium and coarse-grained soils, there should be at least 4 dosing cycles per day based on the design flow for the system. This requirement may, in some cases, and particularly during the first design iteration, appear to conflict with the minimum dose requirement in 10.above. Optimizing the design of the PDS, by adjusting the diameter of the laterals and the diameter and spacing of the orifices, can often eliminate such apparent conflicts, and such optimization should be the goal for every PDS design.

11) For naturally existing fine-grained soils (fine sand, very fine sand, sandy loams, loamy sands), there should be one dose per day based on the design flow for the system and the minimum dose volume requirement indicated in 10) above.

12) Pressure distribution piping, including force main, manifold, and laterals, should be smooth, rigid, plastic pipe. Acceptable pipe and fittings are as listed below, or equal products as may be approved by the Department:

A. Schedule 40 PVC conforming to ASTM D-1785, and plastic ball valves and fittings conforming to ASTM D-2466 with solvent-welded joints. Threaded joints are not acceptable except where otherwise recommended herein.

B. AWWA C900 PVC Pressure Pipe and AWWA C907 PVC Pressure Fittings or cement lined Cast Iron mechanical joint fittings, and resilient seated gate valves (AWWA C509); with thrust restraints as required to prevent separation of the pipe and fitting joints.

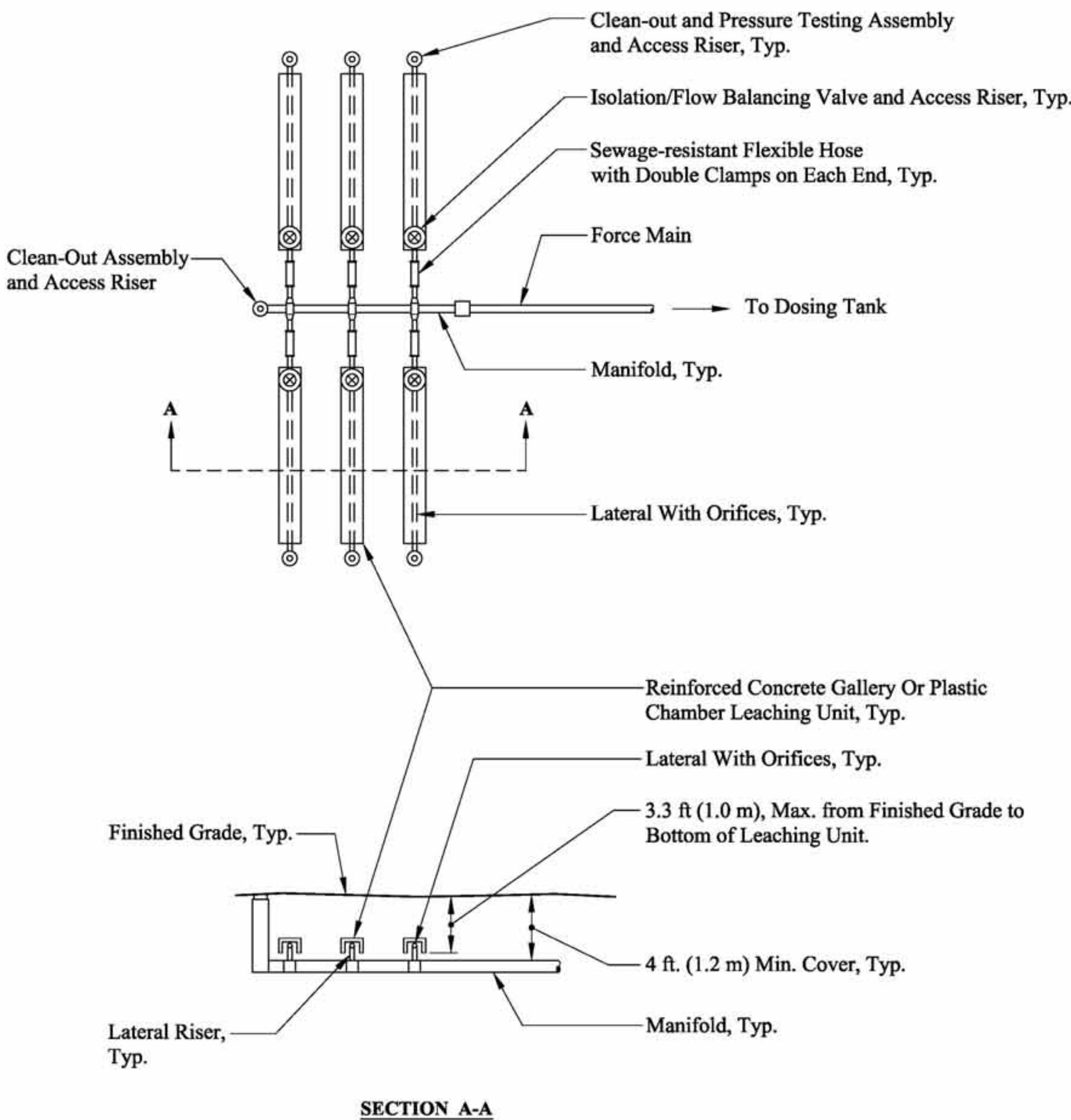
C. Manually operated Plug Valves, and Pinch Valves with elastomer inner bodies or linings, may be used in lieu of plastic ball valves for isolation and control purposes. Pinch valves should be provided with a means to positively open the inner body when the valve is opened by the valve handle.

D. To allow for lateral deflection and/or angular movement of piping components due to earth settling or superimposed loads, flexible connections should be provided wherever such movement can be anticipated. Such connections should consist of flexible hose, or flexible polyethylene pipe, with two stainless steel worm-gear type hose clamps at each end of hose, or approved equal, with thrust restraints to prevent separation of the hose-to-pipe connections.

E. All components of valves and fittings, including flexible connectors, should be suitable for a long life (≥ 30 years) under the expected service conditions (e.g. pressure, temperature, corrosiveness of liquid, frequency of use, etc.).

13) On level terrain, all pressure distribution manifolds and laterals should be laid level. Distribution piping serving separate sections of a large SWAS may be installed at different elevations provided the overall design ensures even distribution. Differences in top of pipe elevation anywhere along the length of any one lateral should not exceed 2 in. (5 cm) from a true level. A typical layout of a PDS on level ground is shown in Figure PDS-2.

14) Alternate layouts of a PDS on sloping ground are shown in Figures PDS-3A and PDS-3B. Other layouts are also possible. However, on sloping ground, the manifold should be located at an elevation below that of the lowest PDL, to avoid siphoning of liquid from the manifold into the PDL. In addition, check valves should be provided on each PDL feeder pipe to prevent backflow from a higher PDL to lower PDLs via the manifold. [N.B. Siphoning could result in overloading the lower PDLs and result in trickling flow to the laterals that can cause clogging of the orifices due to the low orifice velocity that would result.] It is recommended that the “true-union” type of PVC ball check valve be used, since this configuration allows for the working part of the check valve to be removed from the system and repaired or replaced without having to disturb the check valve-to-piping connections.

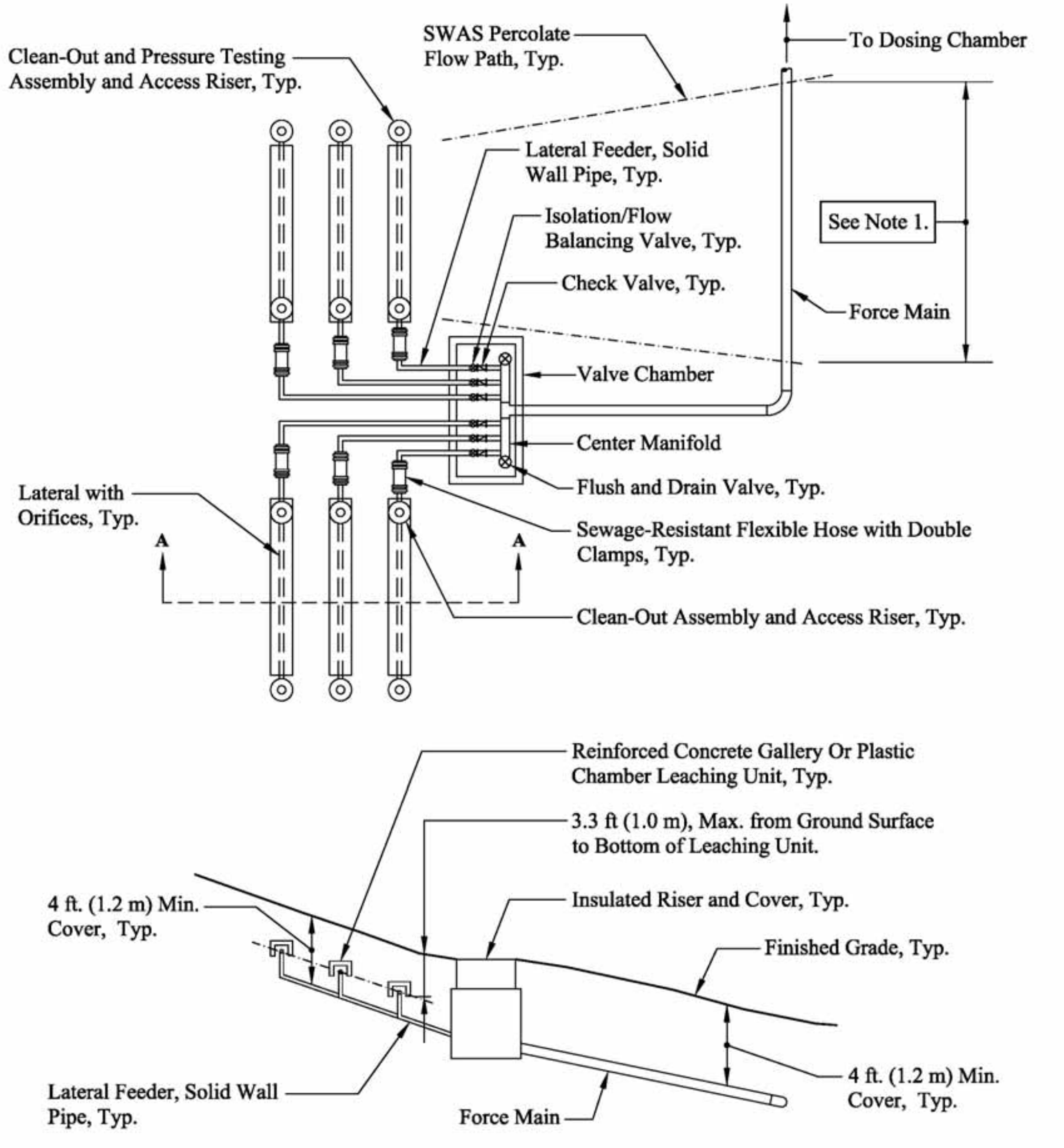


Note: For Additional Details, See Figures PDS-4 and PDS-5

FIGURE PDS-2

PRESSURE DISTRIBUTION SYSTEM LAYOUT ON LEVEL GROUND

Not To Scale



SECTION A-A

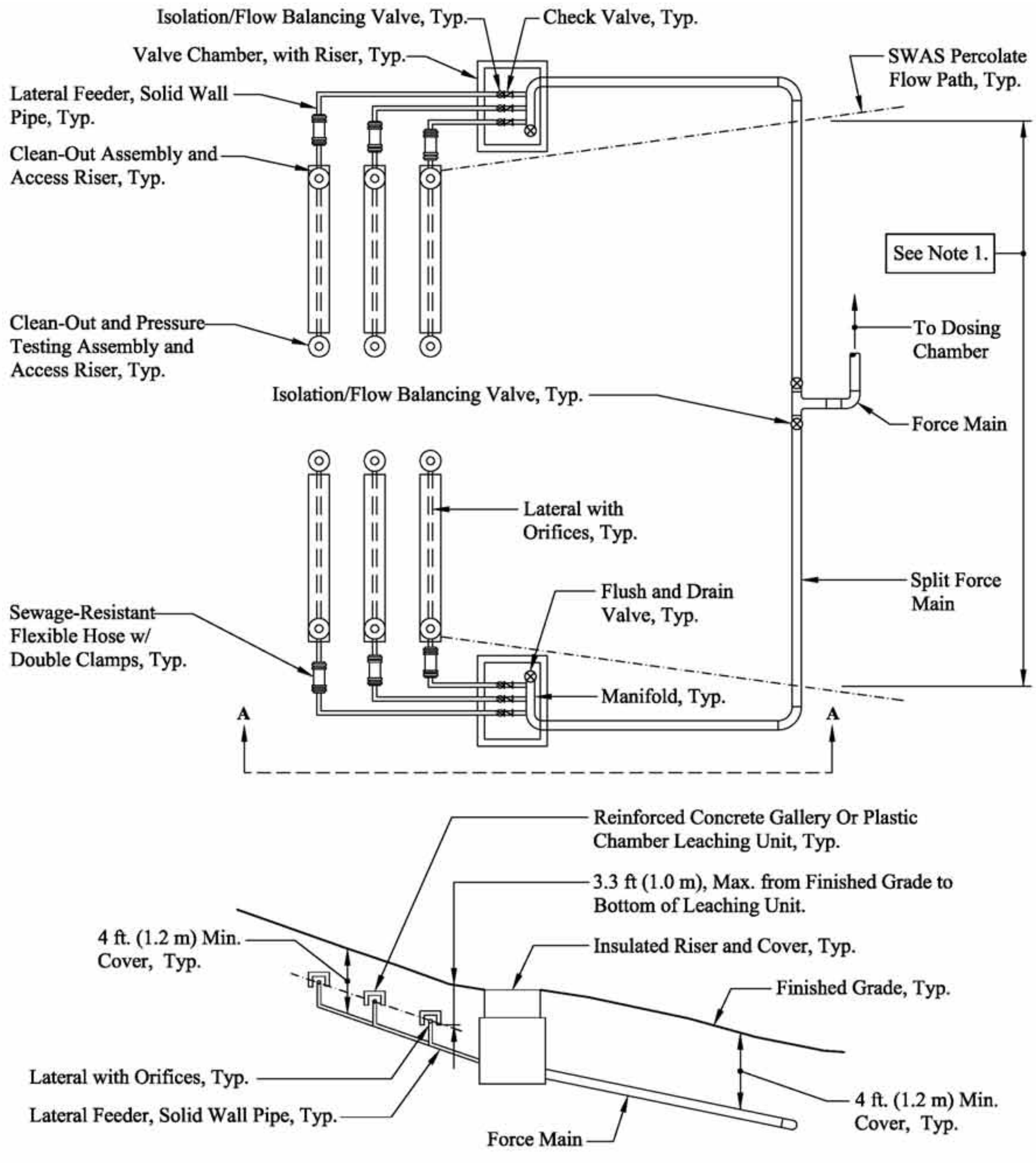
Notes:

1. Trenches for piping installed down-gradient of the SWAS and within the flow path of the SWAS percolate to be backfilled to the elevation of the anticipated ground water table elevation with pervious material having hydraulic conductivity, after compaction, equal to or greater than that of the native soil.
2. For additional details, see Figures PDS-4 and PDS-5.

FIGURE PDS-3A

PRESSURE DISTRIBUTION SYSTEM LAYOUT ON SLOPING GROUND

Not To Scale



SECTION A-A

Notes:

1. Trenches for piping installed down-gradient of the SWAS and within the flow path of the SWAS percolate to be backfilled to the elevation of the anticipated ground water table elevation with pervious material having hydraulic conductivity, after compaction, equal to or greater than that of the native soil.

2. For additional details, see Figures PDS-4 and PDS-5.

FIGURE PDS-3B

ALTERNATE
PRESSURE DISTRIBUTION SYSTEM LAYOUT
ON
SLOPING GROUND

Not To Scale

15) The spacing between PDLs should be as set forth in 3) above.

16) The orifice diameter in a PDL should be not less than 3/16" and not more than 1/2"

17) The PDS should provide as many orifices in the PDLs as is reasonably possible consistent with other design requirements. Orifices should be spaced evenly and in a straight line along the PDL. The spacing between orifices should be not less than 2 ft and not more than 5 ft.

18) Where septic tank effluent is being distributed, the orifices should be located on the bottom of each PDL, and the PDL should be located at the top and centerlines of the leaching system units, as shown on Figure PDS-4. In this case, a hole should be drilled into the top of the distal end of the PDL to permit air to escape when the lateral is being filled during dosing. A small concrete splash plate should be centered below each orifice to prevent erosion of the soil beneath the orifice. [N.B. When calculating the total infiltrative surface area required in the SWAS, the total area covered by the splash plates should be taken into account to determine the gross infiltrative surface area required.] Where enhanced pretreatment of the wastewater is provided, orifices may be placed along the top (crown) of the PDL and the PDL may be located near the bottom of leaching galleries or leaching chambers, as shown in Figure PDS-5.

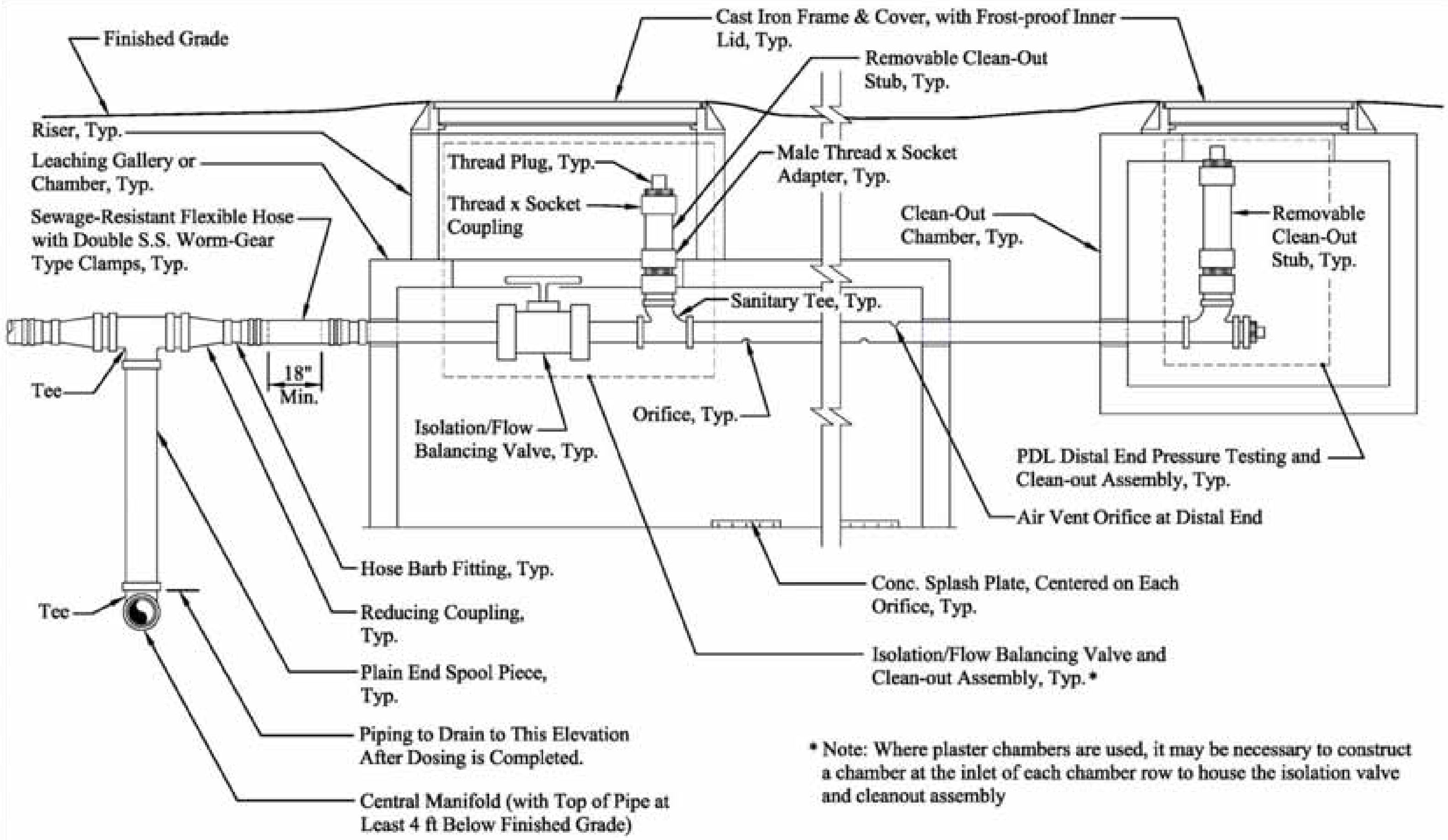
19) All PDLs should be securely supported in place to prevent movement during dosing (filling) of the PDL. Only corrosion resistant supports and hardware suitable for the environment that will exist within the leaching system should be used for securing the laterals in place. [N.B. Some types of plastic electrical cable ties have been found inadequate for this purpose and should not be used. Stainless steel cable ties are available and are recommended.]

20) Where PDLs cannot be secured to the gallery or chamber units, a supporting scheme such as shown in Figure PDS-6 can be used. Supports should be provided in conformance with the PVC pipe manufacturer's recommendations with respect to width of each support and spacing between supports for the maximum expected temperature of the pretreated wastewater.

21) Where pressure distribution laterals are installed in broken stone filled trenches or stone leaching beds, the perforations should face in the upward direction and should be covered with orifice shields to prevent the perforations from being blocked by the surrounding broken stone. The perforated laterals should be covered with at least 2 inches (5 cm) of broken stone to secure them in place and to provide a base for the geotextile fabric covering over the stone.

22) The orifice discharge coefficient used for hydraulic design of the PDLs should be 0.62 unless otherwise approved by the Department.

23) Orifices should be drilled using a drill press with a jig used to assure that all holes are drilled on the same vertical diameter of the lateral piping. A drill bit that does not leave burrs on the inside of the pipe is preferable. (Brad point drill bits are suggested.)



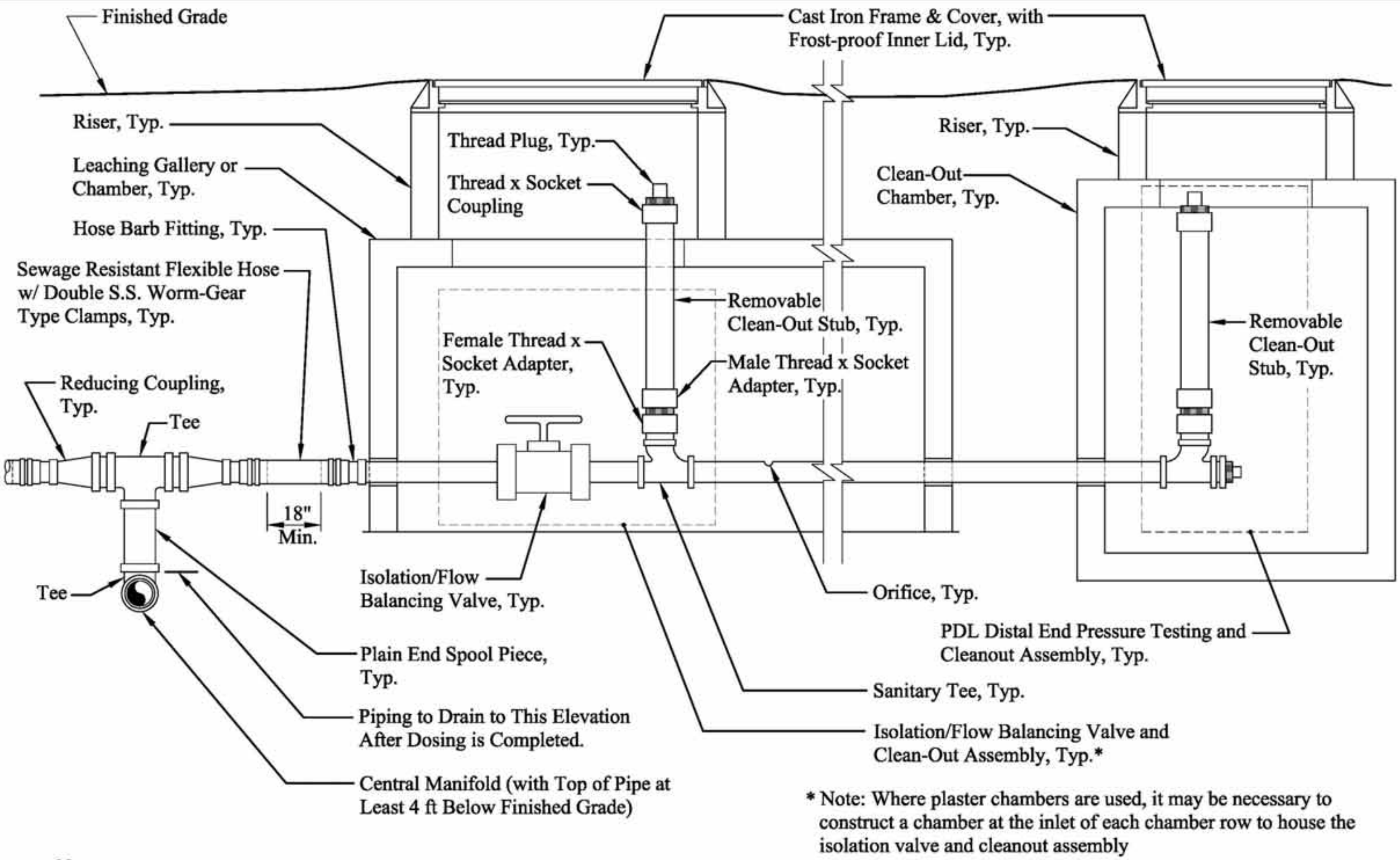
Notes:

1. Details shown above are for installations with orifices facing downward.
2. PDLs can be on located on bottom w/ orifices facing up when enhanced wastewater treatment is provided.
3. All details symmetrical about central manifold.

FIGURE PDS-4

TYPICAL DETAILS OF CENTER MANIFOLD PDS
AND
PDL ISOLATION/BALANCING VALVE AND CLEAN-OUT ASSEMBLIES

Not To Scale



* Note: Where plaster chambers are used, it may be necessary to construct a chamber at the inlet of each chamber row to house the isolation valve and cleanout assembly

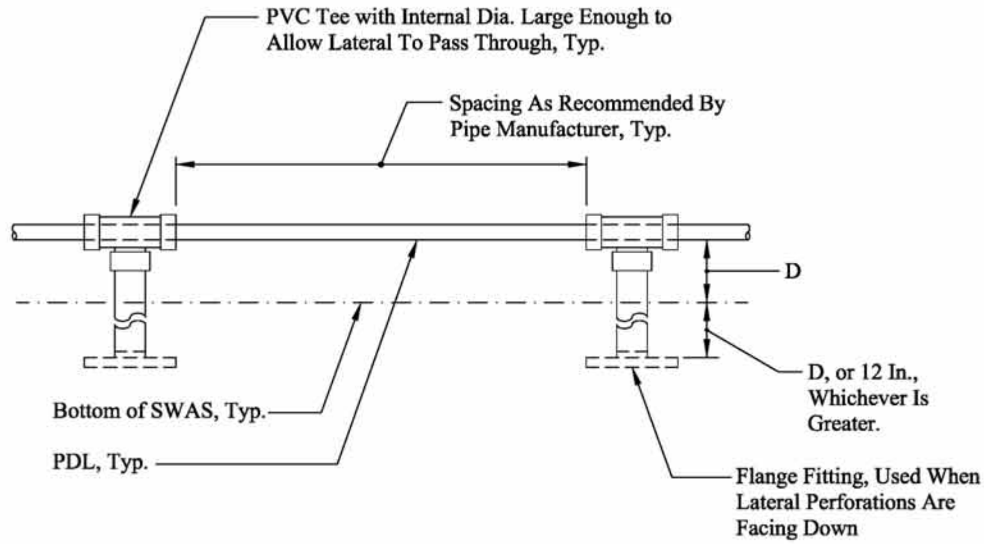
Notes:

1. Details shown above are for installations with orifices facing up.
2. PDLs can be on located on bottom w/ orifices facing up when enhanced wastewater treatment is provided.
3. All details symmetrical about central manifold.

FIGURE PDS-5

TYPICAL DETAILS OF CENTER MANIFOLD PDS
AND
PDL ISOLATION/BALANCING VALVE AND CLEAN-OUT ASSEMBLIES

Not To Scale



Notes:

1. Locate lateral supports so as not to block orifices.

FIGURE PDS-6

PDL SUPPORT DETAIL

Not To Scale

24) Irrespective of what type of drill bit is used, any burrs on the inside and outside of the PDL piping, and any cuttings resulting from drilling of the orifices should be carefully removed without disturbing the drill holes (orifices). Removal of burrs on the inside of the PDL can be accomplished by pushing a pipe of a smaller diameter than the lateral through the pipe with the orifices oriented in the 6 O'clock (downward) position.

25) The PDLs should drain between doses, to prevent freezing. However, in order to minimize the drain-back volume, the force main and pressure distribution manifolds should be installed at a depth below the laterals, where freezing will not occur, and should not be designed to drain between doses. The minimum depth from finished ground surface to the top of the force main and pressure distribution manifolds should not be less than 4 feet (1.2 m). Refer to Figures PDS-4 and PDS-5.

26) Valves should be installed at the proximal (inlet) end of each PDL to permit balancing the flows to all the PDLs. These valves should be PVC True-Union type ball valves, plug valves, pinch valves, or other types of valves suitable for the service conditions to be encountered. Access risers, with frost proof frames and covers, or other means acceptable to the Department, should be installed to provide free access to these valves. Typical details for balancing valve installations are shown in Figures PDS-4 and PDS-5.

27) Provisions should be made for cleaning PDLs (flushing of sediment and unblocking clogged orifices), and to enable checking of operating pressures in the PDLs. These should consist of PVC sanitary tees installed at the proximal (near) end and distal (far) end of each PDL to facilitate cleaning the PDLs and also to provide for visual evidence of balanced flow, as described hereinafter. The branch of the tee should face vertically upwards and should be provided with a removable clean-out stub, as shown in Figures PDS-4 and PDS-5. The clean-out stub should be removable to insure that cleaning tools can enter directly into the sanitary tee if the stub interferes with access.

28) The sanitary tees, clean-out stub, and associated fittings should be of the same nominal diameter as the PDL. Access risers, with frost proof frames and covers or other freeze protection means should be installed to provide free access to the sanitary tee risers.

29) Provisions should be made for cleaning of manifolds. These should be similar to the provisions made for cleaning of PDLs as described above and as shown in Figures PDS-4 and PDS-5.

30) During construction, the orifices, and ends of all PDLs, manifolds, and force mains that have not been sealed with clean-out plugs, should be protected to keep rodents, insects, dirt and other debris out of the piping.

31) After construction has been completed, and before flow balancing is undertaken, the flow balancing valves on each PDL should be closed and the plugs removed from the clean-out stubs at the ends of the pressure distribution manifold(s). The force main and pressure distribution manifold(s) should then be thoroughly flushed with clean water until all debris that may have entered the piping has been removed.

32) The flushing plugs should then be reinstalled at the ends of the pressure distribution manifold(s)to affect a watertight closure.

33) After the force main and pressure distribution manifold have been thoroughly flushed, the balancing valves at the proximal end of each PDL should be opened, beginning with the laterals at the proximal end of the distribution manifold. The threaded plug should then be removed from the top of the clean-out stub at the distal end of each PDL. The PDL should then be thoroughly flushed with clean water, after which the threaded plug should be replaced. This procedure should then be repeated for each PDL, proceeding in the direction toward the distal end of the manifold.

34) After the PDS has been flushed clean, it should be flow-balanced (calibrated) in the following manner:

   a. Remove the threaded plugs at the top of the clean-out stubs at the distal ends of PDLs connected at the same location along the manifold.

   b. Connect clear PVC test standpipes, of the same diameter as the PDLs, with a thread x socket adapter cemented thereto, to the clean-out stubs. The height of each test standpipe should be such that the liquid in the standpipe should remain at least a few feet below the open top of the standpipe when the PDS is being dosed at the normal dose rate. Clean water may be used for calibration purposes.

   c. During dosing, the observed static elevation of the liquid level in the clear standpipes at the ends of the laterals being tested should be marked on the standpipes and recorded. A surveyor's level should be used to determine the static elevations based on the project bench mark elevation.

      The difference in liquid level elevations in the standpipes at the distal end of each pair of PDLs installed at the same elevation and being fed from the same location on the manifold should not exceed 4 in. Also, the average pressure head in any pair of PDLs should not differ by more than 6 in. from the average pressure head in any other pair of PDLs. If either of these tolerances is exceeded, the flow balancing valves should be adjusted until the liquid levels and pressure heads are within the stated tolerances. The pressure heads can be determined by measuring the distance from the liquid level in the standpipes to the top of the laterals.

      [N.B. When the PDS has been suitably flow balanced, the position of the valve handle should be marked on the top of the valve body for future reference in resetting the valve position should the valve setting be changed during cleaning or isolation of the PDLs.]

   d. If the PDS cannot be adjusted to meet the liquid level elevation and pressure tolerances given above, the PDS should be investigated by the Project Engineer to determine the reason for this discrepancy. The Project Engineer should then advise the Department of his findings, conclusions, and recommendations for any corrective actions that may need to be taken.

The design of a PDS should be accomplished using detailed hydraulic calculations rather than using nomographs, curves or tables. These calculations can be performed manually, or by use of a computer spreadsheet program developed for this purpose, or by use of special computer programs developed for this purpose or that can be utilized (adapted) for this purpose.

Care should be taken in using computer programs to insure that the correct orifice coefficient (0.62) is used, as some computer programs may be based on discharge coefficients for flow control devices other than orifices.

The design calculations for a PDS should be submitted to the Department for review along with documentation for any spreadsheet program or other computer program used in designing the system. The documentation should show the methodology used by the program [flow charts, algorithms, hydraulic formulas, spreadsheet formulas, etc.].

In the case of special computer programs, supporting information should also be submitted demonstrating that the program has been checked for various cases by manual calculations or by actual experiments conducted in the laboratory or in the field to substantiate the program results.

5. Design Methodology for Low Pressure Distribution Systems

A methodology commonly used for design of a PDS is given below. Publications containing detailed information and procedures that may be found useful for design of a pressure distribution system are listed in the bibliography at the end of this section.

- Select SWAS configuration. Determine the total length of leaching system galleries or chambers required, based on the total infiltrative surface area required and the infiltrative surface area allowance per linear foot of leaching system used.

- The infiltrative surface area required will be based on the adjusted LTAR, which in turn is based on the saturated hydraulic capacity of the soil and the wastewater strength, as discussed elsewhere in this document. [N.B. When calculating the total infiltrative surface area required in the SWAS, the total area covered by splash plates beneath downward facing orifices should be taken into account to determine the gross infiltrative surface area required.]

- Divide the total length of galleries or chambers required into equal length rows. The length of a row, and the number of such rows, will depend upon the site-specific linear hydraulic loading rate that the soil will accept and convey away from the SWAS while still providing the required depth of unsaturated soil between the bottom of the SWAS and the mounded seasonal high ground water table.

- Determine the configuration of the PDS, including type of manifold. Chose the orifice size and spacing. The orifice size and spacing should conform to the limits established herein and should not be any greater than necessary to meet the dosing requirements while insuring that essentially equal distribution will occur along the length of each lateral and between laterals. Shorter spacing between orifices will provide better utilization of the renovative capacity of the soil.

- <u>Chose the lateral diameter</u>. The lateral diameter should be such that the difference in pressures between the proximal orifice and the distal orifice should not exceed 10% of the operating pressure specified for the orifices.

- <u>Select manifold size.</u> The manifold diameter should be such that the difference in pressure (the total head) between the proximal lateral-to-manifold connection and the distal lateral-to-manifold connection should not exceed 10% of the operating head required at the proximal orifice for each lateral.

- <u>Optimize the PDS design.</u> Repeat steps 1-5 until the PDS design is optimized with respect to flow balancing, minimum dose volume, number of doses per day, and dosing pump capacity.

Pressure distribution systems installed on sloping terrain require special attention to insure that the PDS meets the requirements set forth above. It is left to the ingenuity of the designer to determine the best type of PDS to be used for a particular sloping site. In such cases, it may be necessary to use more than one configuration of a PDS, or to vary the pipe diameters, orifice diameters and/or orifice spacing for individual PDLs, in order to obtain the equal distribution desired. [N.B. Care should be taken if orifice diameters and/or spacing change within the same PDS, as this can lead to errors in construction of the PDS, due to installing a PDL in the wrong location.] Consideration should also be given to using stainless steel orifice plates inserted in unions to provide equal distribution of flow into the PDLs at their various elevations in the PDS. It may also be necessary to utilize more pumps than are required for a PDS situated on level ground, with different pumps being dedicated to each part of the SWAS that lies at an elevation different from that of the other portions.

<u>6. Maintenance of Pressure Distribution Systems</u>

PDLs should be inspected and flushed periodically to ensure proper distribution. This should be done at least once per year. Inspection should include checking the residual pressure at the distal ends of the laterals and comparing the results with the results initially obtained when the PDS was flow balanced. When the results indicate the residual pressure exceeds 130% of the initial value, the PDLs should be cleaned to unclog the orifices. An initial indication of clogged orifices may be a significant increase in the running times of the dosing pumps, but experience has indicated that increases in residual pressure are a more sensitive indication of clogging as compared to pump running time.

Flushing of the PDLs can be accomplished by using the cleanouts provided at the distal end of each lateral. Flushing may be accomplished by use of the pretreated effluent and the pumps in the dosing tank. Provisions should be made for capturing the flushing water and discharging it to a septic tank pumper truck. Periodically, the manifold should be flushed to remove any sediment accumulations, using the cleanout provided at the distal end of the manifold. Flushing may be accomplished by use of the pretreated effluent and the pumps in the dosing tank. Provisions should be made for capturing the flushing water and discharging it to a septic tank pumper truck, or flow equalization tank, if provided, or to the second compartment of the septic tank. In the latter case, the discharge rate should be such as will not unduly disturb the operation of the septic tank.

Unclogging of the orifices can be accomplished by using a plumber's snake equipped with a bristle brush head, or by a water jet device used for such purposes, or by other suitable methods, using the cleanouts provided at each end of the lateral. The residual pressure at the distal end should then be rechecked to ensure it has dropped to its initial value.

J. Recommended Types of Subsurface Wastewater Absorption Systems

1. Rows of Stone filled Trenches
2. Rows of Precast Reinforced Concrete Shallow Galleries.
3. Rows of Factory Manufactured Plastic Chambers.
4. Beds composed of stone filled beds, rows of Shallow Concrete Galleries or Plastic Chambers, but only when enhanced pretreatment of the wastewater is provided.
5. Systems using other types of leaching units that have been approved by the Department.

Note: Enhanced pretreatment is defined as that which will provide an effluent having mean BOD_5 and TSS concentrations $\leq$ 30 mg/L respectively. Where enhanced pretreatment for nitrogen removal is required, the effluent should have a mean BOD_5 concentration $\leq$ 15 mg/L.

Concrete galleries and plastic chambers should be designed and constructed to support the load of the overburden soil and the vehicular wheel loads that can be expected to be imposed upon these units.

For long-term durability, concrete galleries should be constructed using a concrete mixture that will have a 28-day compressive strength of not less than 4,000 pounds per sq. in.

K. Maximum Distance from Ground Surface to Bottom of SWAS

1. It is desirable to minimize the distance from the finished ground surface to the bottom of the SWAS in order to provide the shortest path for diffusion of air into the unsaturated zone beneath the SWAS. Many studies have shown that aerobic conditions in this zone are required in order to provide adequate renovation of the wastewater. Therefore, it is preferable to use shallower-depth trenches, galleries and chambers.

2. A reasonable goal is to keep the vertical distance from finished grade (the ground surface that will exist after construction of the SWAS is completed) to the bottom of the SWAS at 3.3 ft or less. This distance should not be exceeded unless satisfactory provisions are made for introduction of oxygen to the unsaturated zone beneath the SWAS.

L. Surfaces over SWAS

1. No roadway, driveway, parking area, turning area or other area surfaced so as to be considered impervious should be located above a SWAS unless otherwise approved by the Department. Where a SWAS is permitted to be located beneath an impervious surface, it should be provided with an air delivery system to provide a means of introducing atmospheric oxygen into the unsaturated zone beneath the SWAS.

2. Where ventilation piping is provided, it should terminate in an air intake riser with a U-bend, with terminal end of bend at least 30" above finished grade. The intakes should be covered with corrosion resistant screening to prevent entry of vermin and other small animals. The intake risers should be located where they are not susceptible to damage.

M. Enhanced Pretreatment SWAS

When highly pretreated wastewater (BOD_5 and TSS $\leq$ 30 mg/L respectively) will be discharged to a SWAS, the system may consist of beds, as described below.

1. Precast concrete galleries or plastic leaching chambers, with 24 inches of broken stone placed between adjacent rows of galleries or chambers, 3 inches of broken stone beneath the galleries or chambers, and 12 inches of broken stone placed along the outside sidewall areas of the outer rows of galleries or chambers.

2. Rows of galleries or chambers placed side by side, with no broken stone on the bottom, and 12 inches of stone on the outside sidewall areas of the bed. Where chambers are used, broken stone should be placed in the areas between the adjacent chamber walls, to the full height of the chambers.

3. Beds of broken stone at least 12 inches in depth below the PDLs. The PDLs should be placed in parallel lines not more than 6 feet apart.

4. The maximum LTAR for highly pretreated wastewater should be 1.2 gpd/sf of bottom area or 5% of the vertical saturated hydraulic conductivity, whichever is less.

N. Broken Stone and Screened Gravel Aggregate

1. Aggregate used for construction of an SWAS should consist of washed broken stone or washed screened gravel conforming to the gradation given in Table J-1.

TABLE J-1

Gradation Requirements for Broken Stone and Screened Gravel Aggregate

U.S.A. Standard Series Sieves, ASTM E-11 mm	Sieve No*.	Percent Passing
50	2	100
38.1	$1^1/_2$	90-100
25	1	20-55
19	3/4	0-10
9.5	3/8	0-5
0.425	40	0-2
0.075	200	0-1

2. Aggregate should consist of sound, tough, durable stone or gravel, free from silt, dirt, soft, thin, elongated, friable, laminated, micaceous or disintegrated pieces, meeting the following requirements:

   a. Soundness: When tested with magnesium sulfate solution for soundness using AASHTO Method T 104, the aggregate should not have a loss of more than 10% at the end of five cycles.

   b. Hardness: >3 on Moh's hardness scale. (Note: Aggregate that will not leave residue of aggregate material when used to scratch a copper penny, or the penny will not scratch the aggregate, will meet this requirement.)

3. No aggregate fill should be placed on the bottom area of the SWAS within the limits of the inside bottom dimensions of the galleries or chambers, unless the wastewater has received enhanced pretreatment.

O. Horizontal Layout of Trench, Gallery and Chamber Rows

1. All trench, gallery and chamber rows should generally follow ground contours.
2. Where septic tank effluent is discharged to an SWAS, the minimum center to center distance between individual trench, gallery or chamber rows should not be less than 3 times the outside width of the trench, gallery or chamber row.
3. The maximum length of individual gallery and chamber rows should be based on the length of the pressure distribution system perforated pipe laterals determined as set forth elsewhere in this document.

P. Vertical Alignment of Individual Gallery and Chamber Rows

The bottoms of gallery and chamber rows fed by pressure distribution laterals extending from the essentially the same location on a pressure distribution manifold should be at the same elevation and level throughout their length.

Q. Systems in Flood Plains

On-site wastewater renovation systems (OWRS) installed in designated flood plains, or other areas subject to flooding (herein considered as undesignated flood plains), should conform to the following requirements, unless any local regulatory agency or Federal Emergency Management Agency(FEMA) requirements are more stringent, in which case those local or Federal requirements should govern.

1. No fill should be placed in a flood plain for construction of an OWRS, including access ways to the OWRS, unless specific approval for such fill is obtained from those regulatory agencies having jurisdiction over placement of fill in flood plains.

2. An OWRS located in a flood plain should be located on the highest feasible naturally occurring area of the project site and should have preference in location over all other improvements except the water supply well.

3. The minimum bottom elevation of a subsurface wastewater absorption system (SWAS) should be no lower in elevation than one foot above the elevation of the "10 year" still water level designated by FEMA for the area in which the SWAS will be located.

4. Tops of grease traps and septic tanks installed in flood plains should be no lower in elevation than two feet above the elevation of the "10" year flood still water level, and risers should be provided with watertight covers.

5. Pretreatment facilities (excluding septic tanks), control buildings, pump chambers, and emergency electrical generation equipment and their mechanical and electrical components should be located above the 100 year flood still water level designated by FEMA or protected from such floods by methods approved by the Department.

6. In Coastal areas subject to high-velocity wave action (V zones as defined by FEMA), and in floodway and floodway fringe areas in A zones (as defined by FEMA) subject to soil erosion by flowing water, all above-ground structures should be designed and anchored to prevent floatation, collapse, and lateral movement resulting from hydrodynamic and hydrostatic loads, including the affects of buoyancy. All underground tanks, including but not limited to fuel tanks, grease traps, septic tanks, equalization tanks, other pretreatment process tanks, valve chambers, and pump chambers, should be protected from damage due to erosion by wave action and flood water velocity and anchored to prevent floatation, and should be provided with watertight access covers.

7. In flood plains, the surfaces of walkways and drives that provide access to pretreatment facilities (excluding grease traps and septic tanks), control buildings, pump chambers, and emergency electrical generation equipment and their mechanical and electrical components should be no lower in elevation than one foot above the "10 year" flood still water level.

8. As soon as a flooding event is over, and the water levels have receded to the point where the on-site system can be inspected, any damage to the system and the adjacent ground should be reported to the Department and repaired to the satisfaction of the Department. Particular attention should be paid to inspecting and cleaning any grease trap and septic tank effluent filters.

R. Construction of Subsurface Wastewater Absorption Systems

1. Construction of a SWAS should not be undertaken when the ground is frozen or when the ambient temperature is below freezing, or during and immediately following a precipitation event.

2. The initial and any reserve areas set aside for the SWAS, as well as the area immediately down-gradient of the system, should be protected as much as possible from compaction by the contractor's equipment. The use of rubber-tired equipment such as trucks, compactors, backhoes, bucket loaders, etc. should be restricted in these areas. In sandy soils, where such compaction may only be moderate, significant reductions in the soil hydraulic conductivity may occur. In loamy soils, (particularly in silt and clay loams) such compaction can reduce the hydraulic conductivity by orders of magnitude and result in a system failure. Therefore, use of tracked equipment is preferable to wheeled equipment, as the former will exert much less compactive force on the soil.

3. Care should be taken to avoid, as much as possible, the clogging of infiltrative surfaces during construction by smearing with the excavation equipment. If such surfaces are smeared, they should be properly scarified to remove the smeared soils. Loose materials caused by scarification should be removed from the area of the SWAS.

4. Proper materials should be used for construction of the system and careful attention should be paid to the various elevations and grades required for the infiltration surfaces and installation of the various pipes and leaching units.

5. Excavated material should be placed sufficiently distant from the area in which the SWAS is to be constructed so that it cannot be washed into the excavated area during precipitation events. The excavated material should be placed up-gradient of the SWAS and not in the down-gradient area between the SWAS and the nearest point of concern, to avoid changing the hydraulic carrying capacity of the soil by compaction. In some cases, it may be necessary to install a silt fence or hay bales between the piles of excavated material and the area(s) of the SWAS to avoid siltation of such area(s).

6. Where stone-filled trenches, galleries or chambers are to be installed in fill, the fill should be placed to the full design height before excavating for installation of the trenches, galleries or chambers. Fill material should be placed by dumping around the edge of the SWAS area, keeping rubber tired vehicles and equipment off the area. Track-mounted equipment should be used to move the fill material into place.

7. The bottom of the areas excavated for installation of leaching units should be checked to insure that it conforms to the lines and grades shown on the approved construction drawings, and any deviations found should be corrected.

8. Where additional fill is required to bring the bottom of the excavation to grade, it should conform to and be placed in accordance with the construction specifications approved by the Department and as specified elsewhere in this document. When the bottom elevation of the excavation is satisfactory and adequately compacted, it should be raked with a garden or landscape rake to a depth of at least one inch before placement of stone, galleries, or chambers, or pressure distribution piping.

9. After the leaching facilities and associated effluent distribution systems are installed, the top of aggregate fill should be protected by a durable geotextile fabric.

10. The fabric should be listed on the Connecticut Department of Transportation's Approved Products List for geotextile - Subsurface Drainage, Class A, and should be of the non-woven type.

11. Where an SWAS is installed in a fill system, the fabric should extend horizontally to at least 1 ft beyond the top edge of the aggregate fill in each lateral direction. Where an SWAS is installed in original ground, the fabric should extend vertically upwards along the sides of the trench at least 6 inches.

12. Geotextile fabric should also be placed over each joint of leaching units, with at least 6 inches of overlap on each side of the joint, and should extend horizontally or vertically as indicated above for fabric over the top of aggregate fill.

13. Backfill should be carefully placed and over-compaction of this material should be avoided. The top six (6) inches (15 cm) of earth cover material placed over the SWAS should be suitable for establishing a healthy turf.

14. The ground surface over the entire SWAS should be graded and maintained to divert surface water away from the top of the system. The SWAS should be protected from siltation or erosion during and after construction.

15. All of the areas disturbed by construction, not otherwise scheduled to be surfaced, should be limed, fertilized, seeded and mulched so as to establish a healthy turf.

16. Grassed areas in and around SWAS should be moved at least three times during the growing season.

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SECTION XI ENHANCED PRETREATMENT

TABLE OF CONTENTS

SECTION XI ENHANCED PRETREATMENT

A. Introduction

This section provides a discussion on enhanced pretreatment of wastewaters following the typical septic tank (and grease trap if required) pretreatment that is predominately used in on-site wastewater reclamation systems (OWRS) before the wastewater is discharged to the subsurface. An evaluation of the various treatment operations and processes available should be made during the initial phase of designing an enhanced pretreatment facility and prior to selecting the pretreatment operations, processes, and equipment. The purpose of this section is to 1.) provide an overview of various unit operations and unit processes that can be employed to provide enhanced pretreatment, and 2.) call attention to some of the important parameters that affect the design, operation and maintenance of these operations and processes.

It is not intended for this section to provide detailed design criteria for the numerous types of pretreatment facilities usually available as pre-manufactured (packaged) units, although some design criteria are provided for the various processes employed in such units. Detailed design criteria are given in the case of a recirculating granular media filter, a generic facility that is usually designed in-house and constructed on site. There are many references available that will provide detailed design information and procedures and most manufacturers of equipment used for enhanced pretreatment will also provide design assistance. Enhanced pretreatment is normally required when:

1. The wastewater has such a high organic strength (including a high concentration of fats, oils and grease) that it is not feasible to rely upon the usual grease trap-septic tank- SWAS treatment processes for subsurface wastewater renovation.
2. The wastewater has a high concentration of nitrogen that cannot be reduced to the appropriate water quality goal by the usual septic tank-SWAS treatment processes.
3. The soil beneath the SWAS does not have sufficient renovative capacity to remove the phosphorus that is contained in the percolate from the SWAS.
4. The wastewater contains toxic synthetic organic chemicals that must be removed before the wastewater is discharged to a SWAS.

Enhanced pretreatment must be provided where use of reclaimed water is permitted. This is discussed further in Subsection M of this Section.

In selecting pretreatment processes and equipment, the goal should be to utilize such processes and equipment that:

- Yield a consistently high quality effluent that meets water quality requirements as the hydraulic and organic loads vary from low start-up to full design values;
- Are relatively simple to operate;
- Require a minimum of daily maintenance;
- Are not easily upset by unusual variations in such loads;
- If upset, quickly recover; and,
- Are energy efficient.

A guiding principle should be to keep the enhanced pretreatment processes and equipment as simple as possible consistent with effluent requirements.

B. Unit Operations and Unit Processes

1. General

The means used in enhanced pretreatment may consist of physical unit operations, chemical and biological processes or any combination thereof. Those generally used in small-scale pretreatment facilities include:

a. Physical Unit Operations

- Flow Measurement and Sampling
- Flow Equalization
- Pumping
- Mixing
- Gas Transfer
- Flocculation
- Clarification (removal of settleable solids via sedimentation)
- Filtration (may be used for removing residual suspended solids, e.g., post-filtration; as a biological process, e.g. recirculating granular media filters, anoxic reactors.)
- Adsorption

b. Unit Processes

- Chemical (as part of the biological removal of organics and nutrients), including precipitation, adsorption, and pH control.
- Biological (for removal of organics, nutrients)
- Disinfection (may be chemical: e.g., chlorination, or physical; e.g. Ultra Violet Irradiation)

[Note: In larger wastewater treatment plants, unit operations and processes for screening, grit removal, grease removal and primary clarification may be provided. However, in small-scale, on-site treatment facilities, septic tanks and grease traps are normally provided to perform such functions.]

2. Flow Measurement and Sampling

a. Flow Measurement

Continuous flow measurement and recording is a necessary unit operation required for control of enhanced pretreatment facilities. Measurement of the wastewater flow rate should always be provided and in some cases measurement of recycle flow rates will also be required. Such measurements provide the basic intelligence needed to make knowledgeable adjustments to the various physical, chemical and biological processes employed so as to optimize their operation, avoid plant upsets, and meet effluent quality requirements. There are many methods available for flow measurement. They range from simple methods that are relatively easy to maintain and calibrate by plant operating personnel, to complex sophisticated methods that can provide extreme accuracy but require outside expertise for calibration and maintenance. In selecting a flow measurement method, consideration should be given to the level of training and experience that will be required of the plant operators. In most cases, the simpler methods including V-Notch, rectangular and trapezoidal weirs, flow nozzles and flumes will suffice and will provide the level of accuracy normally required for small-scale operations.

The selection of the flow measuring method will depend upon the magnitude and range of the anticipated flows and the location where the flow is to be measured. For relatively small flows, sharp-edged weirs are the usual choice, with the type of weir ranging from a V-Notch at the lower range to rectangular and trapezoidal weirs at the upper range of flows. Where measurement of the plant influent flow is required and for larger flows, consideration should be given to the various flow nozzles and flumes that are available where measurement of the flow depth can be made with devices that are not inserted in the flow path. This is also the case for recycle flows that may contain significant concentrations of suspended solids.

All flow measurement devices should be provided with recording devices that will permit the plant operator to review flow data when analyzing causes of treatment process anomalies. These may be of the chart type, either circular or continuous strip chart, and preferably cover an operational period of a week or more. Electronic data recording devices are also available and may prove suitable in some cases.

b. Flow Sampling

In large plants, flow-sampling equipment often may be dedicated to a particular location in the treatment plant. This is usually not cost effective for the smaller plants under consideration herein. Access for manual flow sampling should be provided at various points in the treatment plant, such as influent, following biological treatment, and effluent.

It is quite helpful to provide a portable automatic sampling device at each plant. Where such devices are provided, a source of electrical power (plug-in electrical receptacles of the ground fault interrupter [GFI] type are ordinarily sufficient) should be provided at each sampling location. Where flow-composited flow sampling is necessary, provisions should be made for conducting the signal from a suitable flow measurement device to the sampling locations. This requires matching the type of flow signal output from the flow measurement device to the type of flow signal that will be recognized by the automatic sampling device.

3. Flow Equalization

The benefit of flow equalization should always be investigated. As discussed in Section X, flow equalization is often cost-effective because it can:

- Dampen the variations in wastewater constituent concentrations.
- Permit downstream processes to operate at more uniform flow rates and contaminant loadings, which is beneficial to the operating stability and efficiency of these processes.
- Result in reducing the size (capacity) of the enhanced pretreatment facilities required.

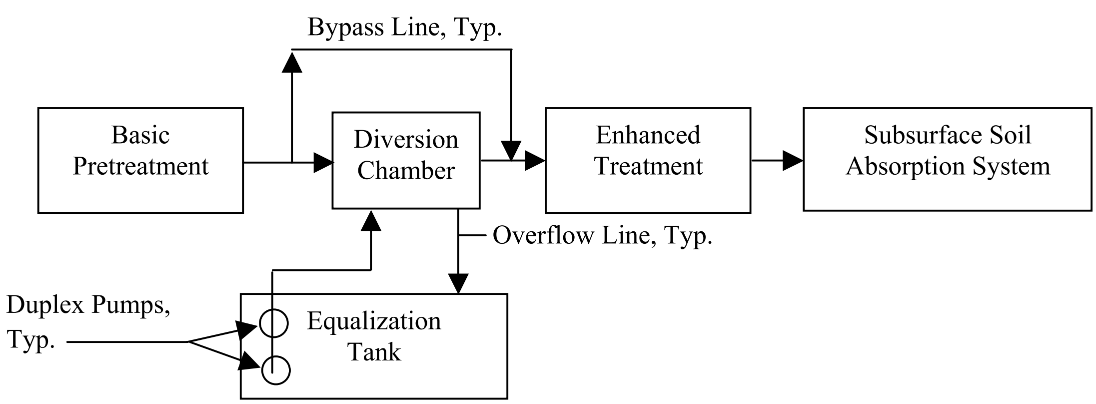
The methodology for determining the volumetric capacity of flow equalization facilities is given in Section X. Factors to be considered in addition to volumetric capacity include:

- Flow control methods for metering flows from the equalization facilities to the downstream treatment facilities.
- Method(s) for removing accumulation of solids from the equalization facilities.
- Control of odors.

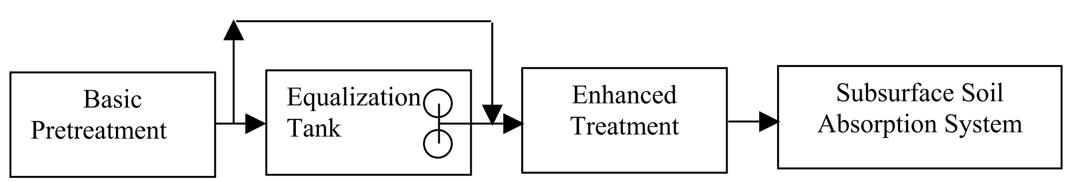
Flow equalization facilities may either be of the sideline or inline type, as shown on the following schematic. The sideline type only receives flows in excess of the average daily flow, while the inline type receives the total flow. The inline type has the greatest effect in dampening wide swings in wastewater characteristics but usually requires pumping of the entire wastewater flow. The sideline type does not have the same effect of dampening the wastewater characteristics as the inline type, but only requires pumping of the flow that exceeds the daily average flow.

Both types of equalization facilities must have provisions for conveying flows in excess of their holding capacity to the downstream treatment units. With careful estimation of flow rates and sizing of the equalization facilities, such overflows should occur rarely, if at all. Facilities to permit intentional bypassing of flows around the equalization basin are necessary in order to permit the periodic removal of accumulated solids and perform other maintenance activities. These facilities can also be used to convey excess flows.

FLOW EQUALIZATION SCHEMATICS



Sideline Flow Equalization



Inline Flow Equalization

a. Sideline Flow Equalization Tanks

Where a sideline equalization tank is utilized, all of the incoming flow is directed to a flow diversion chamber, designed to limit the flow to the downstream facilities to a predetermined average daily flow rate. This can be done by providing the diversion chamber with a weir, orifice or other suitable flow control device for downstream flow control and an overflow device (weir or pipe) for diverting flows in excess of the average daily flow rate to the sideline equalization tank. Flows equal to the average daily flow can be delivered to the downstream treatment facilities via gravity flow provided adequate hydraulic head is made available.

A means for adjusting the rate of flow to the downstream facilities must be provided. This can be accomplished by arranging a manually operated slide gate to operate as a weir, or to vary an orifice opening. A means for measuring the flow rate discharged to downstream facilities should also be provided. Usually, this consists of a simple level-sensing device that delivers a flow depth signal to an indicating and recording device mounted outside of the diversion chamber that is calibrated for the type of flow control device utilized.

Any flow in excess of the desired average daily flow is diverted via the overflow device to the offline equalization tank. When the total flow rate is less than the average daily flow rate, the liquid level in the flow diversion chamber will begin to drop. When this occurs, a liquid level sensor in the chamber will provide a pump start signal to pumps installed in the equalization tank. These pumps will deliver flow from the equalization tank to the flow diversion chamber until the level sensor indicates there is no longer a need for receiving flow from the equalization tank.

For the smaller treatment facilities under consideration herein, the pumps used in the equalization tank are usually of the submersible type, designed for pumping wastewater, and mounted on slide rail systems to permit their removal through an access hatch for ease of maintenance. These pumps operate on the usual automatic alternating cycle method used in most pumping stations; the lead pump operates first, and when it stops, the automatic alternator sets up the lag pump for operation when the next pump start signal is received.

On first glance, selecting the off-line type of equalization would seem to be attractive because all of the inflow would not have to be pumped on a continuous basis. However, there are problems inherent in this method. The flow discharged from the septic tank will be anaerobic and contain dissolved malodorous gases that are easily released to the ambient atmosphere whenever the flow is agitated (such as when the flow passes over or through a flow control device or bypass weir). Therefore, the flow diversion structure should be covered to contain these gases, and the chamber should be vented back to the septic tank, to allow the malodorous gases to escape through the vent stack of the buildings served. It should be noted that these gases are apt to be much greater in volume than those normally released in septic tanks, and thus venting them to the atmosphere via the building vent stack may result in significant odors in the ambient air. In such cases, provisions must be made for odor control.

One of the gases often released is hydrogen sulfide (H_2S), which will tend to condense on the damp, unsubmerged walls and ceiling of the diversion chamber. If any oxygen is present in the airspace of the diversion structure, the H_2S will then be biologically oxidized to sulfuric acid. This will result in the eventual corrosion of the concrete. Therefore, it is necessary to protect these concrete surfaces by lining the walls and ceilings of the diversion structure with a plastic lining that will resist corrosion. The diversion chamber must also be provided with an access opening equipped with a corrosion resistant gasketed cover for periodic removal of solids in a manner similar to that required for septic tanks.

b. Inline Flow Equalization Tanks

In the case of an inline flow equalization tank, pumps are provided to control the rate of flow to the downstream facilities. Pumping the entire flow will permit adjustment of the equalized flow rate by varying the pump delivery rate.

In contrast to the offline equalization method, the inline method provides more flexibility in adjusting the flow to the downstream facilities without experiencing the problem with malodorous and corrosive gases. Careful design of the equalization tank and pumping equipment can avoid agitation of the septic tank effluent and thus avoid release of these gases. Provision of an inflow drop pipe that delivers the inflow to a point below the low liquid level in the equalization tank will prevent agitation at the liquid surface and release of gases into the equalization tank. Since the pump intakes must also be submerged, no agitation of the liquid will occur at this point. If the point of the pumped discharge to the downstream treatment facilities is sufficiently below the liquid level in those facilities, gases should not be released in these facilities. If the downstream facility is a mixed anoxic tank, the mixing device(s) should be such as to not agitate the liquid at the surface of the tank. If the downstream facility is an aerobic tank, the malodorous gases should be oxidized before reaching the atmosphere if sufficient oxygen is available and the discharge into the tank is near the tank bottom.

Since equalization pumps should be able to deliver flow at a relatively constant rate, ideally they should be of the type that have steep head-capacity characteristic curves so that the change in liquid level in the equalization tank will not have a significant effect on pump flow rate. For the smaller treatment facilities under consideration herein, the pumps used for flow equalization are usually of the fractional horsepower, submersible type sewage pumps, mounted on slide rail systems to permit their removal through an access hatch for ease of maintenance. These pumps normally do not have steep head-capacity curves. Further, equalization pumps under consideration herein must be able to deliver at relatively low flow rates. For example, for a design average daily flow rate of 10,000 gpd the equivalent continuous pumping rate would be about 7 gallons per minute, while for a 50,000 gpd flow rate, the equivalent continuous pumping rate would be about 35 gpm. In addition, the total head “seen” by the pump is apt to be fairly low.

Sewage pumps, even the fractional horsepower ones, may have a much greater pumping capacity than required. Throttling a valve on the pump discharge will reduce a pump’s discharge capacity. However, such throttling could force the pump to operate very close to its shut-off head, which is an undesirable condition. Therefore, in most cases a recycling pipe with valve should be installed in the pump discharge piping to allow recirculating some of the flow back to the recirculation tank. In such cases, some throttling of the pump discharge line along with adjusting the valve on the flow recycle pipe will permit adjusting the pumping rate to the desired forward flow rate. The recycle piping should discharge below the low liquid level in the tank to avoid undue agitation of the liquid that could lead to release of entrained gases.

Ideally, variable speed pumps and their associated controls should be used to maintain a constant pumping rate under varying head conditions, but this type of system is usually not warranted for the scale of systems considered herein because of the added cost and complexity.

The range of liquid depths in the equalization tank also has an effect on the ability to maintain an essentially constant flow rate. A smaller range will result in a smaller variation in total head on the pumps and therefore there will be less change in pump delivery rate as the liquid level varies in the tank. The range of liquid depths will be smaller in shallow tanks as compared to relatively deep tanks of the same working volume.

Since in most cases the flow into an equalization tank will come from a septic tank, there should only be a small amount of settleable solids in the flow. Therefore, solids removal will be required rather infrequently. Since the flow discharged from the septic tank may be anaerobic, the equalization tank should be a covered tank similar in construction to a septic tank. Access openings equipped with gasketed covers should be provided for periodic removal of solids in a manner similar to that required for septic tanks. The equalization tank should be vented back to the septic tank, to allow any volatile gases to escape through the vent stack of the building(s) being served. Access openings equipped with gasketed covers will also be required for removal and maintenance of the pumps.

In the case where gravity flow to the downstream facilities via an equalization tank bypass line is not possible, sufficient excess volume should be provided for several hours of peak flow should the pumping system fail. This is necessary in order to provide sufficient response time for repair of the pumping system or to activate standby pumping equipment. It should be noted that the equalization tank also serves as a lift station in this case.

It should be noted that the equalization methods discussed above are not the only ones available, and the designer should review the literature for other methods that may be more suitable for the project at hand.

4. Pumping

Information on pumping systems is provided elsewhere in this section and in Section XII.

5. Mixing

Mixing is used in virtually all enhanced pretreatment facilities. It is used for mixing and maintaining suspensions of non-soluble or partially soluble chemicals in water, to aid in precipitation reactions, and to bring the contaminants in wastewater into intimate contact with chemicals, dissolved oxygen and biomass in various types of reactors. There are many methods and types of equipment used for mixing and their selection will depend to some extent on the processes involved. Mixing facilities can range from high energy mixing of chemicals to much more gentle mixing in bioreactors. Information on mixing in bioreactors is provided in Subsection D and Information on chemical mixing is provided in Subsection E.

6. Flocculation

Flocculation is the gentle stirring of chemicals and biological organisms in contact with wastewater in process reactors. Its purpose is to aid in chemical coagulation processes and for the aggregation of discrete suspended matter into larger clumps of particulate matter (“floc”) to enhance removal of the matter from the liquid in which it is contained by settling in subsequent clarification processes. Flocculation can be accomplished by mechanical and hydraulic means or by aeration.

7. Clarification

Primary clarification (removal of settleable and floatable solids) is usually provided via a septic tank, and grease trap if required, as discussed in Section IX. Clarification is also required to remove suspended solids from the liquids discharged from the aerobic and anoxic processes used for removal of organics and nitrogen, and from any chemical process used for phosphorus removal, except where clarification is carried out in the process reactors (e.g. sequencing batch reactors, membrane bioreactors).

Process design factors for clarifiers following biological treatment processes will depend to some extent upon the type of reactor used for such processes. Where continuous flow dispersed growth reactors are used (e.g. extended aeration mode of the activated sludge process) such factors include:

- Surface loading rate, in gallons per day per square foot of clarifier surface area,
- Solids loading rate, in lb. of suspended solids per square foot of clarifier surface area,
- Weir overflow rate, in gallons per day per linear foot of weir,
- Settled solids removal rate, in gallons per day, and,
- Clarifier depth.

Where fixed film reactors are used (e.g. rotating biological reactors, packed columns) the same factors as listed above apply except perhaps for the solids loading rate. The suspended solids in the liquid discharged from these reactors are a small fraction of those discharged from dispersed growth reactors, thus the solids loading may not be a significant factor if the settleability of the solids is satisfactory. Further discussion of these factors will be given in the discussion of the various suspended and fixed film growth reactors.

8. Effluent Filtration

a. Granular Media Gravity Filters.

A separate filtration step for removal of residual suspended solids in clarifier effluent can be accomplished using either rapid rate granular media filtration or by using the relatively new cloth media disc filters presently available. While membrane filtration can also be used, it is usually not cost-effective except where membrane bioreactors are used for removal of organics and nitrogen. In the latter case, filtration is inherent in the process, as discussed in a subsequent part of this Section.

Granular media filters are often provided to remove most of the suspended solids remaining in the clarifier effluent so as to prevent the possibility of clogging the SWAS and to permit effective disinfection of the treated effluent before it is discharged to the ground. These filters are most often of the down-flow type operated under gravity flow, rather than the pressure flow type of filters sometimes used in potable water treatment. The water to be filtered is discharged to the surface of the filter and travels down through the filter media to a filtered water collection system.

As the solid matter in the wastewater is captured in the filter media, the liquid level above the media rises due to the hydraulic head loss caused by the solids reducing the hydraulic capacity of the media. When it reaches a predetermined level, a level-sensing device actuates a backwash cycle to clean the media of the collected solids. Cleaning these filters involves backflushing the filter with previously filtered water introduced under pressure beneath the filter media. During backwashing, the bed of filter media is expanded and scoured to bring the solids filtered from the liquid to the surface. The surface water above the filter surface that contains the scoured solids is removed by overflow to troughs for further processing to remove the solids from the water. Liquid backwashing is most often preceded with a short period of air scouring of the filter media by compressed air introduced beneath the filter. At least two filter cells should be provided, so that one cell can be backwashed while the remaining cell continues in service.

The controlling parameters for design of a granular media filter include the hydraulic loading and backwash rates, expressed as gallons per minute per square foot of filter surface area (gpm/sq. ft.), the backwash period, and the type, depth(s), effective grain sizes and uniformity coefficients of the filter media.

The hydraulic loading rate recommended by various authorities ranges from 2 to 5 gpm/sq. ft. of filter surface area. A conservative hydraulic loading rate of 2 gpm/sq.ft., with one filter cell out of service for backwashing, is recommended for sizing the surface area of each filter cell. The backwash rate should be ≥ 15 gpm for a period of not less than 10 minutes, and the backwash should be preceded by air scouring of the filter media. The volume of filtered water stored in a clearwell for use in backwashing should be at least equal to the product of the backwash rate x the filter surface area x the backwash period. Equivalent storage should be provided for the spent filter backwash water so that it can be returned at a relatively low flow rate to the head of the plant for solids removal.

The type, effective grain size and uniformity coefficient of the filter media recommended by various authorities varies, but the consensus seems to favor coarse-to-fine filtration using two or more layers of different media, each layer having a different specific gravity, effective grain size and uniformity coefficient.·Dual media filters usually consist of a bottom layer of sand and a top layer of anthracite. The granular material used in multi-media filters are usually specified by the manufacturer, but usually consists of anthracite, sand, and garnet. The recommended effective grain sizes for a dual media filter using sand and anthracite range from 0.5 to 1.0 millimeters (mm.) and 1.0 to 2.0 mm respectively, and it is recommended that the uniformity coefficient of both media not exceed 1.7. The manufacturer usually specifies grain sizes and uniformity coefficients for multi-media filters. The filter media are supported by layers of aggregate, graded coarse to fine, or by other suitable means that will prevent the media from escaping the filter.

b. Cloth media disc filters

Cloth media disc filters have been placed into service over the past 25 years at hundreds of installations in Europe. Within the last decade they have been accepted for use and installed in a number of wastewater treatment plants in the U.S.

The filter cloth media is attached to both sides of vertical discs that have a hollow structure. A number of such discs are mounted on a central hollow shaft in a filtration chamber. The liquid to be filtered enters the tank and flows through the filter media into the inside of each hollow disc under a gravity head. A collection header is connected to the hollow area of each disc via the hollow central shaft and serves to collect the filtrate and convey it out of the filter chamber. The filter unit is controlled by an automatic system that can cycle the filter through cycles that include normal operation, solids wasting, backwash and a pressure spray wash.

During the filtration operation, the discs are submerged in the liquid and there is no movement of any mechanical devices within the filter chamber. Thus relatively quiescent conditions prevail that are conducive to gravity settlement of the heavier solids. As filtration proceeds, solids accumulate on and within the depth of the filter cloth, forming a mat that enhances the filtration process. As the mat is formed, the hydraulic head loss through the cloth increases and this causes the liquid level in the filter chamber to rise. When the liquid reaches a predetermined level, the automatic control unit cycles the filter into a backwash mode.

During backwash, the discs remain submerged and are rotated very slowly (1 rpm) by a drive unit connected to the central hollow shaft on which the discs are mounted. While the discs are rotating, water collected in the filtrate header is drawn back through the filter cloth by suction headers located on either side of each disc, thus backwashing the filter cloth from inside to outside. The reversal of flow removes the majority of the suspended solids that have accumulated on the surface and within the filter cloth. Solids that have accumulated on the surface and within the filter cloth that may not have been removed during the normal backwash cycle are periodically removed by a pressure spray washing system. (Some newer systems with improved filter cloth have eliminated the pressure spray washing system.) Periodically, a sludge pump activated by the control system removes the settled solids through a manifold located in the hopper bottom of the tank.

These filters have been found to be very effective in removing suspended solids. Tests were conducted at two wastewater treatment plants in Florida, which included comparison test runs with both cloth filters and conventional sand filters at various loading rates. The results demonstrated that the cloth media disc filter effluent compares favorably with the effluent of conventional granular media filters, both with respect to efficiency of TSS removal and effluent TSS concentration. As a result, cloth media disc filters have been approved by the Florida DEP for use in lieu of conventional granular filters for producing reclaimed water for reuse in residential and commercial irrigation as well as other purposes. Cloth media disc filters have also been in use for several years in a wastewater treatment plant in Connecticut that produces a very high quality effluent, with turbidities normally below 2 NTU. They produce a filtrate that is equal to or better than that of a rapid rate granular filter while occupying a much smaller floor area.

The design parameters for cloth media disc filters include the hydraulic loading rate (gpm/ft^2 of filter surface area) and solids loading rate (lbs/ft^2 of filter surface area/day). Typical hydraulic loading rates for effluent from extended aeration activated sludge treatment facilities are 4.0 gpm/ft^2 for average daily flows and 6.0 gpm/ft^2 for the peak flows. A typical solids loading rate is 1.8 lbs/sf^2/day on an average sustained flow basis.

Cloth media disc filter units are available for installation in concrete chambers constructed on-site or as packaged units in steel chambers. However, the units may not be cost-effective at the low end of the range of flows encountered in facilities used for onsite wastewater renovation systems. New cloth media drum filter units are being developed that may be more cost-effective than cloth media disc filters for smaller flows.

9. Disinfection

Disinfection is the use of a chemical or physical process to destroy or inactivate pathogenic microorganisms (pathogens). It should be noted, however, that disinfection does not necessarily destroy all microorganisms, such as in sterilization. In the case of wastewater treatment, the goal is to prevent the spread of pathogenic microorganisms, and thus it is their destruction or inactivation that is of concern.

Where only a septic tank and a SWAS are used for onsite wastewater renovation, some of the viable pathogens are eliminated in the septic tank. The greatest amount is eliminated or inactivated in the biomat that forms at the infiltrative surface of a SWAS and in the unsaturated soil zone beneath the SWAS. Some of the remaining pathogens are eliminated or inactivated when the percolate from the SWAS commingles with the ground water and travels in the saturated zone to a point of concern. When enhanced pretreatment of the wastewater is required, often to reduce the concentration of nutrients (nitrogen, phosphorus) in the treated wastewater and sometimes to reduce the organic loading on the SWAS, the formation of a biomat will be severely reduced. However, pretreatment to meet the water quality goals set forth in the Department's Design Standards for enhanced pretreatment will be considered equivalent to the pathogen reduction normally obtained in the biomat.

In certain instances, disinfection of effluent from enhanced pretreatment facilities may be required. Situations where disinfection must be provided include:

- When the Department permits the use of reclaimed water for any beneficial re-use (e.g. irrigation of vegetation, recycling for use in toilets and urinals).

In the case of on-site wastewater treatment, where the renovated wastewater will reach and mix with the ground water, it is important that the disinfection process does not create chemical byproducts that have been found harmful to living organisms. For example, disinfection using chlorine is problematic because there are a number of organic compounds in wastewater that can react with chlorine to form toxic compounds. De-chlorination after chlorine disinfection will not prevent the formation of such byproducts. Thus, where disinfection is required prior to discharging pretreated wastewater to the ground water, it should be determined that no harm will be done to the subsurface environment by the disinfection process. Further information on disinfection is given in Subsection H.

<u>10. Adsorption</u>

Adsorption processes have seldom been used for enhanced pretreatment facilities, the exception being the Zenon Cycle-Let® facilities that include activated carbon adsorption as a polishing step to produce an effluent suitable for re-use in non-potable water facilities.

Adsorption is the attraction and accumulation of one substance on the surface of another. The adsorption process that may be found useful for enhanced pretreatment is adsorption of toxic organic chemicals by granular activated carbon. Activated carbon has a preference for organic compounds and therefore is very effective in removal of toxic organic compounds in wastewater that cannot be easily removed by biological or chemical processes. Since adsorption is a surface phenomenon, granular activated carbon is particularly suitable for this purpose because of its extremely high surface area per unit mass. Granular activated carbons have surface areas ranging from 500 to 1,400 square meters per gram, equivalent to a range of 56 - 157 acres per lb. Activated carbon is made from a variety of carbonaceous materials and thus the quality of the carbon and its ability to adsorb the wide variety of toxic organic chemicals will depend upon its source material as well as the method of production.

Another adsorption process that is emerging for use in enhanced pretreatment facilities is the adsorption of phosphorous on beds of reactive media. Additional discussion of adsorption processes for removal of toxic organic chemicals and P can be found in Subsections G and I.

11. Chemical Processes

Chemical processes used for enhanced pretreatment usually are required for control of biological processes (pH control, external carbon sources to sustain the denitrification process), for disinfection, and for removal of phosphorus where required. Therefore, discussion of such chemical processes is included under the latter headings. Chemical processes may also be used to process commercial laundry wastewater. In such cases, information is usually available from vendors who specialize in such processes and provide a turnkey treatment facility.

a. Storage and Handling of Chemicals

The storage and handling of chemicals must be accomplished in a manner that is consistent with the health and safety of the personnel who operate and maintain wastewater treatment facilities and the general public. Many of the chemicals used in wastewater treatment processes may be highly corrosive, volatile and flammable, or combustible in the presence of organic materials. In addition, some of these constitute a danger to human health due to inhalation of vapors or dusts, ingestion, or skin or eye contact and their unintended release (spills, overfeeding) can be harmful to the environment. The Material Safety Data Sheet (MSDS) obtainable from the manufacturers for each chemical proposed for use at a wastewater treatment facility should be consulted for specific requirements for their storage and handling. In most cases, separate rooms should be provided for storage of chemicals. Sufficient room for storage of chemicals must be provided so that sufficient chemicals will always be available between periods of replenishment. Spill containment facilities, adequate ventilation and provisions for prevention of ignition should be incorporated into the design of such rooms.

- Spill containment facilities should be provided in chemical storage and feed areas and should be capable of safely containing the entire volume of the largest container of each chemical. Spill containment facilities should not be connected to any floor drains, since the spilled chemical from one containment area may not be safely mixed with a possible spill from another containment area, and their mixing in the floor drain waste holding tank may cause a hazardous condition to develop.
- Adequate ventilation should be provided for maintaining a safe environment and to remove explosive vapor and dust concentrations. Where vapors are heavier than air, ventilation intakes should be provided at the floor level. All ventilation facilities should discharge to a safe outdoor location, away from work areas and other nearby inhabited areas and where there is no ignition source nearby.

The requirements of safety codes should be scrupulously followed in areas where flammable or combustible liquids and solids are stored or where explosive vapors or hazardous dust concentrations can possibly be encountered. The safety codes include the National Electric Code (NEC), the Flammable and Combustible Liquids code and recommended practices promulgated by the National Fire Protection Association (NFPA) and all state and local building codes. Only non-sparking tools should be used in such cases, and all containers should be grounded to guard against the possible ignition due to the discharge of static electricity. Containers used for flammable or combustible liquids should be provided with a means to prevent the formation of static electricity in the container when such liquids are discharged into the container. Such provisions usually consist of a tube, grounded to the container, which will convey the liquid being discharged to below the liquid level in the container to avoid splashing.

b. Chemical Feeding Equipment

Chemical feeding for enhanced pretreatment onsite usually involves equipment such as chemical mixing (dilution) and feed tanks, chemical feed pumps (typically of the positive displacement type), and associated piping. Care must be taken to assure that all of the materials used in such equipment and piping, with particular emphasis on all wetted parts, including seals, are compatible with the chemical being used, to avoid problems with corrosion and clogging.

Where highly flammable and potentially explosive chemicals are used (e.g.: methanol), all equipment and piping should be grounded and located in a separate room accessible from outside of the areas in which the enhanced pretreatment facilities are located. All electrical equipment and associated wiring should be explosion proof, UL listed for Class I, Division I, Group D locations. The room should be ventilated to the outside atmosphere and the vent piping should be equipped with an UL-listed flame-arresting device.

Where chemicals composed of a suspension of diluted solids are used that may tend to separate (plate out, or precipitate) from the suspension, (e.g.: magnesium hydroxide, lime) provisions should be made for introducing warm water to the pumping and piping system to remove any clogs that may have developed.

All chemical feed pumps should be provided with pressure relief, anti-siphon, and backpressure valves on the discharge side of the pumps. Where the pump manufacturer will not provide an anti-siphon valve because of the materials being pumped (usually in the case where the pumped liquid is a suspension of solids in water), a separate valve should be provided that will operate under such conditions. A check valve, installed in the reverse position, can provide the anti-siphon function in this case. The discharge from pressure relief valves should be piped back to the chemical feed tank. All chemical feed pumps should be equipped with splashguards to protect the plant operator and adjacent equipment in case of a leak or seal failure. The feed pumps should be provided with a means of varying their pumping rate over at least a tenfold range, using either a stroke or frequency adjustment, or both. The pumps should be securely supported on the chemical feed tank cover or independently supported by a wall or floor mounted stand immediately adjacent to the feed tank.

Tanks containing chemicals should be covered preferably using the same material as the tank. The cover should be easily removed for inspection of the tank interior. Where mixing is required, it is usually accomplished with fractional-horsepower electric motor operated mechanical mixers independently supported by wall or floor mounted stands immediately adjacent to the tanks. The stands should be constructed with sufficient strength to withstand the long-term vibration typical of mixer use and should have a chemical resistant coating compatible with the chemicals being mixed. Operation of the mixers should be initiated using a “manual-off-automatic “selector switch that will permit both manual control and operation by a repeat cycle timer capable of continuous, 24 hour duty in programming the number of pumping cycles required by the process design. In cases where the feed rate will be automatically controlled from flow pacing or other instrumentation, the pump should be equipped to operate based on the signals received from such instrumentation.

Pipe and tubing should be flexible wherever possible and in all cases should be arranged for easy removal and replacement. Pipe and tubing supports should be provided as recommended by the manufacturer of the pipes and tubing. Where chemicals are being conveyed under pressure, the carrier pipe and tubing containing the chemicals should be enclosed in flexible containment piping to avoid accidental release of the chemicals should the carrier pipe develop a leak or burst. The containment piping should also be compatible with the chemical being conveyed in the carrier pipe and should consist of a continuous length of piping wherever possible, to eliminate joints between the chemical feed area and the reactor receiving the chemical. Where joints in piping or tubing are exposed, they should be provided with splashguards. In the case where highly flammable and potentially explosive chemicals are being conveyed under pressure, the carrier pipe should consist of a flexible hose (compatible with the chemical) provided with a stainless steel overbraid, equipped with all necessary couplings and adapters. Suitable warning labels, in a format approved by OSHA, should be mounted on the chemical feed equipment and piping, identifying the chemical being conveyed and the hazards involved.

Provisions should also be made for the safety of plant operating personnel that will be handling chemicals. Such provisions include, but are not necessarily limited to, emergency eyewash stations, emergency showers, emergency medical kits, protective clothing, gloves and safety goggles, and fire extinguishing equipment and should conform to any Federal, State and local safety regulations.

Further information on chemical feed systems is provided in Subsections E and I.

12. Biological Processes

Biological processes are used for removal of organic compounds that exert a carbonaceous biochemical oxygen demand (CBOD) and for removal of nitrogen compounds that exert a nitrogenous oxygen demand (NOD) from domestic wastewater. Aerobic processes have a much higher reaction rate than anaerobic processes, do not produce strong or offensive odors, and are therefore more suitable for removal of organics from domestic wastewater.

Removal of nitrogen involves both aerobic and anoxic processes. While biological processes may also be used to remove phosphorus, the operation is considered too complex for use in the scale of enhanced treatment facilities under discussion herein.

Enhanced pretreatment processes for removal or reduction of CBOD and NOD include suspended growth processes, fixed film processes, and hybrids that combine both fixed film and suspended growth processes. The suspended growth process provides an environment in which the microorganisms that remove the impurities from the wastewater are held in suspension in intimate contact with the wastewater to be treated in a bioreactor.

The fixed film system provides an environment in which the microorganisms are attached to some type of media, with the wastewater either passing through the media or the media passing through the wastewater flowing through the bioreactor.

a. Removal of Organic Materials

Various heterotrophic bacteria and higher life forms (e.g.; protozoa and rotifers, some microscopic and some macroscopic in size), hereinafter for simplicity collectively referred to as "microorganisms" or "microbes", that utilize the organic compounds present in wastewater in their metabolic processes are responsible for the removal of organic materials. Microbes grow by coupling the reactions that produce energy with those reactions involving cell synthesis (U.S. EPA-1993 a). In these processes, the microbes utilize the organic compounds as sources of energy and food for cellular maintenance and synthesis of new cells in a series of oxidation-reduction (redox) reactions.

Chemical energy needed to support the microbial life processes is obtained in the process of oxidizing organic matter from a series of reactions that involve the release of electrons from electron donors (the carbon compounds) to electron acceptors. The electrons are transported via an electron transport chain, utilizing several steps involving various enzymatic reactions, to an ultimate electron acceptor. In aerobic heterotrophic reactions, the ultimate electron acceptor is free (dissolved) oxygen.

The effectiveness of the microbial processes in removal of organic compounds from the wastewater will depend upon the environment in which they exist. The environmental conditions of concern include moisture, dissolved oxygen (D. O.) concentration, pH, temperature, presence of required microbial cell nutrients (nitrogen, phosphorus, various micro-nutrients), and the chemical characteristics of the wastewater.

All of the microbes responsible for removal of organic materials require moisture (water) to remain viable. A sufficient dissolved oxygen (D. O.) concentration in the water is critical to the life processes of the microbes. Generally speaking, a D. O. concentration of 2-3 mg/L is sufficient to maintain a reasonable efficiency of the microbial processes. A higher D. O. concentration may serve to optimize the removal of organics, but may not provide an overall advantage because of the energy (electrical power) costs involved.

While most microbes will function in a pH range 4-9.5, the optimum pH for aerobic microbial processes usually lies between 6.5 and 7.5. This is within the pH range of domestic wastewater and thus pH is not usually a limiting condition for removal of organics. However, as discussed under the following subsection b., pH can be a limiting condition for the nitrification process.

Temperature has a significant effect on the metabolic processes of the microbes. A 10°C rise or fall in temperature from a reference temperature will cause the microbial reaction rates to double or halve, respectively. Optimum temperatures vary with the type of microorganisms, but temperatures below 10° C can significantly reduce the reaction rate. While high temperatures will also have a deleterious effect, they are usually much higher than the ambient temperatures experienced in domestic wastewater treatment.

The major nutrients required to sustain the microbial life processes include nitrogen and phosphorus. Where the carbon content of the organic wastes are represented by the five-day biochemical oxygen demand (BOD_5), for every 100 mg/L of BOD_5 5 mg/L of nitrogen and 1 mg/L of phosphorus are required. Various micro-nutrients (e. g.: sulfur, potassium, calcium, sodium, magnesium and other trace metals) are also required. All of these nutrients are generally found in domestic wastewater.

The chemical characteristics of concern include the presence of slowly biodegradable organic materials and substances that are toxic to the microbes. Slowly biodegradable organic materials may not be removed, or may only be partially removed in the aerobic treatment processes. An example of this is organic nitrogen, as discussed in b. below. Toxic chemicals have been found to adversely affect the operation of some enhanced pretreatment facilities constructed in Connecticut that serve commercial establishments. As indicated in Section IV, the Department is aware of several instances where cleaning chemicals (e.g. quaternary ammonium compounds) discharged to the building sanitary sewer systems inhibited the biological treatment processes, resulting in degradation of the quality of the treated effluent. Domestic wastewater discharged from RV wastewater holding tanks is another case where toxic chemicals may be encountered.

b. Removal of Nitrogen

Both physical/chemical and biological processes can accomplish nitrogen removal. However, physical/chemical processes are not considered to be suitable for OWRS because of the cost of such processes, the operational problems inherent in such processes, and the need for highly skilled operation. In fact, while physical/chemical processes were once considered to be attractive for nitrogen removal at municipal wastewater treatment facilities, they have largely been abandoned in favor of biological processes.

As stated in Section II, to remove nitrogen via biological processes, nitrogen (ammonium-N) in the wastewater must first be oxidized to nitrates in the nitrification process, and then reduced to nitrogen gas in the denitrification process. Most of the nitrogen in wastewater receiving pretreatment in a septic tank is in the form of the ammonium ion (NH_4^+), with some organic nitrogen, and sometimes trace-to-small amounts of nitrite (NO_2^-) and nitrate (NO_3^-) also present. Most of the particulate organic nitrogen is hydrolyzed to soluble organic nitrogen in the septic tank. Some of the soluble organic nitrogen is then mineralized to ammonium-N in the septic tank, and most of the remaining soluble organic nitrogen is mineralized in the nitrification process. A relatively small amount of organic nitrogen in domestic wastewater is refractory (generally 1-2 mg/L) and thus will pass through the nitrification-denitrification process without alteration.

Nitrification is accomplished in a two-step sequential aerobic process.

Ammonium (NH_4^+)------> Nitrite (NO_2) ------> Nitrate (NO_3)

In the first step, ammonium-nitrogen (NH_4^+) plus oxygen is utilized by autotrophic bacteria (e.g. Nitrosomonas) to produce nitrite (NO_2) plus water + hydrogen ions. The autotrophic bacteria obtain their carbon for cell synthesis from inorganic sources such as carbon dioxide (CO_2), present in wastewaters rather than from the organic carbon utilized by the heterotrophs. The autotrophs obtain their energy from oxidation of ammonium (the electron donor), with electrons released in the process, and oxygen is used as the electron receptor.

In the second step, autotrophic bacteria (e. g.: Nitrobacter) oxidize nitrite to nitrate. This process results in the consumption of approximately 4.6 lbs. of free (dissolved) oxygen and 7.1 lbs. of alkalinity (as $CaCO_3$)/lb. N oxidized. (Note that 7.1 lbs. of alkalinity is the calculated theoretical (stoichiometric) amount, and that overall alkalinity consumption is generally somewhat less.) The consumption of alkalinity tends to lower the pH of the water (unless there is sufficient alkalinity available in the wastewater so that at least 50-mg/L alkalinity remains after nitrification is complete). The optimum range of pH for nitrification = 6.5-7.5. Nitrification will be significantly inhibited when the pH drops below 6.0 or increases to above 8.0. When the pH drops below 6.0, nitrification rates decline significantly and, below a pH of about 4.5, nitrification is usually completely inhibited. Thus, if calculations indicate that there is insufficient alkalinity available in the wastewater being nitrified, it will be necessary to add alkalinity (in the form of lime, sodium bicarbonate, sodium hydroxide (caustic soda) or magnesium hydroxide) to sustain the nitrification process.

The nitrification process is also sensitive to the amount of dissolved oxygen present. The maximum growth rate of the nitrifiers is reported to occur at D. O. concentrations $\geq$ 8 mg/L while the minimum concentration required to assure that the nitrification process is not oxygen limited is typically considered to be 2 mg/L. Maximizing the nitrifier growth rate may appear desirable. However, maintaining a high D. O. concentration in a nitrification reactor may not be cost effective because of the additional energy costs involved in providing the higher D. O. concentration and the increased carbon source requirements needed to purge any excess D. O. in the denitrification process.

Denitrification is accomplished in a multi-step process; e.g.:

Nitrate (NO_3^-)-->Nitrite (NO_2^-)-->Nitric Oxide (NO)-->Nitrous Oxide (NO_2)-->Nitrogen gas (N_2).

However, contrary to the need for free (dissolved) oxygen in the nitrification process, the denitrification process must proceed under anoxic conditions; that is, the dissolved oxygen concentration must be $\leq$ 0.3 mg/L and there must be nitrates present. The facultative bacteria that accomplish the denitrification process will use free (dissolved) oxygen in preference to the chemically bound oxygen in nitrites and nitrates for their metabolic processes. Thus, if dissolved oxygen in any significant amount ($\geq$ 0.3 mg/L) is present, the denitrification process may not be completed. (It should be noted that denitrification has been found to occur in aerobic bioreactors. This is thought to occur in anoxic microsites that exist within the aerobic biomass, or to result from a yet to be understood biochemical reaction. However, this occurrence is not usually taken into account in designing the small scale biological treatment processes of interest herein, except in the case of recirculating granular media filters, as will be discussed in a following subsection.)

The energy needed for facultative bacteria cell growth (synthesis) is obtained from conversion (reduction) of nitrate to nitrogen gas. However, in order for the denitrification process to proceed, there must also be a source of readily assimilable (easily biodegradable) carbon, as well as phosphorus and various micro-nutrients available for cell synthesis. Domestic wastewater itself may provide a sufficient carbon source and usually will contain sufficient phosphorus and micro-nutrients to permit a substantial reduction in nitrogen. If sufficient carbon is not available in the wastewater, various types of external carbon can be added to sustain the denitrification process

Where a high degree of nitrogen removal is required, an external readily assimilable carbon source is normally required, since the denitrification rate for such sources is several times greater than the rates ordinarily experienced using the carbon in domestic wastewater. An ideal external carbon source material for the biological denitrification process is one that is readily assimilable by the denitrifying microorganisms, is free of nitrogen and substances toxic to the bacterial process, and is of uniform quality; that is, the concentration of readily assimilable carbon should not vary significantly. The carbon source material most often used in the past at wastewater treatment plants has been methanol.

Methanol has a very high concentration of soluble (readily assimilable) organic carbon, is uniform in composition, exhibits high denitrification rates, produces less excess biological cell growth than most other carbon sources and is readily available as a market product at a reasonable cost. However, there is some concern with the use of methanol in a denitrification process where the process effluent will be discharged to the groundwater without further treatment. The possibility exists that the quantity of methanol fed to an anoxic reactor could exceed the organic carbon requirements of the process and therefore some of the methanol would pass through the anoxic reactor and be discharged to the groundwater. This is of concern because of the toxic nature of methanol.

Many researchers have investigated the use of various other sources of external carbon for denitrification[1], including commercially produced chemicals and various industrial process byproducts and wastewaters.

Commercially produced carbon sources include chemicals such as Acetic Acid, Citric Acid, Ethanol, Ethyl Acetate, Isopropanol, Lactic Acid, Propylene Glycol, Sodium Acetate, Corn Syrup, and Sugar. Chemical and physical characteristics for some of the commercially produced chemicals that can be used as a carbon source are given in the following table,

TABLE XI-1

Chemicals Cited as External Carbon Sources for Denitrification of Nitrified Wastewater

Chemical Name	Formula	(L), (S)[1]	Molecular Weight	% Carbon	Sp. Gr. @ 20°C	Solubility in Water
Acetic Acid (Glacial)	$C_2H_4O_2$	L	60.05	40.0	1.05	Miscible
Cane Sugar (Sucrose)	$C_{12}H_{22}O_{11}$	S	342.3	42.0	1.587	(2)
Citric Acid (Anhyd.)	$C_6H_8O_7$	S	192.12	37.51	1.665	162g/100 ml*
Ethanol (SDA 35A)[3]	C_2H_6O	L	46.07	52.14	0.796	Miscible
Ethyl Acetate	$C_4H_8O_2$	L	88.10	54.53	0.902	1ml/10 ml*
Isopropanol	C_3H_8O	L	60.09	59.96	0.8	Miscible
Lactic Acid	$C_3H_6O_3$	L	90.08	40.00	1.12-1.23	Miscible
Methanol	CH_4O	L	32.04	37.48	0.796	Miscible
Propylene Glycol	$C_3H_8O_2$	L	76.09	47.35	1.036	Miscible
Sodium Acetate (Anhyd.)	$C_2H_3NaO_2$	S	82.04	29.28	1.53	76g/100 ml**

* @ 25°C

** @ 0°C

(1.) L = Liquid, S = Solid

(2.) Solubility in water; 179.2 g/100 g @ 0°C, 190.6 g/100g @10°C, 203.8g/100g @ 20°C

[1] McCarty et al.-1969; Sollo et. al - 1976; Driscoll and Bisogni-1978; Monteith et al.-1980; Skrinde and Bhagat-1882; Beauchamp et al-1989, Paul, et al.-1989; Christensson, et al.-1994; Cuervo et al.-1999.

Industrial byproducts and wastewaters that have been suggested for use as carbon sources include corn silage derivative, brewery and distillery wastes; cellulosic materials such as coconut shells, pecan shells, sawdust and wood chips; used newspaper, residuals from molasses production, and residuals (such as whey) from production of dairy products.

The attractive advantages associated with using industrial wastes as a source of carbon are that it results in the degradation of the waste while aiding in the removal of nitrogen from wastewater, and it may in some cases be the least expensive carbon source.

However, there are problems associated with the use of industrial wastes and byproducts as carbon sources. These include:

- A lower concentration of readily assimilable carbon than commercially produced carbon sources.
- A lack of uniformity (consistency) in carbon concentration.
- Contamination with undesirable substances (toxic metals, organic chemicals toxic even in trace amounts, etc.).
- May not be permanently (continually) available.

Therefore, where industrial waste/byproducts are proposed as sources of carbon, the Department will review them on a case -by-case basis. If the Department approves the use of such a source, it may require that the enhanced pretreatment facilities be designed to permit changeover to commercially produced carbon sources in the event the originally proposed carbon source becomes unavailable or unsuitable.

The Department will also review proposed use of commercially available carbon sources with respect to their toxicity to human health and the environment and use of any such source may not be made without receiving approval from the Department.

In selecting an external carbon source, it is important to consider the soluble carbon content, the energy obtained by the microorganisms during the oxidation-reduction reactions, the denitrifier growth rate (amount of biomass produced), and any threat to the public health or the environment that may result from impurities contained therein.

Sufficient carbon should be provided in excess of the stoichiometric requirement determined from balanced chemical reaction equations to insure that the denitrification process will be nitrate-limited rather than carbon limited. For example, the actual use of methanol has been reported to range from 1.3 to 1.6 times the stoichiometric amount. Similar ratios have been reported for ethanol. Excess carbon source remaining after denitrification is complete should be removed in an aerobic reactor following the anoxic reactor.

It should be noted that completion of the denitrification process results in the recovery of approximately 50% of the alkalinity consumed in the nitrification process.

C. Approval of Enhanced Pretreatment Processes and Equipment

1. General

The Department will approve enhanced pretreatment processes and equipment on a case-by-case basis where the need for enhanced pretreatment has been demonstrated to the satisfaction of the Department. Approval will be based on the enhanced pretreatment effluent quality goals and the demonstrated ability of the proposed processes and equipment to attain those goals.

Enhanced pretreatment processes for $CBOD_5$ and nitrogen reduction that have been approved by the Department include suspended growth processes, fixed film processes, and hybrids that combine both fixed film and suspended growth processes. Those enhanced pretreatment processes and associated equipment that have been utilized in applications that were approved by the Department in the past include [2]:

- Amphidrome™ (fixed film sequencing batch bioreactor usually preceded by septic tank(s) with or without preceding grease trap(s)).
- Bioclere™ (fixed film bioreactor usually preceded by septic tank(s) with or without preceding grease trap(s)).
- Clarigester™ (A primary clarifier, with a lower compartment used to store solids in a manner similar to a septic tank)
- Disinfection facilities, including ozonation and ultra-violet irradiation facilities.
- Extended Aeration facilities, packaged type (suspended growth aerobic bioreactors followed by clarifiers) preceded by septic tank(s) with or without preceding grease trap(s).
- FAST™ (hybrid system incorporating both fixed film and suspended growth processes, usually preceded by septic tank(s) with or without preceding grease trap(s)).
- Filtration facilities, including rapid rate granular single media, dual media or multi-media type filters, or cloth media disc filters, normally following the processes listed above.
- Recirculating granular media filters (RGMF) following basic pretreatment in septic tank(s) with or without preceding grease trap(s). These include recirculating filters using either a sand media, in which case they are referred to as Recirculating Sand Filters (RSF), or a gravel media, in which case they are referred to as Recirculating Gravel Filters (RGF). These have been of a generic type as compared to packaged or pre-manufactured enhanced pretreatment facilities.
- Rotating Biological Contactors [RBC] (fixed film bioreactors operating in an aerobic mode, anoxic mode, or both, followed by clarifiers), preceded by septic tank(s) with or without preceding grease trap(s).

[2] The mention of trade names, trade marks, proprietary products and processes and does not constitute endorsement or recommendation of use by the Department unless specifically stated otherwise herein.

- RUCK system® (fixed film system incorporating an intermittent sand filter and upflow packed bed anoxic reactor, preceded by septic tank(s) with or without preceding grease trap(s)).
- Zenon Cycle-Let® (membrane bioreactor (MBR) suspended growth system followed by additional advanced treatment processes, preceded by a septic tank or trash tank with or without preceding grease trap(s)).

Notwithstanding the listing of enhanced pretreatment facilities that have been previously approved, the Department reserves the right to approve or disapprove any proposed enhanced pretreatment facilities based on the merits of each individual application and for good cause shown.

D. Design of Enhanced Pretreatment Systems

1. General

Most enhanced pretreatment systems used in onsite wastewater renovation systems (OWRS) consist of one or more commercially produced prefabricated units containing all necessary operating equipment, piping internal to the equipment, and tankage. The exceptions to this are the generic recirculating granular media filters that are constructed on-site. Therefore, the design of prefabricated systems is approached from a somewhat different perspective than the design of the much larger centralized wastewater treatment systems.

In the design of the larger centralized wastewater treatment plants, considerable effort is usually made to establishing a very substantial and detailed database on anticipated wastewater flows and characteristics. In some cases, bench top and pilot studies are conducted to develop detailed process data (e.g. microbial growth, decay, and reaction rates, hydraulic detention time, solids retention time, sludge settleability studies, etc), that will then be used for actual design of reactor tankage, and sizing of piping and equipment. This is a "luxury" that is not usually available to the engineer responsible for designing enhanced pretreatment facilities for an OWRS, due to budgetary restraints.

It also is a fact of life that prefabricated "packaged" treatment systems are usually more cost-effective options for such facilities, both in terms of the procurement cost of the systems and their installation on the project site. Usually all that is required to install the prefabricated "packages" is to place them in the correct locations and at the correct elevations and connect them to the site sanitary sewers, electrical power supply and SWAS. (This last statement is an oversimplification, of course, but is used to emphasize the difference between a packaged treatment system and one that is constructed on-site from basic materials and equipment.)

However, there is a downside to the use of packaged wastewater treatment facilities. This equipment is often designed in a modular fashion to encompass incremental ranges of flow rates and wastewater characteristics and thus cannot be tailored to the particular application in mind. Also, the turndown ratio for such facilities (the ability to operate properly at flows and organic loadings considerably lower than the design flows and loadings) may be limited, although this limitation can often be remedied by using flow equalization and/or multiple units.

Further, the engineer is depending upon the manufacturer's technical staff to select the correct model for his project that will provide the desired effluent water quality. In some cases, the design criteria used by the manufacturer's staff may be somewhat optimistic as compared to the results obtained under "real life" situations. The engineer designing an OWRS that will incorporate packaged wastewater treatment facilities should inquire as to the criteria used for design of these facilities and the treatment efficiency to be expected. This information should be compared with published results found in the literature for the same type of system, wastewater flow and loading.

Reliable manufacturers and vendors of packaged wastewater treatment facilities will request certain information about the flow rates and nature of the wastewater to be treated. The type of information requested may or may not encompass all of the variables that the engineer may wish to be considered in the design of the treatment facilities. Therefore, it is incumbent upon the engineer responsible for designing the enhanced pretreatment facilities to ensure that the manufacturer/vendor are provided with full information on the flows and biochemical characteristics of the wastewater and the ambient conditions at the site where the treatment facilities will operate. The engineer should also make known his preference for the types of equipment and auxiliary facilities he wishes to have incorporated into the packaged unit.

2. Information Provided to Manufacturers and Vendors

The following information should be provided by written communication to the manufacturer/vendor:

- The types of processes required.
- Average Daily Design Flow Rate at Design Year and during initial years of operation.
- Maximum Daily Design Flow Rate at Design Year and during initial years of operation.
- Maximum Hourly Design Flow rate at Design Year and during initial years of operation.
- Minimum Daily Design Flow rate at Design Year and during initial years of operation.
- Expected seasonal variation in the flow rates listed above.
- The number of hours of each day wastewater will be received at the treatment facility
- Types of facilities to be served (sources of wastewater) and influent wastewater characteristics, including daily average and peak values for:
  - Carbonaceous Biochemical Oxygen Demand ($CBOD_5$) (Total and Soluble)
  - Chemical Oxygen Demand (COD) (Total and Soluble)
  - Total Solids
  - Total Suspended Solids (TSS)
  - Total Nitrogen (generally, ammonia-N and organic N, but also include nitrites/nitrates if expect to be present in other than trace amounts)
  - Total Phosphorus
  - pH,
  - Total Alkalinity, as $CaCO_3$
  - Sulfides
  - Seasonal wastewater temperatures (max, min, avg.)
- Seasonal ambient air temperatures (max, min, mean)
- Altitude and seasonal variation of relative humidity at plant location (required for design of diffused air systems)
- Site Constraints (available area, site access, etc.)

- Electrical Power Supply Characteristics (e.g.: single or three phase, voltage)
- Required Effluent Water Quality, including:
  $CBOD_5$
  TSS
  TN
  TP
  pH
- The choices of any alternates of materials or equipment offered by the manufacturer.
- Any special requirements for interior and exterior surface coatings and other corrosion protection of the reactor and any components that will be submerged in the wastewater and mixed liquor.
- Any special guarantee requirements. (See subsection O for a further discussion of such requirements.

3. Enhanced Pretreatment at Seasonally Operated Facilities

Where the facilities to be served are operated on a seasonal basis, the processes selected should be easy to start up and maintain in a stable condition under low microbial mass conditions. Suspended growth bioreactors can be "seeded" with microorganisms obtained from a similar type of bioreactor operating in a stable condition and treating the same type of wastewater. This involves transferring the requisite volume of mixed liquor suspended solids (MLSS) from the operating plant to the seasonal plant when restarting the process.

An example of a fixed film process that can be used in such instances is the recirculating granular media filter because it is an inherently stable process. Both start-up and shutdown are simple to accomplish and there is less likelihood that any startup problems that are encountered will cause problems with any downstream processes or the SWAS due to escape of suspended solids. It may take several weeks for fixed film media to develop the required microbial population that is necessary for an effective biological process. Therefore, it will be necessary to seed the process by importing septic tank effluent from another similar facility so that the process will be up and running when needed. Such seeding may not be necessary if even a small amount of wastewater is being treated in the off-season.

4. Suspended Growth Aerobic Systems.

Suspended growth aerobic systems used for treatment of wastewater employ activated sludge processes in a mixed and aerated bioreactor (aeration tank). The liquid contained in a suspended growth bioreactor is termed "mixed liquor" and consists of a mixture of wastewater and suspended and colloidal solids, including biodegradable and non-biodegradable solids, and suspended microorganisms (biomass) that are collectively referred to as the mixed liquor suspended solids (MLSS), or "activated sludge".

The biomass actually consists of both living and dead microorganisms and thus only a fraction of the biomass is actively involved in waste removal or stabilization. The biomass actually responsible for waste removal or stabilization is included in the volatile solids, referred to as mixed liquor volatile suspended solids, or MLVSS.

The mixed liquor is stirred to maintain intimate contact between the MLSS and the contaminants in the wastewater and provided with sufficient oxygen to maintain the metabolism of the microbes as they oxidize the organics and nitrogen in the wastewater.

The stirring also helps the individual microbes in the biomass to flocculate so that they can be removed from the liquid in which they are suspended by the subsequent clarification process, for without adequate flocculation the individual microorganisms would be very difficult to remove.

In order to maintain a balanced process, excess biological growth resulting from the reproductive capacity of the microbes, termed "waste activated sludge (WAS)", must be removed from the reactor via settling (sedimentation) in clarifiers and wasted to side-stream waste sludge processes.

Oxygen is usually introduced into the bioreactor via air diffused into the liquid by low-pressure air blowers discharging to diffusers submerged within the bioreactor, with the diffused air also providing the required mixing. Other methods of introducing air and mixing involve mechanical aerators, jet aeration and the use of submerged turbine aerators. By far the most common method used in small packaged plants involves air blowers and submerged diffusers.

a. Extended Aeration

There are several modes of the activated sludge (AS) process. However, in most cases, where suspended growth aerobic systems are used as enhanced pretreatment in on-site wastewater renovation systems, the extended aeration (EA) mode is selected.

The BOD_5 removal efficiency of the EA process is $\geq$ 95 % for a properly designed, operated and maintained facility and almost complete oxidation of ammonium (residual ammonium normally $\leq$ 1 mg/L) is also obtained except when temperature of the liquid in the reactor drops below 10°C. Nitrification is thus inherent in the EA process under normal operating conditions.

The EA process is relatively simple to operate and also produces the least WAS. Generally, no primary treatment is provided except for removal of large solids, which can be accomplished in a septic tank or in screening facilities that precede the EA facility. Where a septic tank does not precede the EA process, the large solids must be removed by screening or by reducing the size of the solids (comminution).

The EA process can be accomplished in a continuous flow reactor, or in a sequencing batch reactor (SBR). If a continuous flow reactor is selected, sedimentation must be accomplished in a separate clarifier, where the activated sludge is allowed to settle under relatively quiescent conditions. Most of the settled sludge is then returned to the bioreactor (returned activated sludge, RAS) while some is wasted (WAS) to side stream facilities for further processing. A very small fraction (usually less than 1%) escapes as suspended solids in the wastewater discharged from the clarifier. If a SBR is used, there is no need of a separate clarification stage, as settling is accomplished in the SBR reactor during periodic shutdown of the mixing/aerating cycles.

Factors that must be considered in the design of the EA activated sludge process for domestic wastewater, besides effluent water quality requirements, include:

- Selection of reactor type (continuous flow reactor, sequencing batch reactor),
- Loading criteria (F/M ratio),
- Solids retention time (SRT),
- Hydraulic retention time (HRT),
- Reactor volume and freeboard,
- Oxygen requirements and method of oxygen transfer to the mixed liquor,

- Separation of MLSS from the wastewater,
- RAS flow rate, as a percentage of wastewater flow rate,
- WAS flow rate, in volume per day,
- Alkalinity of the Mixed Liquor, and
- Temperature of the Mixed Liquor.

The selection of the type of process (es) and reactor(s) will depend upon the design flow rates, the effluent characteristics required, relative ease of operation and maintenance, the space available for the reactor and auxiliary facilities, economics, and designer's choice. Where space is limited, a sequencing batch type of reactor (SBR) or membrane bioreactor (MBR) may be desirable, since the biological processes and clarification can take place in the same reactor structure. Chemicals can also be added to these reactors for removal of phosphorus if required.

Another type of bioreactor sometimes used in the EA process is the oxidation ditch, but it is seldom used for the small scale plants involved in enhanced pretreatment for on-site wastewater renovation systems.

The biological loading of an activated sludge process is expressed as the food to microorganism ratio (F/M), in mass units, with the food being the organic matter in the wastewater and the microorganisms being the MLVSS, which are assumed to represent the active mass of microorganisms present. (This assumption is not exactly correct, as the percent of the total MLVSS that is biologically active is an important consideration in the process).

While the EA process can operate at F/M ratios of 0.10 or less, in most cases a F/M ratio of 0.05 or less is utilized. Operation of the activated sludge process at F/M ratios of 0.10 or less results in the microorganisms existing in the endogenous respiration phase, where the microorganisms have little food and consequently use their own cellular material as a food source. This results in a maximum oxidation of organic and nitrogenous compounds and a minimum of excess sludge to be wasted and processed. Any further stabilization of the WAS required is usually accomplished in aerobic sludge digesters. Because of the large aeration volume used and the long solids retention time, the process is not easily upset. However, once upset, it may take some time to restore the process to a stable operating condition.

Volumetric loading is another method of expressing the organic loading. Experience has indicated that a volumetric loading range of 5-15 lb. BOD_5 per 1000 cu. ft. of reactor volume per day is suitable for EA plants. This provides a check on the volume determined on the basis of the hydraulic retention time.

The Solids Retention Time (SRT) is the average length of time that a microorganism remains in the aerobic bioreactor. It is calculated by dividing the mass of solids in the reactor (concentration of solids in the reactor x the volume of the tank) by the mass of solids wasted per day (gallons per day of WAS x concentration of WAS), resulting in units of days. Proper attention must be given to the units of measure to correctly calculate the SRT. The SRT for the EA process will normally range from less than 20 to 30 days or more, with a shorter SRT being used in the warmer summer months because of the higher microbial growth and reaction rates that occur at higher temperatures.

The SRT is a critical operating parameter in the activated sludge process and must be maintained in a relatively narrow range to maintain the process in stable operation. If the WAS rate is too low, there will be a buildup of MLSS in the bioreactor and this will affect the types of organisms present and can severely hinder the ability to remove the MLSS from the treated wastewater in the clarifier. If the WAS rate is too high, there will be a decline in the MLSS resulting in a "washout" of active biomass that will adversely affect the ability of the process to efficiently oxidize all of the organic and nitrogenous compounds in the wastewater. Washout is of particular importance if nitrification is required, as the autotrophic nitrifying bacteria are only a small fraction of the overall biomass, have a slower growth rate than heterotrophic bacteria and thus their loss must be avoided.

The Hydraulic Retention Time (HRT) is the average time the wastewater remains in the bioreactor for treatment. It is calculated by dividing the volume of the aeration tank by the wastewater flow rate and is usually expressed in units of hours. In the EA process, the HRT will normally be in the range of 16 - 24 hours or greater. It should be noted that contrary to the SRT, which can be varied by the plant operator by adjusting the WAS rate, there is no control over the HRT once the volume of a continuous flow aerobic bioreactor has been established. On the other hand, both the SRT and the HRT can be varied if the bioreactor is a SBR. This is one of the benefits of a SBR.

The reactor volume is determined from the design flow and the desired HRT and is calculated by multiplying the hydraulic detention time by the design wastewater flow rate, is expressed in units of liquid volume, and does not include any freeboard allowance. Therefore, a freeboard of at least 18 inches should be provided above the operating liquid level to contain any froth or scum that may develop on the surface of the mixed liquor. As in other similar calculations, attention must be given to the units of measure to correctly calculate the volumetric requirement.

The oxygen requirements for the EA process are the highest of all the activated sludge process modes. This is because the oxidation of organics is carried out essentially to completion, meaning that the ultimate BOD (UBOD) must be met, rather than the 5-day BOD. The organic ultimate oxidation demand is equivalent to approximately 1.3-1.5 lb. oxygen per lb. of BOD_5. In addition, the nitrification that inherently occurs in the EA process results in a high oxygen demand, as approximately 4.6 lb. of oxygen are required per lb. of nitrogenous compounds oxidized. Thus, for example, for a septic tank effluent having a BOD_5 concentration of 175 mg/L and a TN concentration of 30 mg/L, the combined organic and nitrogenous oxygen demands are estimated to be equivalent to about 2.1-2.3 lb. of oxygen per lb. of BOD_5 in normal domestic wastewater. For wastewaters of higher organic strength or nitrogen content, the total oxygen requirement should be calculated using the separate oxygen demands for organics and nitrogen.

As previously discussed, oxygen transfer to the mixed liquor in the EA bioreactor is usually accomplished by means of diffused air. Considerable effort is expended in the design of large activated sludge plants in evaluating the various types of air diffusers (e.g. fine bubble, coarse bubble) for their efficiency in diffusing the air into the mixed liquor. This is done in an effort to minimize the electrical power costs involved in delivering the air to the diffusers. Consideration is also given to the costs of maintaining the diffusers, with fine bubble diffusers usually requiring higher maintenance costs.

However, because of the relatively small size of the EA plants involved in enhanced pretreatment for on-site wastewater renovation systems, the coarse bubble type of diffuser is normally provided. While the oxygen transfer efficiency of the coarse bubble diffusers is considerably lower than that of fine bubble diffusers, the latter are more susceptible to clogging, and thus the maintenance of coarse bubble diffusers is less involved. This is a major factor in small EA plants where a sole plant operator is usually responsible for both operation and maintenance and may only be present for a relatively short time each day. The type of blowers used to deliver the air to the diffusers in small EA plants (rotary, positive displacement type) are also different than those used in large AS plants (centrifugal type) because of smaller volume of air required to meet the dissolved oxygen requirements of the process.

Separation of the MLSS from the wastewater is one of the most important factors in the AS process and can sometimes be particularly troublesome in a continuous flow type of EA plant where a separate clarifier is required.

Parameters that affect the performance of a clarifier are:

- The settling characteristics of the MLSS,
- The dissolved oxygen (D. O.) concentration in the MLSS,
- The mass (solids) loading rate,
- The liquid loading rate,
- The weir overflow rate,
- The depth of the clarifier below the weir level (sidewater depth),
- The provisions (or lack thereof) for removal of floating solids and scum,
- The RAS rate,
- The WAS rate,
- The temperature of the mixed liquor, and,
- Provisions for rapid collection and removal of the settled MLSS (sludge).

The only parameters under the control of the plant operator, once the clarifier has been designed and constructed, are the settling characteristics of the MLSS, the D. O. concentration, and the RAS and WAS flow rates. Therefore, careful design of the clarifier is quite important to the overall activated sludge process.

The solids loading rate is determined by dividing the total solids applied to the clarifier by the surface area of the clarifier, and is expressed as mass of solids (MLSS) per unit area per unit of time (e.g. lb. MLSS./sq. ft./hr). The liquid loading rate is expressed as liquid volume per unit area per unit time (e.g. gallons / sq. ft./day). The weir overflow rate is expressed as liquid volume/unit length of weir/unit time (e.g. gallons/lf of weir/day). For EA packaged plants, the design parameters recommended in the literature for clarifier design (NEIWPCC-1998; Crites and Tchobanoglous-1998) are as follows:

Clarifier Loading Rates:

Loading	Average	Peak
Solids, lb./sf/hr	0.5	1.2
Liquid, g/sf/d	200	450
Weir, g/lf/d	< 10,000	≤ 20,000

The clarifier sidewater (liquid) depth should be not less than 10 ft., not including freeboard allowance. Adequate baffles should be provided where the mixed liquor enters the clarifier to prevent velocity currents from disturbing the settling process and provisions should be made for rapid removal of the settled MLSS and for removing scum and floating solids from the surface of the clarifier.

The RAS and WAS flow rates are critical to the operation of all activated sludge process modes. The return rate for the EA process is highest of all the activated sludge modes. In addition to selection of the RAS rate, the method of transferring the RAS from the clarifier is quite important. Some EA packaged plants use airlifts for removing the RAS and WAS from the clarifier and pumping it back to the bioreactor. However, the ability of an airlift to provide a reasonable variation in flow rate is problematic because of the limited turndown ratio; therefore the use of pumps is preferred.

<u>RAS Rate:</u> Capacity to vary from 30 to 150% of Average Daily Flow.

<u>WAS Rate:</u> Capacity to vary up to 25% of Average Daily Flow.

<u>Type of Pumps</u>: Solids-handling centrifugal sewage pumps.

<u>Velocity of flow</u> in RAS and WAS piping: $\geq$ 2 ft/sec.

The alkalinity in the wastewater helps to control the pH of the mixed liquor, particularly in the case of nitrification where acids are formed during the oxidation of ammonium to nitrates. If there is insufficient alkalinity in the wastewater to buffer the pH changes, provisions must be made for addition of an alkali to the mixed liquor.

The temperature of the mixed liquor is an important factor. Temperatures below 20°C adversely affect the oxidation rates while temperatures below 15° C adversely affect the nitrification rates. Below 10°C, there is a considerable adverse affect on both oxidation and nitrification rates although some oxidation and nitrification will continue at temperatures of 5°C or less. Temperature of the mixed liquor will also affect the settling rate in the clarifier due to viscosity effects. Temperature also enters into the design of the aeration facilities because of its affect on the dissolved oxygen content in the mixed liquor and the design of the air blowers. Therefore, the seasonal variations in ambient air temperatures must be taken into account in the design of the facilities.

The most troublesome problems with clarifiers serving continuous flow bioreactors have to do with formation of foam and scum and bulking of the MLSS. Foaming and scum formation often occurs on bioreactor and clarifier surfaces. A light colored froth or foam usually forms during the initial start-up of continuous flow bioreactors, but this is only transitory and will normally not remain a problem once the MLSS has reached the design concentration. Foam suppression systems consisting of a piping system equipped with nozzles that spray plant water (clarified effluent) onto the foam will usually control this foam.

An excess of filamentous organisms (generally some types of bacteria and fungi) causes a more persistent foaming problem that can occur both in the bioreactor and clarifier. A well-formed biomass in the mixed liquor (MLSS) consists mostly of bacteria with a small percentage (~5%) of the higher life forms such as protozoa and rotifers, and these organisms will tend to cluster into floc particles that settle out of the mixed liquor during the clarification process. Some filamentous organisms in the biomass are useful in that they tend to bind the small floc particles into larger and stronger ones, enhancing the settleability of the MLSS. If no filamentous organisms are present, the floc will tend to be very small in size and subject to being broken up during the aeration process, resulting in problems with producing a clear clarifier effluent. However, when they become excessive, filamentous organisms cause persistent foaming problems and bulking of the MLSS floc in the clarifier, hindering the settling process in the clarifier.

The foam that results from excessive filamentous organisms is usually a viscous, brown-colored foam that is difficult to break up in the bioreactor with the typical water spray type of foam suppression system. Such foam, if discharged from the bioreactor to the clarifier, can form to such a depth as to overflow the final clarifier weirs, resulting in a high concentration of suspended solids in the effluent and the loss of biomass. When this occurs the activated sludge process will be adversely impacted, to the detriment of any downstream processes, and the effluent quality will be degraded.

Therefore, a means of controlling the formation of this foam must be included in the design of an activated sludge process. Controlling the amount (concentration) of filamentous organisms will also control bulking of the MLSS in the clarifier, as bulking is usually caused by too many filamentous organisms that produce a diffused floc with poor settling properties.

Bulking can result in the settled MLSS rising and overflowing of the clarifier weirs. This is the so-called “burping” effect that results in a loss of viable biomass, a poor quality effluent, and severe problems with downstream processes such as filtration and disinfection.

Provisions should be made for controlling the formation of foam, including the collection, removal and disposal of the foam from the bioreactor. It has been found that introducing a chlorine solution into the foam will often result in controlling the growth of the filamentous organisms. This method of foam control is problematic where the treated wastewater will be discharged to the subsurface because of the concern with formation of toxic byproducts of the chlorination process and the persistence of these byproducts in the ground water.

Most of the filamentous microorganisms are obligate aerobes that thrive in conditions where dissolved oxygen is present in concentrations too low to support other types of aerobic bacteria. Thus, one natural method of controlling their growth is to expose them to anoxic or anaerobic conditions. Under such conditions the competition of facultative and anaerobic microorganisms for the available food supply will result in a substantial reduction of the filamentous microorganisms.

In large activated sludge plants, this is accomplished in reactors termed “selectors” that precede the main bioreactor. This same procedure can be used where small packaged activated sludge facilities are used. The selector can also serve as an anoxic reactor to denitrify the MLSS. It is also possible to use aerobic selectors, where a relatively high D.O. concentration is maintained. This favors the growth of aerobic microorganisms with better flocculating and settling properties than the filamentous microorganisms.

Where equalization is provided following the septic tank, the equalization tank might also be designed to serve as the “selector” provided its capacity is increased to accomodate the recycled flow and proper baffling and a means of mixing the tank contents are provided. In the same manner, an equalization tank can be designed to also serve as an anoxic reactor when nitrogen removal is part of the treatment requirements provided additional volume is made available for the denitrification process. Thus, an equalization tank upstream of an aerobic bioreactor can be designed to serve several useful purposes.

It should be noted that rising sludge can also be caused where nitrification has occurred in the aerated bioreactor if the settled MLSS (sludge) is allowed to remain too long in the bottom of the clarifier. Under such conditions denitrification will occur and the nitrogen gas, rising as it is released, will cause clumps of sludge to float to the top of the clarifier and escape over the effluent weirs. To prevent this from occurring, it is important that the settled sludge be removed (as RAS and WAS) from the bottom of the clarifier as quickly as possible. Thus the clarifier must be provided with an effective means of rapidly collecting and removing of the sludge.

b. Sequencing Batch Reactors

The sequencing batch reactor (SBR) process is a variant of the activated sludge process that operates on a batch basis. All phases of the SBR process take place in sequence in the same reactor instead of the wastewater moving through a series of reactors and clarifiers for completion of the treatment process phases, as is the case in the usual activated sludge treatment process.

There are two types of sequencing batch reactors. In the standard fill and draw SBR operation, flow through the reactor is halted during the phases where the mixed liquor is allowed to settle and during decanting of the clarified wastewater.

In the other type of operation, referred to as the intermittent cycle extended aeration system (ICEAS), flow is continuous through the SBR reactor during all of the batch treatment phases. The SBR process is very versatile, in that oxidation of organics and ammonia-nitrogen, denitrification, phosphorus removal and clarification can take place in the same reactor.

In the standard SBR mode of operation, where flow to the reactor is halted during the settle and decant phases, ideal conditions are provided for clarification of the SBR effluent under quiescent conditions. In the ICEAS mode of operation conditions for clarification of the SBR effluent are not as ideal as for the case where flow is interrupted during the settle and decant phases. However, both modes provide a very good effluent quality.

The standard SBR mode of operation involves five basic phases, including FILL, REACT, SETTLE, DECANT and WASTE. A sixth phase (IDLE) may be used when the flow to the treatment facilities is considerably below the design flow. Control of the process is via a microprocessor (programmable logic controller, or PLC). The FILL phase can be divided into sub-phases, such as STATIC FILL (no mixing), MIXED FILL and REACT FILL, to accomplish certain treatment objectives. All of these SBR phases are described below.

The STATIC FILL sub-phase begins with a certain amount of biomass in the SBR reactor, with the DECANT or IDLE phase having just been completed. At this time, the biomass exists as a layer of MLSS at the bottom of the reactor, having accumulated there during the previous SETTLE and DECANT phases. A layer of clarified liquid exists above the solids layer, and the water quality in the upper portion of this liquid layer is essentially the same as the quality of the SBR effluent that has just been decanted. The layer of biomass provides the means to initiate the biodegradation of the pollutants contained in the incoming batch of wastewater.

During the STATIC FILL sub-phase, wastewater is introduced at the bottom of the SBR reactor without mixing. This sub-phase is used as a means to condition the existing biomass by reducing the number of filamentous microorganisms that may be present in the mixed liquor. As previously stated under subsection D.4a, excessive amounts of filamentous microorganisms can hinder settling of the biomass.

In the STATIC FILL sub-phase, the food supply (pollutants present in the wastewater) is high, the dissolved oxygen content is very low, and this favors the growth of a mass of facultative bacteria. The facultative bacteria multiply more rapidly than filamentous organisms and in doing so take in and store most of the available food supply, thus in effect "starving" the filamentous organisms. Thus the STATIC FILL sub-phase acts as a means of "selecting" the type of microorganisms desired.

During the MIXED FILL sub-phase, the influent wastewater is rigorously mixed with the existing contents of the SBR reactor without adding oxygen. Thorough mixing of the influent wastewater with the mixed liquor allows removal of organics and conversion of organic nitrogen to ammonia nitrogen to take place under these conditions by the action of the facultative bacteria in the biomass. Denitrification of nitrates that may be present in the mixed liquor can also occur during this phase.

The REACT FILL sub-phase consists of continuing to feed wastewater into the SBR reactor and mixing the reactor contents while aerating the mixture to dissolve oxygen into the mixed liquor. The dissolved oxygen supports the oxidation processes of the facultative fractions of the biomass that convert organic pollutants to carbon dioxide, water and new microbial cell mass, and the nitrification processes of the aerobic nitrifier fraction of the biomass in the oxidation of the ammonia-nitrogen to nitrates.

The REACT phase (aerobic phase) consists of continuing to mix and aerate the mixed liquor in the SBR reactor without feeding of the wastewater to the reactor. Further removal of organics and nitrification of ammonia-nitrogen takes place in this phase until oxidation of the organic pollutants and nitrification of the ammonia-nitrogen present is essentially complete.

It should be noted that the removal of organics takes place throughout most of each batch treatment FILL and REACT sub-phases, while nitrification and denitrification take place only in certain of the sub-phases of each cycle. To accomplish nitrogen removal using the influent wastewater as a carbon source for the denitrifiers, the FILL and REACT sub-phases are alternated several times within one batch treatment cycle by turning the source of oxygen on or off. During each aerobic period some of the organics are oxidized and ammonia-nitrogen is nitrified, while during each period when no oxygen is being supplied the SBR reactor reverts to an anoxic condition and the nitrates formed during the aerobic period are denitrified and some additional removal of organics takes place.

The SETTLE phase consists of stopping the mixing and aeration of the mixed liquor and allowing the biomass to settle to the bottom of the SBR reactor. One of the major advantages of the standard SBR system is the creation of essentially perfect quiescent conditions during the SETTLE phase. During this phase, there is no inflow of waste, no mixing and no aeration occurring. This permits rapid settlement of the suspended bio-mass without any disturbance.

During the DECANT phase, the fully treated and clarified wastewater is skimmed from the upper portion of the clarified liquid in the SBR reactor by floating decanters without disturbing the settled bio-mass at the bottom of the reactor.

The WASTE phase can take place near the end of the DECANT phase, or during the IDLE phase, when the biomass layer at the bottom of the reactor has reached its highest concentration. During the WASTE phase, some of the biomass is removed from the reactor by pumping settled solids in the bottom of the reactor to sludge processing facilities. This wasting is done to remove excess biomass that has grown during the various FILL, MIX and REACT phases so as to maintain the correct amount of biomass in the reactor.

The IDLE phase is not needed as an operational phase but is rather that period between completion of the DECANT phase and the beginning of the next FILL phase. The IDLE phase only occurs when actual flows are substantially less than the design flows. (During an extended IDLE phase, it may become necessary to periodically aerate the mixed liquor to prevent it from becoming anaerobic.)

The various FILL, MIX and REACT phases, the SETTLE phase and the WASTE phase are controlled on a time basis, and the time spent in each phase can be readily adjusted to suit required operating conditions by reprogramming the PLC. The actual time of the DECANT phase is usually controlled by the liquid level in the reactor following completion of the various FILL, MIX and REACT phases. The liquid level in the reactor can also be readily changed to suit required operating conditions.

Also, when unusual peak flows occur, such as from extraneous inflow during storm conditions, the SBR programmable logic controller can be set to reduce the cycle times so as to be able to accommodate the increased flow. Where a single tank SBR system is used, a flow equalization basin is required upstream of the SBR to store the influent flow during the react, settle, decant & waste phases of each cycle. Because of the intermittent high rate of discharge that occurs during the decant cycle, an equalization tank is also usually provided following the SBR reactor(s) so as to avoid over-sizing of downstream facilities (e.g.: filtration, disinfection).

In the ICEAS mode of SBR operation, the reactor is separated into two zones, PREREACT and MAIN REACT, by a baffle wall. The wastewater flows continuously into the PREREACT zone, which acts as a selector to limit the growth of filamentous microorganisms. The mixed liquor from the PREREACT zone flows through openings in the baffle wall and into the MAIN REACT zone. Oxidation of organics and ammonia-nitrogen, and denitrification, is accomplished in the MAIN REACT zone by alternating on-off periods of air diffusion into the mixed liquor that produce periods of aerobic and anoxic conditions.

After a period of time, aeration is stopped and a SETTLING phase is initiated, allowing the MLSS to settle to the bottom of the reactor, leaving a layer of clear water on top. A floating decanter removes the uppermost clear water from the reactor. Excess biomass is periodically removed from the bottom of the reactor. The ICEAS SBR purportedly requires a smaller reactor volume than a standard SBR.

Phosphorus can be removed in either mode of SBR operation by either biological or chemical methods, or both, depending upon the degree of removal required. The air on/off cycles can be managed to provide an anaerobic condition for biological removal of P. Where an effluent P concentration ≤ 1 mg/L is required, chemical addition is usually necessary. The SBR process is capable of producing an effluent with concentrations of BOD_5, TSS and TN ≤ 10 mg/L.

c. Membrane Bioreactors

A membrane bioreactor (MBR) consists of a continuous-flow suspended growth (activated sludge) bioreactor coupled with a membrane micro-filtration system. The key feature of a MBR is the employment of a low pore size membrane for high efficiency solids separation. Thus, an MBR unit replaces the conventional arrangement of separate bioreactor, clarifier and post-filtration facilities. This eliminates the sludge settling process and allows elevated levels of biomass to be utilized without causing problems with poor settling of the MLSS. A MBR can be operated with MLSS concentrations of 15,000 mg/L or higher, and at a long SRT, resulting in low overall sludge generation. The MBR process operates with the biomass in the endogenous growth phase similar to an EA facility, but usually at a longer SRT and much higher MLSS concentration than that of an EA facility and with no MLSS recycled. A programmable logic controller (PLC) monitors and automatically controls most of the MBR functions making the system fairly easy to operate.

Since the biochemical reactions and solids removal occur in the same reactor, this results in a substantially smaller footprint than the typical EA facility. A MBR is also more stable than an EA facility since it avoids the problems with clarifier upsets and maintaining the proper RAS rates. Further, the excess solids removed from a MBR are more fully digested than those of other AS processes because of the long SRT employed.

Also, because of the much higher MLSS concentration that can be carried in the MBR, it is more capable of absorbing shock loadings. Grit and coarse solids removal is required to precede the MBR. This can be accomplished in a septic tank or separate grit and fine screening removal facilities.

There are basically two types of membranes systems used. Both use hollow membrane units having pore sizes in the fractional micron range ($\leq 0.4\ \mu$) that exclude all particulate matter. In one type, the mixed liquor is pumped at relatively high pressure through the interior of the membrane and clear water permeates through the membrane into a collection system while excess mixed liquor is recirculated back to the bioreactor. In the other type of membrane system, the membranes are submerged in the bioreactor, in direct contact with the mixed liquor; clear water permeates to the inside of the membrane under a differential pressure produced by pumping of the permeate, with all solids remaining in the bioreactor.

An MBR can be designed for removal of organics, nitrogen and phosphorus. Removal of organics and nitrogen is accomplished by operating the MBR in both aerobic and anoxic phases. In this case, the bioreactor is divided into two sections by a baffle wall. The first section is operated as an anoxic reactor to promote denitrification. Mechanical mixing is usually provided, rather than mixing using aeration. In the second, aerobic section, aeration by diffusion of air under low pressure into the mixed liquor provides the oxygen and mixing required for removal of organic matter and nitrification of ammonia-nitrogen by facultative and aerobic microorganisms.

Mixed liquor in the second section is recycled (by pumping) back to the first section for denitrification. The dissolved oxygen in the mixed liquor is depleted rapidly in the first section, because of the high concentration of MLSS and the resulting high D.O. demand, resulting in anoxic conditions being established.

Phosphorus can be removed chemically by flocculation using metal salts as discussed in another part of this section. However, a smaller amount of a metal salt is required as compared to P removal in non-membrane processes, since it is not necessary to produce a large floc for gravity settling. The carbon required by the denitrifying microorganisms is normally provided by the incoming wastewater, although in some cases an external carbon source is required.

The membrane pores are prevented from being fouled by applying the diffused air in a manner that will maintain scouring velocities within the mixed liquor in the vicinity of the membranes. A reversed, pulsed flow is also used to clean the membranes in place.

Periodically, the membranes must receive a more through cleaning (using chemical cleaning). However, depending upon the type of membrane modules used, provisions are made for easy removal for cleaning and reinsertion of the cleaned membrane modules or for chemical cleaning in-place.

MBR systems used in enhanced pretreatment facilities for onsite wastewater renovation are available as pre-manufactured (packaged) systems and the manufacturer provides design of such systems. The key design parameters are: (1.) type of membrane used; (2.) hydraulic and organic loading rates; (3.) the ability to provide uniform distribution of dissolved oxygen and mixed liquor around and across the membrane modules submerged in the MBR; (4.) the ability to prevent fouling of the membrane pores; (5.) the life cycle of the membranes; and, (6.) the ability to easily maintain and replace the membranes when necessary.

With proper operation and maintenance, a MBR process is capable of producing an exceptionally clear effluent, suitable for reuse as reclaimed water for non-potable purposes. It will remove all suspended solids, virtually all organics, most of the nitrogen, and provide a several log reduction in the number of pathogens present in the wastewater.

As previously discussed, the temperature of the mixed liquor in continuous-flow, suspended growth bioreactors is an important factor in the overall process efficiency. Therefore, the bioreactors should be protected from low temperatures by installing the reactor tankage in or below the ground surface. Where installed below ground, provisions must be made for easy access to the reactor for maintenance purposes.

5. Fixed Film Bioreactors

Fixed film bioreactors include rotating biological contactors (RBC), recirculating granular media filters (RGMF), trickling filters (TF) and packed bed reactors (PBR). While some of the design parameters for fixed film bioreactors are similar to those for suspended growth bioreactors, it is important to note the difference in the manner in which the organic loading rates are defined. For suspended growth bioreactors, the loading rate is expressed as the ratio of the load (e.g. organic, nitrogenous) to the biomass suspended in the mixed liquor (MLVSS). In a fixed film bioreactor, the equivalent loading rate is the ratio of the unit mass loading rate to the specific surface area of the media (unit surface area per unit of media volume) which will differ for each particular type and shape of media used.

A RGMF is efficient in removal of organics and nitrification, and also can achieve denitrification (usually resulting in at least 40-50% removal of total nitrogen). (Denitrification has been found to occur in some fixed film aerobic bioreactors at anoxic microsites within granular media filters and in the inner portions of the fixed biofilms where dissolved oxygen has not penetrated.)

Where the denitrification obtained in a RGMF is insufficient to meet nitrogen removal requirements, recirculation of the RGMF effluent back to the septic tank will increase the removal of total nitrogen, with up to 70% or more removals attainable. Such removals will depend upon the ratio of soluble BOD_5 concentration to nitrogen concentration, the recirculation rate to the septic tank, and the hydraulic residence time of the recirculated effluent in the tank. Where a high degree of nitrogen removal is required, a packed bed anoxic reactor is often used for further denitrification of the RSF effluent.

An aerobic RBC is efficient in removal of organics and nitrification, and an anoxic RBC is efficient in denitrification. As in the case of a RGMF, recirculating of the nitrified effluent from the aerobic RBC back to the septic tank can provide a fair degree of denitrification, but packed bed reactors are normally used for denitrification where a high degree of nitrogen removal is required.

a. Rotating Biological Contactors (RBCs)

The rotating biological contactor (RBC) type of treatment plant has been used for wastewater treatment for more than 3 decades in the United States. A rotating biological contactor consists of corrugated plastic media discs vertically mounted on a horizontal shaft and slowly rotated while partially or fully submerged in a tank through which the wastewater flows.

Thus, the RBC is characterized as a continuous-flow, fixed film bioreactor. The rotating media is available with both standard and high-density surface area configurations. Rotation is provided either by mechanical drives or by air drive systems that used low pressure compressed air directed at cups fixed to the media discs. The RBC process is somewhat simpler to operate than a continuous flow suspended growth bioreactor because there is no need to recycle the biomass.

The media is usually divided into several stages, depending upon the nature and degree of treatment required. Biological growth develops naturally on the rotating surfaces and by utilizing the organic and nitrogenous pollutants present in the wastewater as a food source, removes or alters the composition of the pollutants. Rotation of the plastic media allows the biological growth to come in intimate contact with the wastewater and also results in the shearing of excess biological growth from the media. The mixing action of the rotating media keeps these excess growths in suspension until the treated wastewater flow carries them out of the RBC tank for separation and disposal.

Where the metabolic processes of aerobic microorganisms are employed for oxidation of organic pollutants (BOD_5) and ammonia present in the wastewater, the RBC is operated in an aerobic environment. In this case, the rotating media is approximately 40 % submerged in the wastewater and rotation of the media exposes the biological growth directly to the air for absorption of oxygen. Where denitrifying microorganisms are used for the removal of nitrogen, the rotating media is usually submerged so as to exclude the presence of oxygen.

Flow equalization should be incorporated into the design of a RBC facility. The flow equalization tank will not only smooth out the daily flows and loadings, but can also serve as an anoxic reactor for denitrification, if sufficient volume is provided in the tank for such purpose. Plant scale operations have shown that it is possible to obtain significant de-nitrification, up to 70% or more, by using the wastewater, which has a substantial concentration of dissolved organic pollutants, as a carbon source. This mode of operation requires recycling part of the nitrified effluent from the aerobic portion of the RBC plant back to a tank containing wastewater.

While recycling back to the septic tank is an option, this will affect the removal of settleable and floatable solids. Another option is to recycle back to a flow equalization tank equipped with mixing facilities and designed to also function as an anoxic reactor. Denitrification will proceed if the hydraulic retention time of the nitrified wastewater in the equalization tank is sufficiently long to allow the denitrification process to proceed.

Use of the flow equalization tank as an anoxic reactor will permit thorough mixing of the reactor contents and this will increase the efficiency of the denitrification process. Mixing can be accomplished hydraulically by providing multiple points of recycle into the equalization tank or by mechanical means. A method for periodically removing settled solids from the equalization tank should be provided.

As discussed earlier in this section, use of the soluble BOD_5 in the septic tank effluent as a carbon source in an anoxic reactor located upstream of an aerobic bioreactor, in this case the RBC, would reduce the organic loading on the RBC facilities. However, there is usually insufficient soluble carbon (soluble BOD_5) available for complete denitrification, and the septic tank or anoxic reactor tank effluent will contain residual nitrates and possibly nitrites as well. Therefore, where a greater nitrogen reduction is required, the feeding of an external carbon source will be required.

1. Aerobic RBC for Oxidation of Organics and Nitrification

The controlling parameters for design of an aerobic RBC for oxidation of organics and nitrification, exclusive of required effluent quality, are as follows:

- The soluble BOD_5 and ammonia-nitrogen mass loadings, expressed in lbs./day/1000 square feet of active surface area of the media, including average and peak loadings,
- The dissolved oxygen content in the RBC bioreactor,
- The number of media stages and the density (surface area) of the media,
- The rotational velocity of the media,
- The liquid detention time in the RBC tank, defined as a tank volume to media surface area ratio (gal./sq. ft.),

- The pH of the wastewater, and,
- The wastewater temperature during the colder months of the year.

The soluble BOD_5 ($SBOD_5$) loading limit for the first stage of an aerobic RBC recommended by RBC manufacturers' ranges up to 4.0 lbs./day/1000 sq. ft. However, studies by the U.S. EPA and others have shown that the $SBOD_5$ loadings recommended by the RBC manufacturers may sometimes result in less than adequate process performance due to the oxygen demand in the first stage(s) exceeding the oxygen transfer capability of the RBC. EPA recommended that the media stages be conservatively designed for an $SBOD_5$ loading in the range of 2.5 to 3.0 lbs./1000 sq. ft., particularly when there may be sulfur compounds present in the influent wastewater, as would be the case for septic tank effluent. The design loading used for sizing of the media surface area should be not more than 2.5 lb. $SBOD_5$ /1000 sq. ft. at the average day design flow and not more than 3.0 lb. $SBOD_5$ /1000 sq. ft. at the maximum day design flow. Standard density media should be utilized for removal of BOD because the heavy biological growth that occurs in the stages used for BOD removal would tend to clog the higher density media.

When wastewater receives pretreatment in a septic tank, some of the non-soluble BOD_5 is removed and some is hydrolyzed to $SBOD_5$ by the action of the facultative bacteria present in the tank. Thus, the $SBOD_5$ in the effluent of the septic tank will usually be higher than that in the raw wastewater. This should be taken into account when calculating the $SBOD_5$ loading applied to the aerobic RBC.

The loading rate recommended for nitrification by one of the leading manufacturers of RBC equipment is 0.30 lb. of ammonia-nitrogen (NH_3-N) /day/1000 sq. ft. of media. Studies by the U.S. EPA and others have confirmed this loading rate as being reasonable and the nitrification stages of the aerobic RBC media should be designed so as not to exceed this rate. Because a much lighter biological growth occurs during the nitrification phase, the more cost-effective higher density media should be used.

The total nitrogen (TN) concentration in the raw wastewater will include NH_4^+, organic nitrogen (ON), and in some instances, NO_3^-. When this wastewater receives pretreatment in the septic tank, some of the non-soluble organic nitrogen will be removed in the tank and virtually all of the remainder will be converted to NH_4^+ by the action of the bacteria present in the tank or by the bacteria in the RBC reactor. Thus, the NH_4^+ concentration of the wastewater in the RBC bioreactor will be higher than that in the raw wastewater and will be close to the TN concentration, the difference being the amount of refractory organic nitrogen.

The prerequisite for a high degree of nitrogen removal (de-nitrification) is essentially complete nitrification, which in turn requires a high degree of organics removal. Where such high removals are required, the RBC manufacturers recommend that the media be arranged in 3 to 4 stages, and prudent design would call for four stages. Provisions must also be made for the addition of an alkali to the wastewater prior to the nitrification media stage(s) to counter the destruction of alkalinity resulting from the nitrification process so as to avoid depressing of the pH below the low end of the desirable range for nitrification. (This pH range has been previously discussed under Subsection B12.b.)

Studies have shown that increases in the liquid volume-to-media surface area ratio beyond 0.12 gal./sq./ft./day did not increase removal efficiencies at a given hydraulic loading rate. Therefore, this ratio is adequate for sizing of the RBC tank.

As previously discussed, the pollutant removal efficiencies of wastewater treatment facilities that depend on biological processes are substantially affected by the temperature of the wastewater. The wastewater temperatures that occur during the colder periods of the year control the process design. The RBC manufacturers recommend that the media surface areas be increased when the operating temperature may be less than 55°F (~13°C) and provide curves for temperature correction factors. Since the average temperature of septic tank effluent in Connecticut can drop to as low as 50° F (10°C), the media surface areas of the organic oxidation and nitrification stages of the aerobic RBC should be increased based on the correction factors for that temperature.

One of the earlier major problems associated with RBC operation was the failure of the shafts on which the rotating media are supported. This was mainly due to underestimation of the effects of the heavy, often unequally distributed biological growth that occurs on the media. While the RBC manufacturers have increased the strength of these shafts to overcome such failure, it is still necessary to monitor the weight imposed on the shaft. Monitoring of the weight supported by the shaft is accomplished by utilization of a hydraulic or electronic load cell device, installed beneath the bearing on the idler end of the shaft, that provides an output to a load-indication display device.

While the wastewater is normally discharged into the upstream end of the RBC tank and then flows under gravity from one media stage to the next, provisions should be made for step feeding the wastewater to the individual baffled compartments containing each media stage. This will permit adjusting the loading on the media in each compartment for more efficient operation and to mitigate the occurrence of noxious odors in the first stage compartment due to overloading of the media.

As previously discussed, provision should also be made for recycling a portion of the effluent from the RBC to an upstream anoxic reactor and also back to the first stage to provide oxygenated liquid that will also mitigate the occurrence of noxious odors. Means should also be provided for removal of an excessive buildup of biomass on the media discs. This can be accomplished by use of a pressurized jet of water or compressed air directed on the media by a hand held wand.

RBC facilities should be protected from the weather, particularly cold weather temperatures. They can be installed within a building or within containment structures buried in the ground and provided with insulated covers that are easily removed for maintenance and repair. In either case, adequate ventilation should be provided.

2. Anoxic RBC for De-nitrification

The controlling parameters for design of an anoxic RBC for de-nitrification are much the same as for an aerobic RBC except that the mass loading rate of concern is that of the nitrate-nitrogen NO_3^- in the effluent of the aerobic RBC. The NO_3^- loading rate recommended by a leading manufacturer of RBC equipment is about 1.0 lb. /day/1000 sq. ft. of media. However, studies by the USEPA and others raise doubt as to whether the assumptions used to derive this loading rate are appropriate. Accordingly, a safety factor of at least 50% should be used for sizing the media surface area. Thus, the design loading rate should not exceed 0.67 lb/day/1000 sq. ft. at 55° F. As discussed above, a temperature correction factor should be used to increase the design media surface area to provide for a lower wastewater temperature. The anoxic RBC should be designed with several stages, and the final stage should be operated as an aerobic RBC to remove any excess external carbon source that may be contained in the effluent from the anoxic stages.

3. Flexibility of Operation of RBC Facilities

It is recommended that flexibility be provided in the design of RBC facilities to enable them to operate in three different modes. In mode No 1, effluent from the pretreatment (septic) tank would flow through the aerobic RBC first, with the nitrified effluent flowing to an anoxic RBC for denitrification. In this mode, a supplemental source of carbon would be required.

In mode No. 2, effluent from the septic tank would flow first to the anoxic RBC that would also receive recycled nitrified effluent from the aerobic RBC. The recycled effluent would be denitrified in the anoxic RBC using the septic tank effluent as source of carbon; thus some of the $SBOD_5$ will also be removed. The effluent from the anoxic RBC would then flow to the aerobic RBC for oxidation of the remaining $SBOD_5$ and the NH_3^-.

A portion of the aerobic RBC effluent would be recycled back to the anoxic reactor, as discussed above, and the remainder would be discharged to any downstream treatment facilities that may follow the RBC units. Available information indicates that the recycle rate should be at least 200% of the average daily wastewater flow rate and the ability to recycle at up to 300% should be provided. Actual experience will indicate whether this mode of operation will be able to meet the project denitrification requirements without requiring addition of a supplemental source of carbon. If successful, this would eliminate, or at least substantially reduce, the cost of providing a supplemental source of carbon.

In mode No. 3, nitrified effluent from the aerobic RBC would be recycled back to the septic tank or flow equalization tank for denitrification, as previously discussed. Plant scale tests have shown that, using recycle rates of up to 300%, this mode of operation has the potential for substantial reduction of nitrates (~ 70%) without the use of a separate RBC anoxic reactor or a supplemental source of carbon. Again, actual experience will indicate whether this mode of operation will be able to meet the project denitrification requirements. If successful, this could eliminate the operation and maintenance costs associated with the anoxic RBC where denitrification to ≤10 mg/L total nitrogen is not required.

4. Final Clarifier

A gravity type final clarifier is usually provided to remove most of the suspended solids in the RBC effluent. These solids are relatively low in concentration compared to the MLSS concentrations discharged from suspended growth bioreactors and thus sludge bulking is not a problem. Since the clarifier will follow the de-nitrification process, the problems associated with rising sludge in a clarification tank with a long sludge detention time (due to de-nitrification occurring at the bottom of the tank) will not be experienced and special mechanisms for rapid sludge removal will not be required.

Therefore, the controlling parameters for design of the clarifier should be the surface overflow rate, expressed as gal./day/sq. ft. of clarifier surface area, and the weir overflow rate, expressed as gal./day/ L.F. of weir. The clarifier overflow rates and weir overflow rates recommended by various authorities vary, but generally range from 400 to 800 gal/day/sq. ft. and 5,000 to 20,000 gal/day/L.F. respectively (based on the average daily design flow rate). The overflow rate selected will depend upon the maximum concentration of suspended solids permitted in the clarifier effluent. In order to minimize solids carry-over in the clarifier effluent, the clarifier should be designed so as not to exceed the lower end of the range of values given above. The sidewater depth of the clarifier should be at least 10 ft.

5. Recirculating Granular Media Filters (RGMF)

Treatment of septic tank effluent by intermittent filtration using a recirculating granular media filter can accomplish the removal of organic matter and nitrification required as well as partial de-nitrification and filtration. Intermittent filtration is the intermittent application of wastewater to the surface of a specially prepared bed of granular material, which is underdrained to collect and discharge the effluent from the bed.

There are many intermittent granular media filters used throughout the United States to treat wastewater from many types of residential, commercial and institutional establishments. Most of these utilize sand media and are known as "single pass" sand filters. The process is highly efficient and capable of producing a high quality effluent while requiring substantially less skill and time for operation and maintenance as compared to other treatment processes producing effluent of comparable quality.

This process is often used to "polish" septic tank effluent prior to disinfection and discharge to surface waters where such discharges are permitted. Purification of the wastewater is accomplished through the mechanisms of straining, absorption, and by the biochemical processes of microbes living within the filter bed.

Until relatively recent times, intermittent sand filters were designed either as the open bed or buried type and were operated in a once-through or "single-pass" mode, where the pretreated wastewater was applied to the filter and the filter effluent discharged directly to downstream facilities. When used to polish septic tank effluent, the filters were usually of the buried type, since the application of septic tank effluent to open sand filters resulted in the creation of substantial odors. However, the surface layers of sand tended eventually to become clogged and either required raking to break up the clogged layers or replacement of the top few inches of sand. This can become a costly maintenance problem when the filter is of the buried type.

An innovative concept of intermittent sand filter operation, developed in Illinois about 50 years ago (Hines and Favreau -1974) employed recirculation of the filter effluent back through the sand filter bed in several applications, or passes. This type of filter became known as a recirculating sand filter, or RSF. Today, RSF can be a misnomer, since various types of granular media have been substituted for the sand. Currently, a more descriptive name may be "recirculating granular media filter, or RGMF".

A typical RGMF consists of a septic tank, a recirculation tank equipped with pumps, one or more granular media filter beds, an electrical control system, and a means for diverting the return flow (filtrate) from the RGMF to either or both the recirculation tank or to other downstream facilities. Except for the recirculation pumps, and perhaps one, very simple automatically operated flow diversion valve in the recirculation tank, there are no other mechanical facilities involved other than some manually operated flow isolation valves.

The recirculation tank provides for storage of the septic tank discharge and recycled filter effluent between recirculation pump cycles. The recirculation pumps (typically of the submersible "effluent pump" type) are usually controlled by a time clock that can be set to provide the desired recirculation rate. Septic tank effluent flows into the recirculation tank and is then pumped to and distributed over the surface of the filter media. As the wastewater flows down through the media, most of the suspended solids are removed, and most of the pollutants that exert biochemical and nitrogenous oxygen demands (BOD_5, NOD) are oxidized to carbon dioxide, water and nitrates.

In addition, in most cases some of the nitrates formed during the treatment process are reduced to gaseous nitrogen, carbon dioxide and water by biological denitrification. The liquid is also oxygenated to a reasonably high degree as it passes through the filter media.

The filter consists of one or more compartments containing a bed of filter media installed over an underdrain system that collects the filtrate and returns all or a portion of it (depending on the flow diversion method selected) to the recirculation tank where it mixes with the septic tank effluent. Mixing of the recirculated oxygenated filtrate with the septic tank effluent results in a relatively fresh liquid being applied onto the surface of the filter, thus mitigating the odor problems normally associated with applying septic tank effluent onto open bed filters. Thus, there is no need to bury a RGMF below ground surface and it is designated as a“ free access” type of filter that permits ease of maintenance of the filter media.

Treatment of the wastewater applied to the filter results from several complimentary processes, including straining, sedimentation, adsorption onto the media particles, and, most importantly, the metabolic processes of the biomass that develops in the filter media. Most of this biological activity occurs at the surface and in the upper portion of the filter media. A complex population of organisms develops and dwells on and within the filter media. The biological population in a RGMF has been found to include numerous species of single celled bacteria and such higher life forms as protozoa and rotifers. Macro-organisms such as nematodes (round worms) and annelids (earthworms) have also been found to exist in the filter. The most important of these biological organisms, from a treatment standpoint, are the bacteria. Some of these microorganisms attach themselves to the filter particles, which thus act as tiny fixed film reactors, while others may be found in liquid micro-sites in the smaller spaces between the filter particles. While some of the organisms can and do exist in an anaerobic or anoxic state, the most important ones are those that flourish under aerobic conditions.

The processes that take place in a RGMF are essentially the same as those that occur, in a single-pass intermittent sand filter, with one major difference. Both types of filters are highly capable of reducing BOD_5, and suspended solids concentrations by 90-95% or more and of converting 90-95% or more of the ammonia-nitrogen present in the wastewater to nitrates. However, the RGMF can also provide a substantial reduction of nitrates through biological denitrification. This makes a RGMF attractive where it is necessary to limit the concentration of nitrates in the treated wastewater.

The effluent from a properly designed, operated, and maintained RGMF is of high quality, odorless and quite clear in appearance. Typically, 90-95% or more of the five-day Biochemical Oxygen Demand (BOD5) and Total Suspended Solids (TSS) and a significant percentage of the pathogenic bacteria in the applied wastewater are removed by the cleansing mechanisms of the filter. This high quality can be obtained using either sand or other type of granular media if the filter is designed for the type of media selected. A septic tank- RGMF treatment system is also very efficient in converting the ammonia-nitrogen and organic nitrogen in the applied wastewater to nitrates, with typical conversion of nitrogenous compounds to nitrates of 90-95% or better during all but the coldest periods of the year.

Generally, some denitrification also occurs. Data obtained from experimental laboratory studies and operation of full scale recirculating granular media filters have shown total nitrogen reductions through the RGMF of 40-70 % or more without special provisions for denitrification. How denitrification occurs in a RGMF has yet to be fully understood, but it appears to be an autogenous effect that may occur either in the recirculation tank or the filter media, or both. The conditions required for denitrification can occur in the recirculation tank under anoxic conditions that may sometimes prevail when the BOD of the septic tank effluent depletes the dissolved oxygen in the nitrate-laden filtrate. Similar conditions can occur at anoxic micro-sites in the filter media where facultative bacteria, soluble carbon and nitrates are also present.

The major problem encountered with an RGMF (as well as with a single pass intermittent sand filter) is clogging of the surface or upper portions of the filter media. Since some of the particulate matter carried in the liquid applied to the filter surface is inert and thus non-biodegradable, it is inevitable that such matter will accumulate in the filter. In addition, the non-degradable byproducts resulting from growth, death and decay of the biological organisms will also accumulate in the filter and this accumulation also tends to clog some of the pore spaces in the media. Polysaccharides and other slimes produced by the bacteria also cause clogging of the filter (Miller, 1992, Mitchell, 1964).

If the unit organic loading applied to the filter is too high an over-abundance of biological organisms will develop. This will increase the production of bacterial slimes and other byproducts of bacterial action that cause surface clogging (Tyler, et al-1977) and cause a reduction of the hydraulic conductivity of the filter media. This will result in a slow draining filter in which the time available for air to enter the filter media is reduced.

If the clogging becomes severe, liquid will pond on the entire surface of the filter and severely restrict the passage of air into the filter media. Under such conditions the filter will become anaerobic and its performance will become significantly degraded.

Experience has indicated that clogging of the filter media may occur in the late winter-early spring of each year. The reason for this phenomenon is not fully understood. One hypothesis is that during the colder weather, the metabolism of the biomass in the filter is slowed down while the organic loading remains essentially the same as during warmer methods. Not being able to oxidize all of the organics because of their slower metabolism, the excess organics are stored in excessive extracellular bacterial slimes that increase as the cold weather progresses, resulting in the seasonal clogging observed. Regardless of the cause, the operator of a RGMF should anticipate that this might happen and make provisions for alleviating the clogging conditions.

Experience has shown that if clogging occurs to the extent that it cannot be remedied by periodic raking of the filter surface, occasional removal of the top inch or so of the media may restore the RGMF to normal operation. If that does not alleviate the clogging problem, discontinuing flow to the filter for a period of a month or more, depending upon ambient temperature, will result in rejuvenation of the media.

Therefore, as a preventative maintenance procedure, multiple filter compartments should be provided, with provisions of isolating each compartment from the wastewater and filtrate flow so as to permit it to rest for a period of one to two months, depending upon seasonal temperatures. After the resting period, the rejuvenated compartment can be placed back into service and another compartment taken out of service and allowed to rest.

To maintain the quality of the filter effluent, the filter compartment being placed back into service should receive reduced surface loading rates for a period of time, depending upon seasonal temperatures, until it ripens (re-establishes the biotic population). During that time, the filter next in sequence to be rested should remain in service. After some length of time, usually measured in years, the media will have to be replaced due to clogging by the accumulation of non-biodegradable matter.

Since one compartment will be out of service, prudent design would consider reducing the hydraulic and organic loading rates used for overall design of the RGMF so that the loading rates applied to the remaining compartments do not exceed the maximum rates recommended herein.

It is important to design an RGMF to operate under aerobic conditions with clogging potential minimized. This can be accomplished by assuring:

- the filter is properly sized to accommodate the hydraulic and organic loadings,
- uniform distribution of the wastewater- filtrate mixture over the entire filter surface,
- provisions are made for adequate circulation of air through the media,
- the potential for the fouling of the filter surface by weeds, leaves and other airborne debris is minimized, and,
- adequate access to the filter surface is provided for ease of maintenance.

The factors that must be considered in design of a RGMF include:

- Hydraulic Loading Rate
- Organic Loading Rate
- Recirculation Rate
- Recirculation Method
- Recirculation Tank Volume
- Dosing Intervals
- Media Gradation
- Media Depth
- Underdrain System
- Method of Flow Distribution to Media
- Harsh Weather Conditions
- Fouling of Media Surface by Extraneous Matter

Recirculating granular media filters used for treating residential wastewater are normally sized on the basis of the hydraulic loading rate of septic tank effluent applied to the surface of the filter. This loading rate typically ranges from 3-5 gallons per day per square foot (gpd/sf), and conservative designers will select a rate near the lower end of this range.

(Note that the hydraulic loading rate is based on the wastewater flow rate, rather than on the recirculated flow rate.) For example, normal residential septic tank effluent BOD_5 concentration is about 150 mg/L. If a surface loading rate of 4 gpd/sf were selected, the maximum organic loading rate given below would not be exceeded.

However, the organic loading must also be considered where the wastewater characteristics differ from that of residential wastewater. The organic loading is expressed as the amount of BOD_5 applied to the filter.

Values given in the literature range up to 0.005 pounds of BOD_5 per day per square foot of filter surface area. Prudent design would suggest selecting a loading rate below the upper end of the range. Thus, it is necessary to compare the hydraulic and organic loading rates and select the controlling rate for design of the filter.

The ratios of the recirculated flow rate to the wastewater flow rate cited in the literature range from 3:1 to 5:1. For example, for a recirculation rate of 4:1(4 parts of filtered effluent mixed with one part of septic tank effluent) the recirculation flow rate will be five times the design daily flow rate.

The recirculation method used can have a subtle effect on the RGMF effluent quality. There are two methods used for controlling recirculation of the filter effluent, including the use of a flow-splitting device or a simple and automatically operating floating ball diversion valve.

In the case of a flow-splitting device (usually a chamber containing a moveable gate, adjustable weirs or orifices, or similar mechanisms), a portion of effluent is directed back to the recirculation tank while the remainder is discharged to downstream processes, if any, and thence to the SWAS. Where a floating ball diversion valve is used, all of the filtrate flows back to the recirculation tank that contains the diversion valve. The floating ball is contained in a cage that allows it to float up and down with the liquid level. When the liquid level in the recirculation tank is low, the flow diversion valve remains open and all of the filtrate is returned to recirculation tank through the open valve. When the liquid in the recirculation tank reaches a predetermined level, the floating ball rises and seals against the rim of the downward facing recirculated filtrate inlet fitting. This prevents further return of the filtrate and diverts it to the downstream facilities.

The advantage of the flow splitting method of recycle flow control is that it conserves hydraulic head. The invert of the flow splitter chamber is controlled by the elevation of the RGMF effluent underdrain piping, rather than by the elevation of the recirculation tank that must be below the elevation of the septic tank outlet piping. Since the septic tank effluent and recycled filtrate is pumped to the RGMF from the recirculation tank, the filter and the flow splitter chamber can be elevated above the recirculation tank and thus the downstream facilities can be located above the elevation of the septic tank outlet piping. The disadvantage of the flow splitter method is that, regardless of the rate of flow into the recirculation tank from the septic tank, during each recirculation cycle, some of the filtrate bypasses the recirculation tank and continues on to downstream facilities.

The advantage of using an automatic flow diversion valve is that, at times of low wastewater flows, all of the filtrate continues to be recirculated. During such periods of low flow, the wastewater receives additional “polishing” treatment, resulting in an enhanced filtrate quality. However, in using this method, some hydraulic head is lost because of the lower elevation of the diversion valve in the recirculation tank.

It is often recommended in the literature that the recirculation tank volume for a RGMF serving a residence should be at least equal to the design daily flow volume. In this case, the recirculation tank also functions as an equalization tank. Another method of determining the working volume has been used successfully in designing several RGMF installations in Connecticut that serve commercial facilities having a much greater design flow. This volume is calculated as the sum of the filter dose volume required for the design maximum time interval between successive dosing cycles plus a volume equal to the maximum amount of wastewater that could be expected to be discharged into the recirculation tank during that same maximum time interval. The latter volume is usually based on the peak hourly flow rate of the wastewater.

In addition to the working storage volume, the recirculation tank should be sized to retain sufficient liquid at all times so as to submerge the pump volutes in order to reduce the chances of the pumps becoming air-bound. Also, some freeboard between the liquid level at maximum storage capacity and the level of activation of a high level alarm float switch, and between the high alarm level and the invert of wastewater piping, should also be provided. A 6-inch freeboard allowance in each case is usually sufficient.

Provisions should also be made to account for possible malfunction of the recirculation pumps or electrical equipment. This can include provision of a high level overflow to an emergency holding tank of sufficient capacity to provide time for a response to the malfunction condition. In some cases, the Department may permit the high level overflow to discharge to the SWAS provided it will receive assurance of a very short response time. To provide for the malfunctioning pumping equipment, the recirculation tank should be equipped with dual, slide rail mounted submersible pumps operating on alternate cycles, and suitable access hatches. A discussion on wastewater pumps and appurtenances is given in Section XII.

The intervals between dosing of the commingled septic tank effluent and recycled filtrate from the recirculation tank onto the surface of the filter media commonly range from 30 minutes to 2 hours. Sufficient time should be provided between dose cycles for the filter media to drain and become completely re-aerated before the next cycle is initiated.

A simple timing device (normally a time clock) that actuates the automatic pump alternator that is part of the recycle pump control system controls the dosing intervals. Thus, the time clock must be capable of initiating at least 48 pumping cycles per day.

The filter dose volume is the total recirculated flow volume per day divided by the number of dosing cycles per day. The dose volume should be equivalent to that needed to cover the entire surface of the filter media, in order to use the media in the most efficient fashion and maintain the environmental conditions (food supply, nutrients, and moisture) required by the population of organisms in the filter.

The interval ultimately to be used is normally selected based on the results obtained from fine-tuning the actual operation, and may require changing with the seasons. An electronic type of time clock will provide significant flexibility in establishing dosing times and cycles and should be used. The dosing cycle should provide sufficient time for the filter media to drain and become completely re-aerated before the next cycle is initiated.

The gradation of the granular media is a designer's choice, subject to certain restrictions. The gradation is usually expressed as the percentage (by weight) distribution of media grain sizes. The controlling grain sizes include those passing the #5, #10,#18, #60, and #100 U.S. Standard Sieves. The percentage by weight of the media grains that pass a # 10 and # 60 sieve are designated as the d_{10} and d_{60} size respectively. The d_{10} size is designated as the effective size and the ratio of d_{60}/d_{10} is designated as the uniformity coefficient. The larger the d_{10} size, the coarser the media, while the media sizes become more uniform as the uniformity coefficient becomes smaller.

The range of d_{10} sizes for sand media is 1.0 to 2.0 mm, and 2.0 to 4.0 mm for pea-gravel media, with a uniformity coefficient ≤ 3.0 and not more than 2% should pass the #100 sieve. Thus, the media should be carefully and thoroughly washed, repeatedly if necessary, to remove virtually all of the fine material that may be included in the raw material from which the media is obtained. The media also must be sound, durable, and free from soft, thin, flat or elongated particles, with a hardness value > 3 on MOH's scale of hardness.

The media depth used in a RGMF ranges from 24 to 30 inches. While most of the treatment in a RGMF appears to occur in the top 6-12 inches of the media, additional depth will serve two purposes. Additional depth will provide a polishing effect on the filter effluent, enhance the removal of pathogens, and will permit occasional removal of the top layer of the media for a period of years without compromising the treatment process. Thus providing additional depth will under normal operating conditions prolong the time before complete replacement of the media is required.

The functions of the underdrain system are to support the filter media and prevent migration of the finer particles of the media out of the filter, to provide a means of collecting the filtrate and to provide a means of venting the filter. Ventilation of the filter is important so that fresh air can diffuse and be drawn into the filter media after each dosing cycle to provide the oxygen required by the biomass in the filter.

A properly designed underdrain will consist of several graded layers, normally not more than four, of progressively smaller sized stones from bottom to top of the underdrain system, with the total depth usually around twelve inches and a filtrate collection system.

Perforated pipes are installed at the bottom of the stone underdrain to collect the filtrate. These filtrate collector pipes are connected to a header that discharges to a main drain that returns the filtrate to the recirculation tank. The end of each perforated filtrate collector pipe opposite from the header connection is connected to a vertical vent riser that extends above the surface of the filter media and is open to the atmosphere. These vents aid in circulation of fresh air through the filter media after the filtrate has drained from the filter, and it is important that the perforated collector pipes and vents be spaced in such a manner as to insure that adequate venting of the filter media occurs.

The gradation of the underdrain stone will depend upon the type of filter media used. The following gradations have been found satisfactory. Stone size refers to U.S. Standard Sieve size designations.

Underdrain Stone for Sand Media:

Layer	Depth, in.	Stone Size, in.
Bottom	3	$1^1/_4$
Second	3	3/4
Third	3	1/2
Top	3, min.	1/4

Underdrain Stone for Pea Gravel Media:

Layer	Depth, in.	Stone Size, in.
Bottom	4	$1^1/_4$
Second	4	3/4
Top	4, min.	1/2

The underdrain stone must be sound, durable, and free from soft, thin, flat or elongated particles with not more than 1% passing the #200 mesh sieve. As in the case of the filter media, the stone should be carefully and thoroughly washed, repeatedly if necessary, to remove virtually all of the fine material that may be included in the raw material from which the media is obtained.

Underdrain piping should consist of perforated PVC pipe, ≥ 4 inches in diameter, installed approximately 6 ft. on centers. The discharge end of each underdrain should connect to a header pipe that will convey the filtrate back to the recirculation tank or flow splitting chamber. The other end of each underdrain pipe should be connected to a vertical riser that extents above the top of the media. These risers will permit the underdrain piping to also provide ventilation of the filter media. It is advisable to provide valves at the end of each underdrain pipe to permit isolation of each filter compartment.

Flow distribution of the mixture of wastewater and recycled filtrate onto the filter surface has been accomplished by numerous methods. The simplest method used in the past for open intermittent sand filters consisted of gravity flow pipes with outlets (tee branches or perforations) discharging to splash blocks supported on the surface of the filter media. This method is not adequate for the coarser media used in an RGMF, because it leads to unequal distribution of the liquid onto the filter surface. Such unequal distribution results in over-utilization of part and under-utilization of the remainder of the filter surface and often leads to progressive clogging of the filter surface. A more suitable method is pressure flow distribution via a network (manifold and laterals) of piping, having closely spaced small orifices (perforations), which receives the wastewater/recycle mixture under pressure and distributes it fairly evenly over the filter surface. In some cases, shields are placed over the orifices to further aid in distributing the liquid over the filter surface. In cold climates, such piping is often covered with coarse aggregate to protect against freezing of the pipe orifices.

Another suitable method utilized for flow distribution is a system of pressure flow distribution manifold and laterals, equipped with riser pipes and spray heads spaced above the filter media in such a pattern as to provide a uniform distribution of the wastewater/recycle mixture over the entire filter surface. In this method, the flow distribution manifold and laterals are buried between the bottom of the filter media and the top layer of the media support gravel. This is an excellent method of flow distribution, and also provides oxygenation of the applied wastewater. This method must be used when pea gravel is used as the filter media, in order to insure that short-circuiting does not occur due to the high hydraulic conductivity of this media. An isolation valve should be installed on the inlet to the flow distribution manifolds to permit isolation of each compartment.

Care must be taken to insure that the spray head orifice is large enough to prevent clogging by solids carried in the wastewater and/or by microbial growths and that the spray head is installed in such manner as to prevent freezing during cold weather operations. To prevent freezing problems from occurring, spray heads should be installed in an upright position on the riser pipes and the entire flow distribution system should be designed to be self-draining, upon completion of a dosing cycle, to a distance below the filter surface where non-freezing temperatures prevail.

A very simple but effective spray head consist of a 3/4 inch diameter PVC threaded pipe nipple with a slot cut into the side of the pipe nipple at a slight upward incline from a horizontal plane. The slot has a depth of approximately 1/2 the internal diameter of the pipe nipple. The pipe nipple is capped just above the slot opening with a threaded PVC pipe cap, which can be easily removed should spray head cleaning be required. When operated at a pressure of 6 psi, the spray heads provide a good spray pattern over an arc of almost 180° and, when spaced at 6-ft. center-to-center, complete coverage of the filter surface is obtained. Wider spacing of the spray heads can be used with higher operating pressures. The spray head method does produce some aerosol that should be contained within the filter area. This matter is addressed below.

There are a number of variations on the configuration of the filter itself. The filter may be constructed at grade within earthen dikes or within a depression made by excavating below existing grade. In such cases, a plastic watertight membrane liner is used to contain the filter media and underdrain facilities and is supported by the soil bottom and sidewalls of the excavation or the dikes.

In other cases, the filter may consist of a structure consisting of a concrete floor slab and walls constructed of concrete, concrete blocks or timber. (Where timber structures are used, they may also be lined with a plastic watertight membrane liner.)

Most filter structures have a rectangular footprint. The filter surface may be left open to the atmosphere, may be covered with a layer of gravel, may have removable covers or may be located within an enclosure. While successful operation of uncovered filters under cold weather conditions has been reported (e.g.: Louden-1984), many filters located in areas which experience cold winter weather conditions are provided with removable covers or with fixed covers providing headroom for maintenance. All of the filters constructed to date in Connecticut are provided with covers.

Covers serve to exclude precipitation, contain aerosols, retard heat loss (including protection against cold winds), and keep the filter free from wind borne debris such as leaves, paper and plastic wrappers, etc., which when deposited on the filter surface tend to cause uneven distribution of the wastewater. In addition, the covers should be opaque to exclude sunlight so as to prevent the growth of weeds and algae, problems often associated with uncovered filters. Where walls are used to enclose the filter media, they should be surrounded by earth fill extending to an elevation not less than the surface elevation of the filter media in order to provide insulation under cold weather conditions.

If covers are provided, however, they must be designed to provide easy access to the filter surface. They should be relatively light and easily removed or hinged at one end so they may be propped open to permit maintenance of the filter. If fixed in place they must provide ample headroom so that the operator does not have to continually stoop while working within the filter.

Several of the filters constructed in the State have been retrofitted with easily installed prefabricated enclosures consisting of corrosion resistant metal frames covered with a heavy duty, weather resistant plastic coated fabric. These have been in use for several years and to date have withstood the ravages of severe storms, cold weather and exposure to the environment within the filter. The fabric covers are fastened to the metal frames and filter substructures in a manner that permits their easy removal for repair or replacement. These enclosures permit operating personnel to stand in a comfortable manner while maintaining the filters.

Where the inherent denitrification that occurs in a RGMF is insufficient to meet effluent total nitrogen limits a portion of the filtrate can be recycled back to the septic tank, or preferably to an anoxic reactor, for further denitrification. Experience obtained from a technology demonstration project funded by the Department indicated that a total reduction of up to 70% or more of the nitrogenous compounds might be obtained in this manner. Where a greater reduction is required an anoxic reactor fed with an external carbon source will be required.

The portion of the filtrate to be recycled might be calculated from a mass balance of the amount of nitrates in the filtrate and the soluble BOD_5 available in the septic tank for denitrification, providing the ratio of soluble BOD_5/ NO_3 is known. However the available soluble BOD_5 may vary from time to time and most probably by season of the year, as colder temperatures will affect the biological activity of the microorganisms responsible for converting the particulate BOD_5 to soluble BOD_5 in the septic tank.

In addition, the amount needed to reduce the nitrates to nitrogen gas will depend upon wastewater characteristics and ambient operating temperatures. Also, additional soluble BOD_5 will be needed to remove the dissolved oxygen in the recycled filtrate the amount of which is also somewhat dependent upon ambient temperatures. Therefore, a method of varying the recycled portion of the filtrate should be provided to "fine-tune" the recycle rate base on actual experience.

It should be noted that recirculating some of the filtrate back to the septic tank for denitrification would result in a significant reduction of the organic loading on the RGMF, since BOD_5 will be consumed in the denitrification reaction. This can result in reducing the filter surface area if organic loading would otherwise have controlled the surface area required.

Operation and maintenance of a RGMF is not an involved task. Generally, O&M requirements include inspection of the filter surface and spray heads on a weekly basis and cleaning them as needed, checking the recirculation tank pumps and equipment on a monthly basis, checking the solids and scum levels in the septic tank (and grease trap if required) on an annual basis, periodic cleaning of the filter media, overseeing the sampling and testing and filing the discharge monitoring reports required by the State discharge permit.

The advantages of an RGMF include:

- A highly reliable, stable process that produces a high quality effluent.
- Less skill and time required for O & M (ease of operation).
- Tolerance of peak hydraulic and organic loadings.
- A minimum of mechanical and electrical operating equipment
- Protection of downstream facilities from high suspended solids loadings, as clogging first occurs on the filter surface and provides ample warning for remedial action.

The disadvantages of a RGMF include:

- More land area required than most other types of treatment facilities.
- May have higher capital costs.
- Not suitable for high degree of nutrient removal.
- Possible periodic odors if not properly maintained.

It should be noted that there are several variants of the RGMF process that have been developed in various parts of the U.S. Most of these have to do with modifications for more efficient nitrogen removal. Engineers involved in designing an OWRS are encouraged to search the literature to become familiar with the various ways that a RGMF can be configured to meet specific goals so as to be able to fully evaluate the RGMF process with respect to other enhanced pretreatment processes.

c. Trickling Filters

The trickling filter (TF) was one of the earliest fixed film bioreactors used to provide biological treatment of wastewater. A TF consists of a bed of media, contained in a reactor of either circular or rectangular cross-section, where wastewater is applied in a uniform method onto the top of the bed and trickles down through the media on and in which a biomass has been established.

Depending upon the types and distribution within the media of microorganisms that make up the biomass, a trickling filter can be designed to oxidize both organics and nitrogenous compounds. It should be noted that the name "trickling filter" is a misnomer, as a TF does not generally perform a filtration function. One notable exception is when sand or fine gravel is used as the medium, such as in a RGMF that in fact operates in much the same way as a TF but also serves as a filter.

Various types of media are used in modern trickling filters. The media originally used included rock (broken stone) and other granular media. Media used in modern trickling filters include:

- Fabrications of synthetic plastic sheets into modules of tubular configurations, or wooden slats arranged as stacked pallets, having high unit surface areas and porosity;
- Individual pieces of plastic material of various shapes and sizes;
- Open cell foam blocks,
- Crushed glass;
- Crushed brick; and
- Lightweight expanded shale or clay aggregates.

Where heavy media are used, the beds are usually of shallow depth. Where lighter weight media are used, the beds can be much higher. Most of the packaged trickling filter (TF) reactors available for small scale enhanced pretreatment facilities now employ some type of synthetic media.

Soon after wastewater is initially applied to the bed, the surfaces (and inner void spaces) of the media become coated with a zoogleal biomass, slimy in appearance. Similar to other packed bed type of reactors, the biomass is made up of bacteria and higher life forms that utilize the carbon in the wastewater as a source of food and energy. The oxygen required by the biomass for their metabolic processes is obtained from natural or forced circulation of air through the voids in the media.

The biomass removes organic matter by adsorption of particulate organic matter and assimilation of soluble organic carbon. As mentioned above, a TF can also be operated so as to nitrify the nitrogenous compounds in the wastewater. Eventually the biomass growth reaches such thickness that it begins to slough off the media and flow down with the liquid to the bottom of the reactor. The continuous sloughing of the biomass from the media is a required part of the process, since excessive accumulation (thickness) hinders the treatment process by limiting or preventing oxygen from reaching the inner portion of the biomass.

The biologically treated wastewater is collected at the bottom of the reactor and either all of it proceeds on to a downstream clarification process or a portion is recirculated in one or more passes over the bed while the remainder is discharged downstream. The contact time of the wastewater with the biomass on the media is fairly short, and thus a substantial amount of active biomass must be present in order for the process to be efficient in oxidation of organics and nitrogenous compounds. Where a high degree of treatment is required, recirculation must be employed.

In the once through, or single pass mode, the wastewater is periodically applied (dosed) onto the surface of the TF and the treated effluent collected at the bottom of the TF is discharged to downstream facilities. In the recycle mode, a portion of the TF effluent is recycled to a flow equalization tank and is returned to the TF for one or more times (passes), while the remainder (the "forward flow", equivalent to the influent wastewater quantity) is discharged to downstream facilities. The efficiency of treatment approves with the recirculation ratio (usually ranging from 1:1 to 4:1) and the dosing rate employed; in this respect the effect of dosing rates and recirculation is similar to that obtained in an RGMF. The recirculating mode permits a decrease in the surface area of the TF but requires more or higher capacity pumping equipment for application of the wastewater to the top surface of the TF.

Recirculation rates and frequency of dosing will affect the ability of a TF to fully nitrify the effluent. Oxidation of organics typically occurs in the upper portion of the TF media while oxidation of nitrogenous compounds (nitrification) occurs near the bottom. Increased recirculation rates can result in a smaller depth of the TF media required for oxidation of organics, leaving an increased depth for the nitrification process that results in enhanced nitrification. However, where denitrification is required, excessive recirculation rates and dosing frequencies can result in an excessive D.O. in the TF effluent that will affect the efficiency of the denitrification process.

The components of a TF include the reactor structure, media, media support system, underflow collection system, pumping equipment, the device(s) used to distribute the wastewater onto the TF surface, and, the equipment used to deliver forced air into the TF if that method of aeration is employed.

The clarifiers that receive the effluent from the trickling filters are designed on the same basis as those used in the RBC process. Since the biomass is attached to the media, recycling of settled biomass, as in suspended growth bioreactors, is not required.

Factors to be considered for design of trickling filters include:

a. Wastewater characteristics
b. Type of filter media (material, porosity [percent of void space], specific surface area [unit surface area/unit volume])
c. Filter depth
d. Method of applying wastewater to surface of TF
e. Hydraulic loading rate (total volume of liquid, including recirculation, per unit of time per unit of filter cross-sectional area
f. Organic loading rate (unit mass BOD_5/unit volume of TF or, alternately, unit mass BOD_5/unit of media surface area)
g. Nitrogenous loading rate (unit mass NH_4-N/ unit volume of TF or, alternately, unit mass NH_4-N /unit of media surface area)
h. Recycle rate
i. Dosing method
j. Method of aerating the TF (natural or forced air movement)
k. Temperature

The treatment efficiency of trickling filters can vary substantially, but a properly designed and operated TF reactor facility is reputed to provide up to 85% -90% or more removal (oxidation) of organics and nitrogenous compounds. The use of crushed brick as a potential means of removing phosphorous, as discussed by Anderson, et. al. (1998) is worthy of consideration. However after a period of time the phosphorus sorption capacity of the brick would be reached and it would have to be replaced. The effect of shielding of the brick, by the biomass, from sufficient contact with the wastewater may also be of concern. Lightweight expanded aggregate also has a similar potential for P removal.

Problems with trickling filters include possible generation of odors and growth of nuisance organisms (e.g. filter flies, snails) and excessive growth of biomass that does not slough off of the media. Odors can be of particular concern when septic tank effluent is applied to the surface of the TF bed. Many of the packaged types of TF are covered and suitable for burial below ground, which may tend to mitigate the filter fly nuisance problem, and they can also be vented to odor removal facilities. If snails are encountered, provisions must be made for their removal before the effluent reaches any mechanical equipment. Uneven and irregular sloughing of media can impact the efficiency of the process, and continual organic overloads can lead to clogging of the media due to excessive biomass growth. The application of wastewater containing a concentration of fats, oils and grease (FOG) higher than normal residential wastewater may also cause clogging problems, thus interfering with the biomass metabolism and reducing the efficiency of the process.

d. Packed Bed Anoxic Reactors for Denitrification

Studies have shown that pretreated, nitrified wastewater can be effectively denitrified using a packed bed (fixed film) reactor (PBR) operating in a low oxygen (anoxic) environment.

A packed bed anoxic reactor consists of a reactor vessel filled with inert packing material through which the nitrified wastewater is passed under saturated flow conditions. The packing material, which may consist of stone or various types of artificial media manufactured from plastics or ceramics, provides the surfaces on which a film of the denitrifying bacteria can grow in the absence of dissolved oxygen. The packing is completely submerged in the wastewater, thus avoiding exposure of the bacterial films to atmospheric oxygen.

Factors to be considered in the design of packed bed reactors for denitrification include:

- Wastewater characteristics,
- Type of bioreactor (shape, direction of wastewater flow; e.g. upflow or downflow),
- Type of media (shape, size, specific surface area (surface area per unit volume of media), porosity),
- Total packing depth,
- Temperature,
- Denitrifier growth rate,
- Denitrification rate,
- Specific Surface Loading Rate (unit mass of NO_3-N applied /unit surface area of media,
- Hydraulic Loading Rate (gal/sf of gross reactor area perpendicular to direction of flow, and,
- Empty Bed Detention Time (detention time of the wastewater, based on the gross volume of the reactor.)

Denitrification rates for various types of packing materials and operating temperatures have been experimentally determined by a number of researchers. Sutton et al. (1975) conducted a significant study on low temperature biological denitrification of wastewater using pilot scale packed bed reactors containing various packing materials. The results of his studies and those obtained by others have been summarized and published by the U.S. EPA (1975, 1993.) Studies have shown that 90-95% or more of nitrate-nitrogen can be removed in a packed bed reactor operating at hydraulic detention times as short as several hours or less. However, operating a backed bed denitrification reactor at short detention times (synonymous with a high mass loading rate of nitrate) results in a buildup of bacterial cells until eventual plugging of the reactor occurs (Requa and Schroeder-1973). Therefore, a much longer hydraulic detention time may be desirable, particularly where the reactor is not cleaned on a frequent basis. Studies have shown that a high nitrate removal efficiency can be obtained when the reactor is operated at a hydraulic detention time of several days with much less accumulation of biomass (Lamb, et al - 1987).

Nitrate-laden wastewater is often introduced into packed bed denitrification reactors at the bottom of the packed bed reactor and flows in an upward direction through the packing material under saturated flow conditions that enable anoxic conditions to be maintained. The upward direction of flow will also aid the nitrogen gas resulting from the denitrification process to rise in concurrent flow with the liquid until it escapes to the atmosphere above the surface of the liquid. Since temperature has a significant effect on the rate of denitrification, it is desirable to bury the reactor in the ground. A means of gaining access to the reactor for removal of excess biomass is required.

e. Fluidized Packed Bed Bioreactors

A fluidized packed bed bioreactor is a unique type of PBR operated in a submerged upflow mode for denitrification of nitrified wastewater at relatively high rates because of the high concentration of denitrifier biomass (as much as 25, 000 mg/L or more) that can be contained in the reactor. This permits the use of smaller reactor volumes than other types of packed bed reactors. In a fluidized PBR, the packing material is expanded by the upward flow of fluid through the bed, thus enhancing the removal of nitrogen gas and excess biomass. Periodically, the PBR is "bumped" by an air backwash to assist in the stripping of gaseous nitrogen from the media.

Because fluidized packed bed reactors used for small scale enhanced pretreatment facilities are of the proprietary, packaged type with varying media characteristics and methods of operation, design parameters will vary. Sizing criteria utilized by the manufacturer is generally based on consideration of reaction kinetics, empirical methods, pilot test data and performance data from full-scale facilities. The loading rate is usually expressed as mass NO_3-N/unit of volume, often given as lb. NO_3-N/1000 cu. ft. of media.

The beds should be capable of being backwashed and provisions must be available to skim and remove the solids washed to the top of the bed. Carbon required for denitrification can be from external sources or by feeding of settled wastewater. However, if wastewater is used as a carbon source, provisions must be made to nitrify the unoxidized NH_4-N and then to reduce the NO_3-N that will otherwise bleed through with the effluent.

E. Chemical Feed System for pH and Alkalinity Control

As previously discussed in this section, the nitrification process has a strong effect on the pH of the wastewater by increasing the hydrogen ion concentration and thus decreasing the pH. Low alkalinity source (potable) water will exacerbate the pH problem, as there will be less alkalinity available to buffer the increased hydrogen ion concentration. While the alkalinity of the wastewater will be increased due to the waste discharges, there may still be insufficient alkalinity present in the wastewater for complete nitrification to occur. Therefore, provisions should be incorporated in the enhanced pretreatment facilities for storage and feeding of an alkaline chemical as necessary to permit control of the pH of the wastewater.

Either sodium bicarbonate or magnesium hydroxide are most suitable as the alkali source for small facilities with limited operational control, as these chemicals are non-toxic, non-corrosive, and, if overdosed, will not raise the pH above the range required in the nitrification process. While consideration should be given to an increase in the sodium content of the ground water from the use of sodium bicarbonate, it is unlikely that a major increase in sodium content of the ground water will result from the small quantities of sodium bicarbonate used in enhanced pretreatment for on-site wastewater renovation facilities.

F. Enhanced Pretreatment for Food Processing and Serving Establishments.

As previously discussed in Sections IV and IX, the high organic content of wastewater discharged from food processing and serving establishments have often caused early failure of the subsurface wastewater absorption systems (SWAS) serving such establishments. Therefore, the long-term acceptance rate (LTAR) used for design of a SWAS for such establishments must be significantly down-rated, resulting in large areas required for the SWAS.

Major constituents of such wastewaters are fats, oils and grease (FOG), and high concentrations of FOG reaching the SWAS have resulted in complete clogging of the infiltrative surfaces. To avoid such failures, FOG must be removed to the greatest extent practicable. Underground grease traps, if properly sized and maintained will intercept and remove a significant portion of FOG. However, FOG removal in underground grease traps to concentrations that will not significantly affect a SWAS (20-30 mg/L or less) is problematic. Therefore, enhanced pretreatment should be considered for removal of FOG for such establishments.

One method (Nibbler™) developed for such purposes, reputed to be successful for reduction of FOG concentrations by greater than 90%, utilizes a grease trap, flow equalization tank equipped with pumping equipment, a hybrid type of attached and suspended growth aerobic process carried out in upflow type bioreactors, and a clarifier. The effluent from this process reputedly has been found to be no stronger than residential wastewater. The aerobic bioreactors consist of tanks in which buoyant media, held in place just below the liquid surface by plastic retainers, provide a large surface area to support the biomass required for aerobic digestion of the FOG and other organic matter. An air blower and air tube arrangement creates aerobic mixed liquor that circulates through the media providing the oxygen required for the aerobic biomass with sufficient turbulence to promote sloughing of the biomass from the media. The sloughed biomass collects at the bottom of the reactor and is periodically removed by a septic tank pumper truck. The clarifier provides protection of the SWAS from solids escaping from the bioreactor. Another method used successfully at a restaurant in Connecticut consists of an underground grease trap discharging to a septic tank followed by a secondary grease trap that discharges to a recirculating granular media filter. This treatment system has been in operation for 20 years and produces a high quality effluent, with ≥ 90% removal of BOD_5 and TSS and total nitrogen removal ≥ 40%. No ponding has occurred in the SWAS underlain by medium sand that receives the RGMF effluent.

G. Enhanced Pretreatment for Removal of Toxic Chemicals

Where toxic organic or inorganic chemicals are anticipated to be present in the domestic wastewater, the design engineer should provide specific information on the types and concentrations of such chemicals and the methods to be used for their removal from the wastewater.

Because of the wide range of such toxic chemicals, it is not feasible to present in this document a review of methods available for their removal from the wastewater. In general, such methods may include physical, chemical and biological treatment processes. Chemical removal processes may include chemical addition, mixing, precipitation, flocculation, sedimentation and filtration. Physical processes such as adsorption using activated carbon or other special media may be warranted in some instances. It is also possible that the microorganisms used in biological processes for removal of non-toxic organics and nitrogen may become acclimated to such chemicals, if they are present in trace amounts, and remove them along with the non-toxic organics. However, before relying on biological processes, laboratory and/or pilot plant tests should be conducted to determine if such treatment is possible without subjecting the biological processes to stresses that will prohibit them from providing the efficient removal of non-toxic organics and nitrogen expected from such processes.

If the toxic organic chemicals are contained in a small sidestream contribution to the overall wastewater flow, pretreatment of that sidestream by adsorption on granular activated carbon may be the most effective method for their removal. Small activated carbon reactors are commercially available and once the removal capacity of the carbon is exhausted, the reactors may be exchanged with the vendor for new or recharged reactors. A filtering process to remove suspended solids that could adversely affect the carbon adsorption process should precede activated carbon adsorption reactors.

It is virtually impossible to predict the adsorption capacity of the carbon media due to the wide range of toxic chemicals that may be present in the wastewater, unless a pilot testing program is conducted. Absent pilot testing, a granular carbon should be selected that is suitable for adsorption of a broad range of toxic organics.

H. Enhanced Pretreatment for Pathogen Removal/Inactivation

1. General

Disinfection is normally not provided where wastewater is discharged to SWAS, since both the biological mat that forms at the soil/leaching system interface and the natural soil beneath and downgradient of a properly designed SWAS are very effective in removing pathogenic bacteria and viruses. However, experience indicates that the usual biological mat does not develop where a highly treated wastewater is discharged to a SWAS. Therefore, it is prudent to provide an additional safety factor to ensure that the groundwater at the boundaries of the zone of influence of the SWAS will meet water quality goals.

2. Chlorination-Dechlorination

Disinfection using chlorine is problematic because there are a number of organic compounds in wastewater that can react with chlorine to form toxic compounds. Further, chlorination may not be the most effective means of disinfection where parasitic protozoa (e.g.: *Cryptosporidium parvum*, *Giardia lamblia*) and viruses are the pathogens of concern.

The Department does not typically approve the use of chlorination where wastewater is discharged to a SWAS.

3. Ozonation

Ozone (O_3) is a very strong disinfectant and functions by direct oxidation of the cellular walls of bacteria, by damage to the nucleic acids of bacteria and viruses, and by causing other deleterious effects on living organisms. Destruction or inactivation of pathogens by ozone occurs rapidly, usually within 30 minutes or less. The major factors to be considered in design of an ozone disinfection process are dosage rates, mixing, and contact time.

Ozone must be generated on-site for immediate use, as it is unstable and decomposes rapidly to oxygen in water and air. Ozone generators at wastewater treatment facilities generally operate by imposing a high voltage alternating current across a dielectric discharge gap in the presence of very dry air or oxygen. The effectiveness of the process depends on the susceptibility of the pathogens, the concentration of ozone and the contact time of the pathogens with ozone.

Ozonation of pathogens is accomplished by feeding ozone via diffusion into the wastewater in a small chamber that provides sufficient contact time. As in all chemical disinfection processes, thorough mixing of the diffused ozone with the liquid to be disinfected is important to assure all pathogens are contacted with ozone for the required contact time. Any residual ozone gas that escapes from the liquid in the chamber must be destroyed before it is released into the atmosphere, because it is extremely irritating to respiratory organs and may be toxic. For the same reason, leakage of ozone from the ozone generator must be monitored and immediate steps taken to correct any leaks. Ozone in gaseous form is also explosive, but at concentrations well above the normal concentrations used for ozone disinfection.

Ozone generators are available as pre-manufactured units in many sizes and pre-manufactured ozone destruction equipment is also available.

4. Ultra-Violet Irradiation

Ultra-violet irradiation is an accepted means of disinfecting pretreated wastewater. It is a safe technology with respect to disinfected water quality, as it does not produce any residual toxicity and produces negligible chemical by-products. The absorption of UV energy results in photochemical damage to the nucleic acids (DNA and RNA) of the pathogenic microorganisms, thus preventing the pathogens from reproducing and causing an infection in a host.

UV irradiation has been found to be very effective for inactivating bacteria, the pathogenic protozoans Cryptosporidium and Giardia, and viruses. There have been significant studies showing that UV irradiation for disinfection is at least as good as chemical disinfection, and some studies have found UV disinfection to be superior, particularly with respect to viruses and pathogenic protozoans such as Cryptosporidium and Giardia (Dykstra, et. al-2002).

The UV dose is a product of the UV light intensity (I), measured in milliwatts/cm^2 (mW-sec/cm^2) or the equivalent milliJoule/cm^2 (mJ/cm^2), and the exposure time, T, in seconds. Thus, the UV dose is expressed as I T. This method of expressing dosage is analogous to that used for chlorine disinfection, which is expressed in Concentration x Time, or CT.

A UV system basically consists of a closely spaced array or battery of low-pressure mercury arc lamps individually encased in quartz tubes and submerged in a compartment through which the wastewater flows. The UV compartments can consist of a sealed reactor or an open channel. The only maintenance required is periodic cleaning of the quartz tubes and periodic replacement of the lamps themselves, which have a reported useful life of at least 7500 hours.

The ability to "overdose" with UV light and still not adversely affect the water quality allows for a less rigorous control of the disinfection process, with the only result of overdosing being the expenditure of additional power. In most cases, a UV system of the size required to disinfect the effluent from a proposed on-site wastewater treatment facility will have an energy requirement on the order of hundreds of watts, rather than many kilowatts. Therefore, the cost consequences of overdosing are not severe. The system can therefore be designed for peak flow requirements and operated at constant power levels so as to eliminate the need for flow pacing controls, although such control can be provided for UV systems designed for large flows.

The controlling parameters for design of the UV system include:

- the wavelength of the light emitted from the lamps, measured in nanometers (nm.),
- the ultra-violet light intensity, expressed as milliwatts/sq. centimeter,
- the residence time (the period of time that the wastewater is exposed to UV radiation), expressed in seconds,
- the concentration of suspended and colloidal solids in the water, and
- the flow conditions in the UV compartment.

The wavelength for optimal germicidal effect ranges from 250-270 nm. and approximately 85% of the output of a low-pressure mercury arc lamp is at 253.7 nm. Thus, the UV dosage rate is expressed as milliwatt-seconds/sq. cm. at 253.7 nm. The dose rate recommended by the Department is not less than 60 milliwatt-seconds/sq. cm**.** This dosage should be available at 65% UV transmission and 65% of new lamp output. The flow characteristics through the UV compartment should be as close to plug flow as can reasonably be obtained. Unlike chemical disinfection, UV disinfection is independent of temperature and pH of the water. Factors that impact on UV disinfection efficiency are the chemical species in the water, and suspended and colloidal solids. Iron and calcium can form fouling deposits on the quartz sleeves that encase the individual UV bulbs. Suspended and colloidal solids reduce the transmittance of the UV light in the wastewater and serve to protect pathogens encased within the solids. Thus, for effective UV disinfection, the wastewater must have a high transmittance and low solids concentration, and provisions must be made for periodic cleaning of the surfaces of the quartz sleeves.

The UV equipment should be provided with a means of measuring and indicating the UV dose and for indicating "Lamp out" conditions for each mercury lamp included in each UV module included in the disinfection system. The "lamp out" detection system should be capable of providing an alarm signal to a central alarm station in the pretreatment facility.

I. Enhanced Pretreatment for Phosphorus Removal

1. General

Chemical pretreatment for removal of phosphorous (P) is normally not provided for in onsite wastewater renovation systems, since P sorption in the soils beneath and downgradient of the SWAS is usually quite effective. However, as previously discussed in Subsection G.4 of Section X, situations may arise where the ability of the soils beneath and downgradient of the SWAS to remove phosphorous from the percolating wastewater is, or may become, insufficient to meet the Department's water quality goals. In such cases, provision for enhanced pretreatment for P removal may either be initially required, or the design of the OWRS must be such that provisions for P removal can be easily incorporated into the pretreatment facilities in the future.

As discussed in Section X, phosphorus is usually found in raw wastewater as organic phosphorus, polyphosphate, or orthophosphate. For efficient removal of phosphorus, all forms of phosphorus must be biologically converted to orthophosphate. Such conversion will occur in most biological wastewater treatment processes under normal operating conditions and no special effort is required for such conversion.

Phosphorous removal may be accomplished using either biological or chemical processes, or by adsorption on beds of reactive media capable of adsorbing P for a considerable length of time. A small amount of P is also removed in cellular synthesis by the biomass in bioreactors used for oxidation of organics and nitrogenous compounds and for denitrification.

2. Biological Processes

Removal of phosphorus by incorporation into new cellular matter resulting from biosynthesis reactions is usually < 2 mg/L, depending upon the processes involved. Biological processes can also attain phosphorous removal beyond that obtained from cellular synthesis. However the operation of such processes requires skilled operation and more constant attention than is normally available for the small scale enhanced pretreatment facilities used for an OWRS. Therefore, it is not anticipated that dedicated biological phosphorous removal processes will be used for enhanced P removal.

3. Sorption on Reactive Media

As stated in subsection D.5 c, crushed brick and lightweight expanded shale or clay aggregate have been investigated for use as reactive media for removal of P because of their metal oxides and clay content and the results appear to be promising. This method is reputedly able to remove ≥ 95% of the phosphorous in the wastewater contacting the reactive media.

Recently, a proprietary method of using reactive media for P removal has been developed and is now available. The media is a waste product (slag) from steel manufacturing which has a chemical composition high in metal oxides, especially calcium (Leverenz, H and G. Tchobanoglous - 2002).

Reactive media will have a finite life with respect to P removal and will have to be replaced once its P sorption capacity has been exhausted. Estimates of useful life range from 10 to 20 years; however, such media have not been in use long enough to be certain of the actual useful life. This approach to P removal appears to have significant advantages, including:

- The avoidance of chemical usage (resulting in no sludge production),
- Essentially maintenance free, and,
- The passive methodology involved, (which only requires that the wastewater be free of excessive organic compounds and suspended solids that could result in clogging of the media, and sufficient time is available for the adsorption to be completed).

4. Chemical Removal

Chemical removal of phosphorus may be accomplished using various chemical coagulants, the most prominent of which are iron and aluminum salts and lime. The metal salts include aluminum sulfate (alum), sodium aluminate, ferric chloride, ferrous chloride and ferrous sulfate. These coagulants change the soluble phosphorus present in the wastewater to insoluble precipitates, which are then removed by settling in clarifiers and/or by filtration.

Metal salts are usually chosen over lime, particularly for small-scale wastewater treatment facilities. This is due to the greater amount of sludge generated by the use of lime, the high costs associated with equipment and maintenance costs for lime storage, feeding and handling equipment, and because metal salt addition for phosphorus removal is a reliable, well documented technique used throughout the country. Liquid alum is often used for chemical P removal. Since liquid alum is 48% alum in a water solution, a premium is paid on the transportation costs because the water in the alum solution increases the shipping weight of the product. However, this is offset by the relative ease in handling and feeding of liquid alum and its cost relative to other aluminum compounds.

Liquid alum is a clear, light green to light yellow aqueous solution, weighing approximately 11.2 lb./gal, containing about 8.2% soluble aluminum expressed as Al_2O_3 or 4.37% expressed as Al and is available in 55-gallon drums and larger containers, and in bulk delivery to on-site storage tanks. Liquid alum is a corrosive (pH ~ 3.5) solution and its use requires careful design of storage and feeding equipment. Care must be taken in on-site storage and feeding to assure that the temperature of an alum solution is maintained above the point at which it begins to crystallize. Suppliers recommend a storage temperature of 45°F or higher. The material safety data sheet (MSDS) for liquid alum should be obtained from the supplier and the instructions for it's storage, handling and feeding scrupulously followed to prevent injury to personnel and deterioration of enhanced pretreatment plant facilities.

When liquid alum is used for P removal, stoichiometric calculations indicate that 9.6 lb. of alum will react with 1 lb. of phosphorous, or 9.6 mg/l alum will react with 1 mg/l of phosphorus. In practice, however, the quantities of alum required are higher than the stoichiometry would predict, due to competing reactions of alum with other dissolved solids present in the wastewater. Therefore an Al:P mole ratio of 2.2:1 is often used for design of chemical phosphorus removal using alum.

Using the 2.2:1 ratio, the estimated feed rate of alum would be 2.2 x 9.6 = 21.1 mg/L alum per mg/L phosphorus. Assuming it is desired to remove 1 mg/L PO_4-P concentration in a wastewater flow of 1000 gpd, the amount of liquid alum required would be calculated as follows:

0.001 MGD x 8.34 lb./gal x 1 mg/L PO_4-P to be removed x 21.1 mg alum/mg/L PO_4-P = 0.18 lb./day (equivalent to 0.016 gpd) of liquid alum. These values can be used to ratio up to the lb. or gal. of liquid alum required for any PO_4-P concentration and wastewater flow.

The solubility of the aluminum phosphate precipitate depends on the pH of the water. It is reported that the theoretical optimum pH for removal of phosphorus by chemical precipitation is 6.3 and that the optimum may range from 5.5 - 6.5, although removals will occur above a pH of 6.5. Addition of alum will lower the pH of the water because of neutralization of the alkalinity and release of carbon dioxide. The alkalinity neutralized by this reaction is theoretically 0.5 mg/l (as $CaCO_3$) per mg/l alum added; however, actual consumption may differ from 0.5 mg/l because of competition for the aluminum ions from other side reactions. Thus, following the precipitation and removal of P from the treated wastewater, the addition of an alkali may be needed to bring the pH of the wastewater to the effluent water quality value required by the discharge permit.

Feeding of the alum solution should be accomplished where good mixing with the wastewater will occur. This can be at a weir, flow measurement flume or other similar places where flow agitation occurs. High energy mixing should be avoided where the alum is added to the mixed liquor in an aerobic bioreactor or between the bioreactor and the clarifier to avoid shearing of the MLSS floc. With adequate mixing and subsequent flocculation, clarification and filtration, P removal to a residual of < 1 mg/L can be achieved.

Addition of metal salts for P removal will generate a significant amount of chemical sludge. This has to be taken into account when considering the removal, processing and disposal of the sludge. For addition of aluminum salts to remove P down to around 1 mg/L, the increase in sludge quantities from a suspended growth aerobic process will be about 35%. Removal of P to below 1 mg/L will cause a significant increase in the chemical sludge due to the formation of aluminum hydroxides.

The dose of any chemical used for P removal should be determined from jar tests conducted at various dose rates so as to avoid under-dosing or overdosing of the chemical. The results sought from a jar test are the optimum dose that will result in a chemical floc that will settle well and leave a low P residual in treated wastewater.

Under dosing can result in the formation of a “pin-point” floc that is difficult to remove from the wastewater except where the chemical is added to a membrane bioreactor.

Overdosing can result in the formation of an excess sludge volume and unreacted chemical remaining in the treated wastewater.

J. Solids Processing and Disposal

All wastewater treatment processes utilized for pretreatment of wastewater prior to its discharge to a SWAS produce solids that must ultimately be disposed of off-site in a manner approved by the Department. In the absence of enhanced pretreatment, these solids are removed in grease traps and septic tanks and then disposed of by septage waste removal firms. Where enhanced pretreatment is provided, secondary solids (sludge) are produced, the volume of which will depend upon the process(es) used. These solids are usually in very dilute form containing only a few percent solids concentration at most, and thus will require some means of storage in a slurry form until they can be removed and disposed of in a manner similar to that used for septage.

These solids can be stored in septic tanks, sludge holding tanks, and aerobic sludge digesters. For small facilities, it may be cost-effective to route these slurries back to the septic tank(s), provided the tank(s) have been appropriately sized to contain the additional solids that will collect as a result of settling and floatation. For larger facilities, the choice will be either to route the sludge to aerobic digesters or holding tanks.

Aerobic digestion is used to further stabilize (oxidize) biologically degradable organic compounds remaining in the sludge generated in biological treatment processes. An aerobic sludge digester operates in much the same manner as the extended aeration process, with the main differences being that the sludge is retained in the digester until it is removed for ultimate disposal, the SRT is usually much longer, and recycling only pertains to the periodic decanting of the digester supernatant and its return to the enhanced pretreatment process.

Where suspended growth bioreactors operated at long solids retention times (SRT) are used for enhanced pretreatment (e.g. EA, SBR and MBR processes), the volatile solids content of the sludge has usually been significantly reduced. Therefore, an aerobic digester may not significantly reduce the volume of sludge to be stored, and a holding tank might be the preferred option.

A holding tank can be either an anaerobic or aerobic type. However, since either type will have to be periodically decanted, with the supernatant returned to the wastewater treatment process (es), the supernatant from an aerated holding tank will have less of an adverse impact on these processes. Also, since many onsite enhanced pretreatment facilities will be located in reasonably close proximity to inhabited buildings, the use of dedicated anaerobic holding tanks may be problematic because of the disagreeable odors that usually result from decanting of the supernatant and storing and removing solids from the tank.

The main difference between aerated sludge holding tanks and aerobic digesters is the SRT. Sludge holding tanks are aerated to prevent odors that would likely occur from anaerobic holding tanks. On the other hand, aerobic sludge digesters are usually provided when additional reduction of waste solids is desired and when the digested sludge is proposed for use as a soil supplement. Aerobic digesters require a much longer SRT than is usually provided in an aerated holding tank, and this requirement imposes greater operational control and additional aeration requirements.

For example, assume that an aerobic digester is to be designed to meet U.S.EPA Class B sludge classification (U.S. EPA - 1993b). To meet the pathogen reductions of Part 503 regulations, the SRT required is 40 days at 20° C and 60 days at 15°C. Should the temperature be expected to drop below 15°C, the SRT would have to be increased. To meet the Vector Attraction Reduction requirements of Part 503 regulations, a 38% reduction in volatile solids is required. However, at the present time, the Department has not developed permitting regulations for land application of digested sludge. Therefore, it is doubtful that aerobic digestion would fulfill a useful function for disposal of sludge generated at the scale of enhanced pretreatment facilities used for an OWRS.

An aerated holding tank is usually operated in batch fashion. Batch operation involves three separate steps: sludge feeding, supernatant removal, and solids removal. An aerated holding tank can be used to thicken the waste sludge to reduce the volume of liquid sludge to be removed for ultimate disposal. The thickening process involves turning off the aeration/mixing equipment for a period of time and allowing the sludge to settle. At the end of the settling period, the liquid supernatant is decanted and returned to the enhanced pretreatment facilities. Methods for decanting the supernatant include floating decanters, telescoping valves, weir gates and multiple gated draw-off outlets arranged at several depths below the liquid surface and connected to a main manifold. Solids are removed on a periodic basis, when the volume of thickened sludge is such that relatively clear supernatant can no longer be obtained during a decant cycle.

Waste activated sludge (WAS) removed from clarifiers receiving MLSS from suspended growth bioreactors will usually consist of 1±% solids by weight. This sludge can be thickened to 2% or more in an aerated holding tank. Sludge removed from fixed film bioreactor processes may have a slightly higher percent of solids and a higher percent of volatile solids but will usually not thicken much more than WAS. While this may not appear to be a significant thickening process, thickening waste sludge from 1% solids to 2% solids results in reducing the volume of sludge to half its original volume. This will result in significant cost savings for ultimate disposal. The thickened sludge is removed from the lowest point in the holding tank via sludge withdrawal piping or directly by a septage waste pumping truck, with the latter method being adequate for small scale facilities.

The volume of the tank should be based on the average daily volume of waste sludge produced, including biological solids and chemical sludge if chemical phosphorous removal is practiced and the estimated average solids concentration in the tank. The average solids concentration should be conservatively estimated, usually not more than 2% solids by weight. Tank sidewater depth should be not less than 10 ft. to permit adequate mixing by diffused air. Additional tank wall height should be provided to accomodate any foam that may be generated and to partially shield the liquid surface from the wind if the tank surface is exposed. If at all possible, a buried tank is preferable, with adequate means to gain access to the air diffusion equipment for maintenance purposes. Provisions should also be made for foam suppression by a water spray system using the clarified effluent from the enhanced pretreatment facilities.

Aeration of the holding tank contents should be provided for both mixing and addition of sufficient oxygen to maintain the upper portion of the tank contents in an aerobic condition to prevent development of noxious odors. Submerged coarse bubble diffusers are normally used, since the varying liquid level in the tank will preclude the use of surface aeration. The diffused air requirement ranges from 20 to 30 cu. ft./min/1000 cu. ft. of tank liquid capacity. This should usually provide sufficient mixing capacity and sufficient oxygen to maintain a D.O. concentration of 1-2 mg/L in the tank liquor. However, some additional aeration capacity, up to 40 cu.ft./min./1000 cu. ft of tank capacity, should be provided as a safety factor and for re-suspending thickened solids that collect at the bottom of the tank during the supernatant decant process. The air blowers (normally rotary positive displacement type) should be designed to permit adjusting the air supply for the conditions actually encountered, to avoid excessive power costs.

K. Standby (Emergency) Power Supply

An emergency power generation system must be provided for all electrically powered enhanced wastewater pretreatment equipment where the wastewater generating facilities are served by a public or community water supply system capable of providing water to those facilities during a power outage. An emergency power supply generation system must also be provided for all enhanced pretreatment facilities incorporating biological processes where a prolonged power outage would result in the death of an aerobic biomass. The emergency generator may either be part of the enhanced pretreatment facilities or may be of the portable type, depending upon the electrical load requirements. Where portable generators are used, they should be available for immediate use upon loss of the normal electrical power supply.

Provisions for an emergency power supply should conform to the requirements of the State and local electrical codes and the requirements of the electrical utility providing the normal power supply.

L. New and Emergent Technologies

Technology development in wastewater treatment has burgeoned in the past few years, as new treatment processes and equipment suitable for enhanced pretreatment for onsite wastewater renovation systems are developed, tested and brought to market. In particular, new processes are being developed for nutrient removal, and for creating or enhancing aerobic conditions in and beneath a SWAS.

Hybrid bioreactors have been developed utilizing a combination of suspended growth and fixed film processes that permit reduction in the footprint of the reactor without sacrificing treatment efficiency. Hybrid systems that include oxidation of organics and ammonia-N and reduction of nitrates have also been developed where the reactor for each process is optimized to perform a particular function. Some early hybrid processes developed for individual residences have been improved and may be suitable for large-scale OWRS applications. Specialized, passive methods are now available for denitrification and for removal of phosphorous, using reactive media (e.g. Nitrex™, Phosphex™) without the need for adding chemicals, which only require the wastewater to flow through the media for a sufficient contact period. Facilities for using the new technology are often available as pre-manufactured units sized particularly for the onsite market. While the reactive media reputedly will last for a number of years, eventually its reactive capacity becomes exhausted and it must be replaced.

Continued research is also being conducted on the use of man-made wetlands for enhanced pretreatment of wastewater under year-round climatic conditions similar to those encountered in Connecticut. Drip irrigation systems have been developed for distribution of pretreated wastewater to the upper soil horizons that contain the most suitable soils for wastewater renovation. This method of wastewater distribution to the subsurface may be suitable provided that it can be demonstrated that freezing of the piping and drip emitters will not occur under the various cold season environmental conditions typical to Connecticut.

The engineer responsible for design of enhanced pretreatment facilities should research the recent literature in professional engineering and science journals to keep abreast of new technologies. A recent publication that provides an overview of a large number of new and emergent technologies for enhanced onsite pretreatment of wastewater is that prepared by Leverenz and Tchobanoglous (2002).

M. Beneficial Use of Reclaimed Water

The beneficial use of reclaimed water (wastewater that has received a high degree of treatment and disinfection) is not a new practice. Direct reuse of non-potable water, where there is a direct link from the treatment system to the reuse application, had its beginning in California in 1912 when the City of Bakersfield began using reclaimed water to irrigate crops and pastures. In 1918, California promulgated regulations for reclaimed water reuse, Arizona soon followed and permitted reclaimed water to be used for irrigation projects in the 1920s, and in the 1960s, Colorado and Florida began using reclaimed water in urban settings (Asano, T. – 1998).

Since those early beginnings, there has been a significant increase in the number and type of reuse projects and a considerable body of knowledge has accumulated concerning the safe use of reclaimed water.

A common misconception is that the use of reclaimed water for irrigation is only applicable in water-poor, semi-arid, or arid environments. However, as more communities are faced with increased water supply costs, watershed protection plans, water conservation plans, and public concern about water usage, reclaimed water can provide a recycling solution that is "environmentally friendly" and acceptable to the public. Such use reduces the demand upon existing water supply systems and ground water sources.

The Department may currently permit the use of reclaimed water for beneficial reuse (e.g., irrigation of vegetation, flushing of toilets and urinals). The water quality requirements for reclaimed water use are given in the Design Standards. Any use of reclaimed water will depend upon the nature of the use, and will be considered on a case-by-case basis. Enhanced pretreatment facilities utilized for producing reclaimed water should meet the requirements of the U.S. EPA for Class I Reliability Standards (U.S. EPA -1974).

Where the Department permits use of reclaimed water for any purpose, provisions must be made for discharge of the reclaimed water to the subsurface in an approved manner during times when its use for irrigation or other approved purposes is not needed.

Requirements governing the use of reclaimed water for golf course irrigation are given in the Design Standards.

N. Provisions for Monitoring and Control of Treatment Processes.

The ability to monitor and control enhanced pretreatment processes, and the equipment associated therewith, is vital to successful use of such processes. The means that should be provided for monitoring and control functions will vary with the complexity of the process. However, a common thread runs through the monitoring and control functions of all processes. The operator(s) of enhanced pretreatment processes should have instrumentation and equipment available to be able to determine:

- That the process variables are within operating limits required for process stability and efficiency,
- The operating status of all electrically and mechanically operated equipment,
- That critical liquid levels are within the normal range.

The operator(s) should also be able to easily vary operating conditions as required to maintain process stability and efficiency.

With respect to operating status of equipment and critical liquid levels, the intelligence required should be available to the operator(s) at the site of the treatment facilities and off-site at a monitoring location capable of forwarding such intelligence to plant operating personnel at all hours of the day and night.

Where packaged types of pretreatment facilities are used, most manufacturers will either provide or recommend some type of monitoring and control equipment. However, in some cases, the equipment provided or recommended is rudimentary in nature and consideration should be given to supplementary equipment.

A listing of the monitoring and control equipment that might be used is given below. The listing is not exhaustive, and new types of monitoring and control equipment are constantly being brought to market. The engineer responsible for overall design of a OWRS should review the need for the type of equipment required to monitor and control the particular process(es) employed and then review the literature and contact responsible vendors for detailed information and guidance.

a. Monitoring Equipment

- Flow measurement, indication and recording
- Liquid level detection, display and reporting
- Equipment operating status indicating lights
- Fault detection, display and reporting for all mechanically and electrically operated equipment. This includes, but is not limited to:
  - Loss of normal electrical power supply,
  - Emergency electrical power supply failure,
  - Fire alarms,
  - Equipment overloads,
  - High and low liquid levels,
  - High and low equipment and process operating temperatures, and
  - Failure of equipment to start or stop upon receipt of start/stop initiating signals.
- Alarm panels, with indicating lights actuated by relays or dry contact switches incorporated in electrical motor controls, capable of signaling local and remote alarm detection and notification facilities
- Indoor and outdoor alarm lights and horns
- Dialing alarm monitors or radio transmitters
- Modems
- Running time meters, for all electrically operated equipment
- Event recorders or cycle counters for all electrically operated equipment

b. Control Equipment

- Main electrical circuit breakers, secondary circuit breakers, and manual and automatic motor starters meeting requirements of National, State and local electric codes
- Local and remote Manual-On-Off switches and Manual-Off-Automatic switches for all electrically operated equipment
- Time clocks
- Repeat cycle timers
- Liquid level detection equipment (capable of providing a level indication signal output to operating equipment that control, or are controlled by, liquid levels)
- Programmable Logic Controller (PLC)
- Laptop and desktop computers
- Electrical surge protection for all electrically operated equipment
- pH meters
- Dissolved Oxygen meters
- Turbidimeters
- Pressure switches
- Temperature switches

O. Specifying Processes and Equipment

The engineer responsible for design of enhanced pretreatment facilities comprised of packaged treatment units should prepare specifications for procurement of such facilities. While there are many ways of writing such specifications, they should contain at least the following information and requirements:

- Description of the treatment process(es) and all plant materials, components and auxiliary devices,
- Operating Conditions (as listed in D.2 of this section),
- Submittal requirements for review and approval of all materials and equipment,
- Quality of materials and equipment,
- Special requirements for coatings and other corrosion protection provisions,
- Requirements for installation and start-up of the plant facilities, including participation of the manufacturer's authorized representative in the installation and start-up,
- Requirements for testing for approval of installation and operation of all equipment, including participation of the manufacturer's authorized representative and a written report by the manufacturer's authorized representative on the results of such tests and any recommendations resulting therefrom,
- Requirements for providing operation and maintenance instructions to plant operator(s) by the manufacturer's authorized representative,
- Requirements for operation and maintenance manuals,
- Equipment guarantees (typically: equipment to be free from defects in design, materials and workmanship for a period of at least 12 months from date of start-up),
- Process performance guarantee (Packaged treatment system guaranteed to produce an effluent quality based on information provided under Operating Conditions), and
- Requirements for spare parts, special tools and supplies.

These specifications, along with the drawings and design data, should be submitted to the Department for review. Any review by the Department, including any review comments or the lack of such comments, will not relieve the engineer, the manufacturer, the vendor, and the facilities owner(s) and their contractor(s) from their respective responsibilities for the proper design, manufacture, installation, operation and maintenance of the packaged treatment facilities.

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SECTION XII WASTEWATER CONVEYANCE SYSTEMS

TABLE OF CONTENTS

Table of Figures

SECTION XII WASTEWATER CONVEYANCE SYSTEMS

A . Introduction

This section provides an overview of the various means of conveying wastewater from its point(s) of origin to and through the various pretreatment facilities and from such facilities to the subsurface wastewater absorption system (SWAS), and to discuss some of the important parameters that affect the design, construction, operation and maintenance of such systems.

Gravity flow sanitary sewers predominate in the systems that serve to collect wastewater and deliver it to centralized wastewater treatment facilities from which the treated effluent is discharged to surface water bodies or the land surface. Gravity flow systems are also used to convey wastewater to and within onsite wastewater renovation systems that ultimately discharge pretreated wastewater to a SWAS. However, alternate types of conveyance systems are often needed to meet specific site restraints and overall onsite wastewater renovation system (OWRS) design requirements.

Alternate types of wastewater conveyance systems include:

- Small Diameter Effluent Sewers (SDES)
    - Small diameter minimum grade (MGES)
    - Small diameter variable grade (VGES)
- Vacuum Sewers, including vacuum pump stations and appurtenances
- Pumped Systems
    - Low Pressure (LP) Sewers
    - Force Mains
    - Pumping Stations and Chambers

B. Conventional Gravity Flow Sewers

Conventional gravity sewers are the most common wastewater conveyance systems. Since they normally convey raw wastewater, they are installed on a slope sufficient to obtain a minimum velocity ($\geq$ 2 ft/sec.) to prevent excessive deposition of solids. When a continuous downward slope sufficient to maintain the minimum velocity cannot be obtained, and tunneling is not a viable option, pump (lift) stations must be used to convey the wastewater over topographic high points to where a continuous downward slope can again be attained. Because they convey raw wastewater containing varying sizes of solids, conventional gravity flow sewers are usually restricted to pipe sizes of 8” or larger, although 6” diameter pipes are acceptable for short lengths of sewers serving a small OWRS. Manholes are required at points when the gravity sewers change pipe size, grade, horizontal direction, and generally at intervals $\leq$ 400 ft for purposes of periodic inspection and cleaning of the sewers. The main advantages of gravity sewers are the high flow capacities that can be accommodated and the minimum need for mechanical devices that require constant maintenance.

However, there are several disadvantages to using gravity sewers. Infiltration and inflow (I&I) entering the gravity sewer system through the manholes and pipe joints increases the total wastewater flow. I&I can be largely controlled using modern materials and construction methods and watertight manhole covers. However, eventually, some I & I can be expected to occur as the system ages and the many pipe and manhole joints begin to deteriorate. Poorly constructed and deteriorated joints can also contribute to contamination of the ground water by untreated wastewater leaking out of the sewers. Other disadvantages include: (1) their high capital cost, particularly in areas of high water tables, extensive subsurface rock formations, or unstable soil conditions, (2) the need to maintain a minimum pipe slope may require deep excavations, which often places the sewers well below the normal or seasonal high water table, thus increasing the risk of ground water infiltration, (3) increased potential for infiltration due to larger and more numerous pipe joints than other types of sewers, and (4) little flexibility in the location of gravity sewers.

Design and construction of gravity sewers should conform to the applicable sections of Chapter 2 - Sanitary Sewers/Collection Systems in the New England Interstate Water Pollution Control Commission (NEIWPCC) publication TR-16.

C. Small Diameter Effluent Sewers (SDES)

Small diameter effluent sewers can be used to convey septic tank effluent to an OWRS. Two types of SDES systems are used. These include the variable grade effluent sewer (VGES) and the minimum grade effluent sewer (MGES). A septic tank equipped with an effluent screen will remove settleable solids and a large percentage of suspended solids from raw wastewater. Therefore, the concern for maintenance of a minimum velocity to avoid clogging of a SDES system is not as great as in the case of a conventional gravity sewer system. Plastic piping (e.g. polyvinyl chloride, or PVC) is used for SDES systems because of the corrosive effects of septic tank effluent.

A variable grade effluent sewer (VGES) also conveys septic tank effluent by gravity. However, unlike a MGES, it generally follows the ground surface profile and is not required to maintain a continual downward slope. It also is not restricted to straight-line segments, as is the case of conventional gravity sewers. The force of gravity is still used to move the wastewater, but there can be some uphill sections and other sections in which the sewer flows full and is depressed below the hydraulic grade line.

A depressed section will cause a backup in the sewer until sufficient flow surcharges the sewer, thus developing a hydraulic head sufficient to propel the effluent over the topographic high located downstream of the depressed section. As long as there is a net decrease in elevation from the upstream end to the downstream end of the VGES, the effluent will reach the downstream end despite any negative grade in the system. The concept is similar to that of an inverted siphon used in connection with conventional gravity sewer systems.

The burial depth of the sewer only needs to be sufficient to prevent damage from superimposed loads and freezing problems. Because of the ability to reverse the slope of the pipe in many instances, the problems associated with deep excavations and costly excavation of subsurface rock required for conventional gravity sewers is greatly diminished. In addition, the smaller pipe sizes permit the use of narrower trenches. Thus, the cost of a VGES is usually significantly less than that of a conventional gravity sewer.

A VGES should be constructed of plastic piping materials with cemented or other types of watertight joints that do not restrict the flow path in the piping. Cleanouts rather than manholes provide access to the sewer, are much less costly than manholes to construct and can be constructed with cemented or other types of watertight pipe joints and tightly sealed access caps to minimize I&I. Cleanouts generally consist of a pipe riser extending to the ground surface and connected to a standard “Y” fitting in the sewer via a 45-degree bend fitting. The riser is provided with a secure watertight cap.

VGES systems are best suited for use in moderately rolling terrain. The advantages of variable grade effluent sewers include less costly construction, the ability to avoid obstacles, the ability to design for future increases in flow without concern for minimum velocities due to low initial flows, and less I&I than conventional gravity sewers. The disadvantages include the need to provide odor control facilities due to the conveyance of anaerobic septic tank effluent that can emit odorous gases under turbulent flow conditions, the need to protect against the corrosive effects of septic tank effluent, and the potential difficulty in clearing of blockages. It is also necessary to provide air release valves at the high points and blow-offs at low points and these can also emit odors. For buildings in very flat or low lying areas that are below the hydraulic grade line of the VGES, a STEP system may be necessary to deliver the wastewater to the VGES. A profile of a VGES is shown in Figure XII-1, with dwelling No. 15 requiring a lift station (e.g. STEP system) in order to discharge the wastewater to the VGES.

A MGES system is similar to a conventional gravity sewer system except that the pipe sizes can smaller and the minimum design velocities (at flow depths of full or 1/2 full) can be in the range of 1 to 1.5 ft./sec. Similar to a VGES, the burial depth of the sewer only needs to be sufficient to prevent damage from superimposed loads and freezing problems, but must also be designed to maintain the minimum grades required. A disadvantage is the need to provide manholes, which are not required for VGES systems. Other disadvantages are similar to those of VGES, including the need to provide odor control facilities and to protect against the corrosive effects of septic tank effluent. The cost savings obtained by use of a MGES system are not as great as those resulting from use of a VGES.

For both VGES and MGES systems, careful estimates of flow rates are required because of the smaller pipe diameters that do not provide the reserve capacity normally available in conventional gravity sewer systems. Also, as is the case wherever plastic piping is installed below ground, a means must be provided for locating the piping for maintenance and repair purposes. This can be done by installing magnetic tracing tape in the trench above the piping and by maintaining accurate as-built records of piping location. The use of either a VGES or MGES system requires a program for septic tank pumping and maintenance. Guidance for design of VGES and MGES is given in publications listed in the references at the end of this section.

D. Vacuum Sewers

A vacuum sewer system makes use of differential air pressure to convey the wastewater. The system includes a vacuum/gravity interface valve installed in a raw wastewater-receiving sump at each point of entry into the system, a collection piping system operating under negative air pressure (vacuum), and a centralized vacuum pump station containing a sealed collection tank, vacuum pumps and wastewater pumps. The vacuum created in the sealed collection tank by the vacuum pumps provides the force required for conveying the wastewater. Wastewater reaching the sealed collection (vacuum) tank via the vacuum piping system is then pumped from the tank into a pressurized pipeline (force main) that conveys the wastewater, either directly or via another conveyance system, to a treatment facility.

The interface valve in the receiving sump is normally closed so as to maintain the vacuum in the piping downstream of the valve. Wastewater flows through the building sewer by gravity to the receiving sump that is normally installed underground. When the wastewater collected in the sump reaches a predetermined level, the interface valve opens to the atmosphere for a few seconds. The resulting differential between atmospheric pressure and the vacuum in the piping system provides the energy needed to propel a slug of wastewater through the valve and into the vacuum piping system. The forceful and rapid entry of the wastewater into the vacuum sewer serves to break up large solids in the wastewater. The vacuum also keeps the lines clean, so manholes and clean-out points are generally unnecessary. There are no electrical connections required at the building(s) being served; a power supply is necessary only at the vacuum station.

The vacuum pipelines are constructed of small diameter (4"-10") plastic pipe, having a pressure rating of 200 psi. The pipes are installed in narrow trenches in a saw tooth profile for level grade and uphill transport and follow descending grades for downhill transport. The saw tooth profile keeps sewer lines shallow and is designed to ensure that sewage will not block the pipe at low flow periods when the wastewater is at rest. Vacuum lines are slightly sloped (0.2%) toward the collection station.

Vacuum systems can be designed to suit a variety of site conditions but have limited capabilities for transporting wastewater uphill (usually a maximum of 15 to 20 ft). They are best suited for areas with flat or gently rolling terrain where shallow bedrock, difficult soil conditions and high water tables are encountered such as in lake and coastal areas.

Among the disadvantages of a vacuum sewer system are the higher O&M costs associated with maintaining the vacuum interface valves and vacuum pump station equipment. A vacuum sewer system requires skilled maintenance personnel. Repair or replacement of vacuum interface valves is required at periodic intervals (i.e. every 5 to 10 years), and more effort is involved in maintaining the vacuum and sewage pumps in the main vacuum collection stations. Odors can be emitted from the vacuum pump exhaust piping. To control the odors, it is necessary to discharge the exhaust air from the vacuum station to an odor control facility.

To date, vacuum sewers have not been used in large-scale onsite systems in Connecticut. At least one such system has been constructed in Massachusetts (Sullivan, J. et al. 2003). Publications such as U.S. EPA (1991) and AIRVAC (2001) provide further information on vacuum sewers. An idealized Vacuum Sewer System is shown in Figure XII-2.

Design and construction of vacuum sewers should conform to Chapter 2, Section 2.9 - Alternative Collection Systems in NEIWPCC TR-16 and the recommendations of the manufacturer selected to provide the vacuum sewer operating equipment. Where conflicts exist between Section 2.9 of TR-16 and the manufacturer's recommendations, the more conservative requirements should govern.

E. Low Pressure Sewers

A low pressure sewer (LPS) system relies upon pumps to propel the wastewater through pipelines much smaller than conventional gravity sewers and in many cases smaller than SDES systems. Pressure sewers can be laid in shallow trenches to follow the contour of the land and their horizontal alignment does not have to be in straight segments. They are best suited to hilly terrain where gravity flow systems cannot be used, or in flat terrain where shallow bedrock, difficult soil conditions and high water tables are encountered in lake and coastal areas. The two main types of LPS systems are the septic tank effluent pump (STEP) system and grinder pump (GP) system.

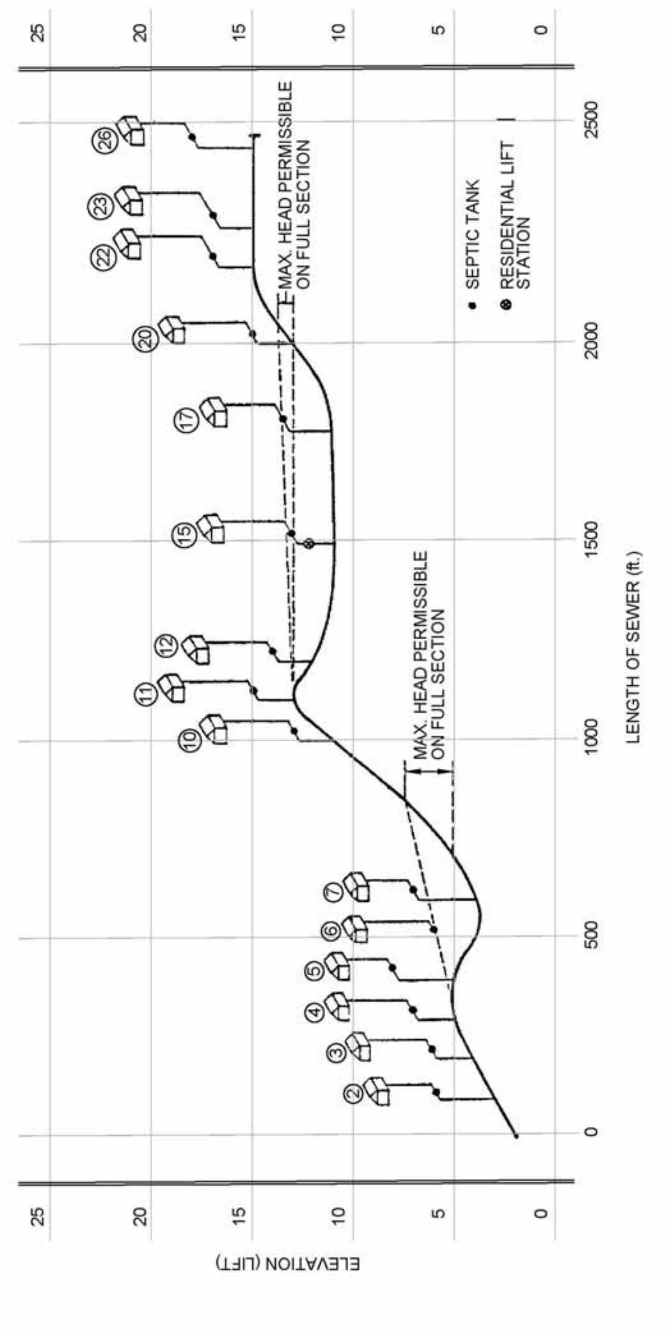

FIGURE XII-1
PROFILE OF SMALL DIAMETER
VARIABLE GRADE EFFLUENT SEWER

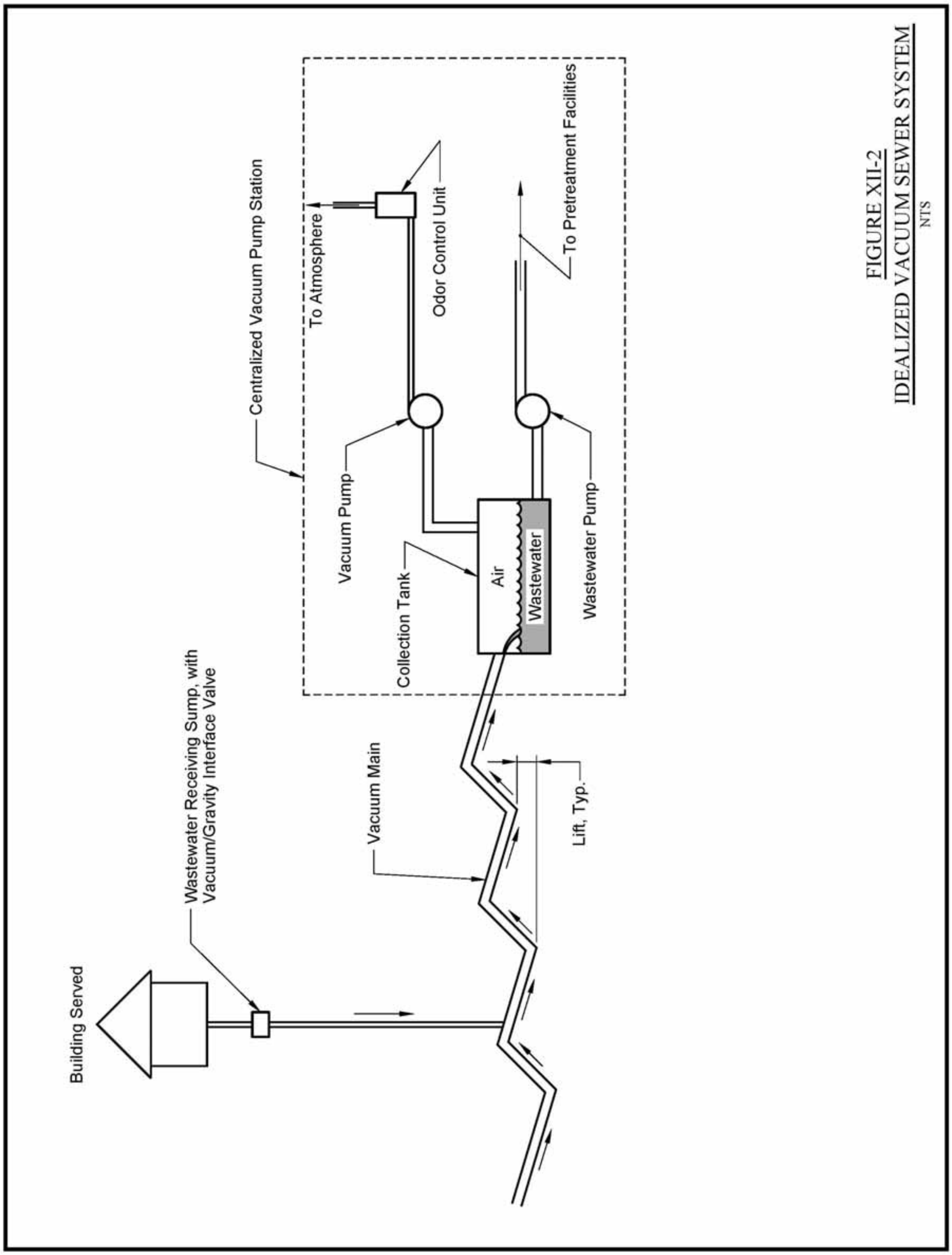

FIGURE XII-2
IDEALIZED VACUUM SEWER SYSTEM
NTS

The STEP system consists of pump chambers that accept the discharge from septic tanks, which is largely free from settleable solids and a high percentage of suspended solids. Therefore, the pumps do not have to have the large solids handling ability of raw wastewater pumps. They are invariably of the submersible, electrically driven centrifugal type, operating under low to moderate hydraulic head conditions and are equipped with level sensing controls to actuate the pumping cycles. The pumps discharge the septic tank effluent into a completely pressurized piping system and thus infiltration through pipe joints is not a concern. In order for a STEP system to be effective, the septic tank(s) must be watertight.

A septic tank effluent pumping system (STEP) is often used for pressure distribution of pretreated wastewater to a SWAS and is also the predominant type of system used in conveying septic tank effluent to an enhanced pretreatment facility.

Plastic piping is invariably used to resist the corrosive effects of the septic tank effluent. The piping normally consists of solvent welded rigid plastic pipe, or piping fabricated from long coils of flexible plastic (e.g.: polyethylene) pipe. The pumps must also be of corrosion resistant construction and electrical controls must be designed for corrosive and potentially explosive environments. Further information on pumping systems is given in the following subsection.

Similar to conditions in SDES systems, septic tank effluent in the STEP system will release odors when turbulent conditions occur, for example, if the effluent is allowed to drop into a manhole or pumping chamber. Turbulent conditions can largely be avoided by use of drop pipes to convey the effluent to below the water level in the receiving structures. Odor problems can also occur via escape of the odorous gases through the pump chamber ventilation system. The simplest method of controlling such odors is by absorption of the malodorous gases released from the vents in soil absorption beds (biofilters).

As in the case of VGES and MGES systems, careful estimates of flow rates are required for proper determination of the pumping capacity required and design of the small diameter pressure piping.

Grinder pump (GP) systems consist of electrically driven pumps equipped with mechanisms for grinding the solids in the raw wastewater into particles small enough to be pumped through small diameter pressure piping systems. Grinder pumps eliminate the necessity to periodically pump a septic tank at each building served by a STEP system and the corrosion and odor problems associated with STEP systems are largely avoided. As is the case for STEP systems, air release valves are required at high points in the pressure piping system. However, there are disadvantages associated with GP systems.

GP systems require more O&M than the various types of gravity sewer systems and the STEP system. Since they carry solids, a minimum velocity of 2 ft./sec. is required to maintain solids in suspension and a minimum velocity $\geq$ 2.5 ft./sec. is required to re-suspend solids that have settled in the pipelines during no-flow periods. In addition, the grinding mechanisms need periodic maintenance and greater inspection frequencies than STEP systems.

Where GP systems are permitted to discharge to septic tanks in onsite wastewater renovation systems, the tanks must be specially designed to mitigate the adverse effects of receiving wastewater containing a slurry of small ground up solids mixed with fats, oil and grease (FOG). These adverse effects create difficulty in the separation of the solids from the wastewater. A scum layer deeper than usually found in a septic tank may also develop because of the film of FOG coating the ground solids. Mitigation of these effects will involve larger tank volumes and enhanced provisions for baffling of the incoming flow to avoid stirring of the tank contents that can result in rapid clogging of the septic tank effluent filter.

In general, pumping wastewater into septic tanks should be avoided because of the adverse effects on the relatively quiescent conditions required in the septic tank. The Department will normally not approve of a system that involves pumping wastewater into a septic tank unless convinced that special conditions make it unavoidable and that special design features will be provided to insure satisfactory operation of the septic tank.

Design and construction of LPS systems should conform to Chapter 2, Section 2.9 - Alternative Collection Systems in NEIWPCC TR-16 and the recommendations of the manufacturer selected to provide the grinder pump equipment. Where conflicts exist between Section 2.9 of TR-16 and the manufacturer's recommendations, the more conservative requirements should govern. Guidance for design of LPS systems is also given in NSFC publications listed in the references at the end of this section.

F. Force Mains.

Force mains used for onsite wastewater renovation systems normally convey pretreated wastewater from a pumping station or chamber and deliver it under pressure to downstream facilities. Design of force mains involves determination of:

- The type of material to be used (e.g. ductile iron, various plastic materials) for pipe, fittings and valves,
- Required minimum and desirable maximum velocities,
- Normal operating pressures,
- Hydraulic head loss due to friction losses and form losses at fittings and valves, and
- Pressure surges (water hammer) caused by rapid starting and stopping of the pumps.

Force mains are designed to maintain minimum velocities under all flow conditions. However, most of the solids in the wastewater that remains in the force main during no-flow conditions will settle out to the bottom of the piping. Where raw wastewater is being pumped, a minimum velocity of ≥ 3 ft./sec. is generally required to scour these solids and re-suspend them in the wastewater. Where pretreated wastewater is being pumped, the minimum velocity should be ≥ 2.5 ft/sec. Maximum velocities will depend on the design flow and selected pump capacities. Generally, velocities should not greatly exceed the required scouring velocity, since higher velocities require greater use of electrical power. However, this may not be possible where the design flows are significantly greater than initial flows.

Pumping facilities will normally be equipped with duplex pumps, as discussed in the following subsection. Therefore, it is possible to design for a 2-ft./sec. velocity under initial flow conditions with a scouring velocity induced once a day by having both pumps operate simultaneously for a short period of time. This can be accomplished under time clock control. However, this complicates the control system of the pumps, as it requires overriding the system provided for alternating the operation of the pumps and may induce velocities greater than the required scouring velocity, which may result in excessive flow rates causing problems at downstream facilities. Thus, using this method of insuring scouring velocities should be approached very carefully.

The type of pipe material used is based on the nature of the liquid being pumped (corrosivity, grit content) and the anticipated pressures (normal and surge) that will be exerted on the pipe.

Force mains used for conveying septic tank effluent should be constructed of pressure rated plastic pipe such as PVC and polyethylene (PE) because of the corrosive nature of the effluent. PVC pipe of the push-on gasketed type should conform to AWWA Standard C900 and associated fittings should conform to AWWA C907. Ductile iron fittings of the push-on gasketed type or mechanical joint type with interior epoxy coatings suitable for use in septic tank effluent environments are also available. Ductile iron pipe and fittings should conform to the applicable AWWA and ANSI standards for ductile iron pipe and fittings.

Where gate valves are used, they should be of the single gate, rather than double-disc type, because of the possible fouling of the open spaces between the discs by solids contained in the wastewater. Resilient seated single gate type valves suitable for operating in the corrosive atmosphere created by septic tank effluent are available with bronze shafts and trim, elastomer coated gate seats and epoxy-coated bodies. These valves should conform to AWWA C509. Eccentric-type plug valves are also available for use in wastewater conveyance systems.

PVC pipe conforming to ASTM standards for Schedule 40 and Schedule 80 PVC pipe and fittings conforming to ASTM standards for PVC fittings for such pipe are also suitable. Threaded joints should be avoided because of the possibility of threaded joints failing at the root of the threads due to bending stresses. For small diameter force mains, PVC ball valves are also suitable.

Force mains used for conveying effluent from enhanced pretreatment processes can also be constructed of cement-lined ductile iron pipe and fittings. Joint restraints or concrete thrust blocks should be provided for all force main piping at changes in direction of the piping.

Valves in force mains are normally used in either open or closed position, but may also be used for throttling pump flow in certain cases. However, resilient seated gate valves should not be used for pump throttling unless approved for such service by the manufacturer.

When pumps are started or stopped, or valves are closed very quickly, a pressure surge (water hammer) condition occurs in the force main that can result in high stresses being applied to the piping that could cause pipe failure. For short, gradually rising force mains (<1,500 ft) of small diameter, with small static heads, typical for on-site wastewater conveyance situations, the surge pressures are usually not excessive. Slamming of the check valve on the discharge side of the pump will usually occur, depending upon the amount of the surge pressure. If a check valve should stick in the open position, the pump can reverse direction due to backflow, which is an undesirable condition. To prevent this from happening, check valves used on pump discharge piping should be of the resilient seated swing gate type with outside lever and weight or other types specifically designed for used in wastewater pumping systems.

Cushioned type outside lever and weight type check valves equipped with hydraulic or pneumatic cylinders are available that mitigate against banging of the valve. For long force mains, detailed investigation of the pressure surges that may develop is imperative and means must be taken to alleviate such conditions, such as the installation of a surge control device, soft start motor starters, or variable speed pumping systems. An interesting discussion on pressure surges in force mains is provided in Metcalf & Eddy (1981) and in the Uni-Bell Handbook (2001), and additional information may be found in other standard references on pressure flow.

Design of force mains should conform to Chapter 2, Section 3.7 - Force Main in NEIWPCC TR-16 and the guidelines presented above. Where conflicts exist between Section 3.7 of TR-16 and the guidelines given above, the more conservative requirements should govern.

G. Construction Practices For Sewers and Force Mains

Construction methods, including leakage testing, for sewers and force mains should conform to the applicable requirements of NEIWPCC TR-16, guidelines and recommendations of the pipe manufacturers and any association of manufacturers for the type of piping material being used, Federal, State and local safety codes, and good practices of the construction industry. Where conflicts exist between NEIWPCC TR-16 and the guidelines and codes referenced above, the more conservative requirements should govern.

Except as noted below, wherever plastic piping is installed below ground a magnetic tracing tape should be installed in the trench to permit locating the piping for maintenance, repair or replacement. Warning (marking) tape should be used in lieu of magnetic tracing tape for all other types of buried piping.

H. Pump Chambers

1. General

Virtually all pumping stations and chambers (hereinafter referred to collectively as pump chambers) used as part of on-site wastewater renovation systems utilize pumps of the submersible, electrically driven, centrifugal type, installed in precast concrete structures that also serve as wet wells. In some cases, durable, long lasting materials other than precast concrete may be used for the pump chambers. Unless otherwise permitted by the Department, the pumping system should include at least two pumps of equal capacity (duplex type), arranged for operating on alternate duty cycles so as to equalize the pumping duty and be of the slide-rail mounted lift-out type. Submersible pumps should be maintained in a submerged condition at all times so as to preclude formation of a vortex at the pump suction inlet that can lead to air binding or a reduction in pumping efficiency. Information should be obtained from the pump manufacturer as to the depth of submergence required.

The control systems for these pumping facilities can be either installed inside a structure or at outdoor locations adjacent to the pump chamber. Pump check valves may be installed on the pump discharge piping within the pump chamber, if they can be lifted out with the pump, or in valve chambers installed in close proximity to the pump chamber. Pump isolation valves should always be installed outside of the pump chamber to avoid having to enter the chamber for operation and maintenance purposes. Pump isolation valves may be installed in a separate valve chamber (along with check valves) or as direct burial valves equipped with valve boxes. Where drain back of piping downstream of the pump chamber is required after a pumping cycle, and check valves are installed within the pump chamber, a drain hole fitted with a long brass nipple and brass elbow angled downward should be provided in the vertical discharge piping, downstream of the check valves, where it will be least subjected to clogging. The direction of the discharge from the drain hole fittings should be such as will not interfere with normal operation and maintenance activities.

2. Pumps

Pumps should be suitable for long-term operation and be designed specifically for pumping the particular type of wastewater and for operation under the most severe ambient conditions in which they will be operated. Pumps used for pumping septic tank effluent should be of the explosion proof type, unless they remain fully submerged in the wastewater at all times. While liquid level controls can ostensibly be designed to provide submergence of the pumps, there is always the possibility that these controls may malfunction. Therefore, the safest procedure is to provide pumps of explosion proof design whenever possible. Pumps of explosion proof design should be certified by the manufacturer for use in Class I, Division I, Group C and D hazardous locations as specified in the National Electric Code (NEC).

Pumps used for pumping of effluent in and from enhanced pretreatment processes do not need to be explosion proof but should be designed to operate submerged at all times. Pumps should be capable of delivering the design flow at the total design head. The pumps should be non-overloading at any point on their characteristic curve and should not operate very close to their shut-off head.

Each pump should be equipped with a submersible type oil-filled motor suitable for operation at the voltage and phases of the power supply source. Power cords should be long enough to extend to the full height of the pumping chamber and through the wall of the chamber to an outside mounted junction box. The motor should have heavy-duty ball bearings to support the pump shaft and take radial and thrust loads. Normally closed automatic reset thermostats connected in series should be embedded in adjoining phases of the stator windings and should be connected to stop the motor if the temperature of the winding exceeds the maximum permissible operating temperature.

Pumps should be of double seal construction using two mechanical seals in tandem with an oil chamber between the seals. Seals should be readily commercially available from third party sources other than the pump and motor manufacturer. Wherever possible, the oil chamber between the seals should be fitted with a moisture sensing probe system to detect the entrance of moisture into the lower oil seal housing and provide an alarm to the pump control panel. The probe should activate a red seal failure light in the pump control panel.

Wherever feasible, replaceable wear rings should be provided at the suction inlet and impellers should be statically and dynamically balanced and equipped with replaceable brass wear rings. An approved corrosion resistant coating should protect all surfaces coming into contact with wastewater other than bronze and stainless steel. The common motor pump shaft and all fasteners should be stainless steel.

Separate pump motor power and control cables, and the method of connecting these cables to the equipment being operated should be suitable for the atmosphere in which they will be located. Cable sizing should conform to NEC specifications for pump motors. Both power and control cords should include separate grounding conductors and strain relief mechanisms should be provided at all points of connection.

3. Slide Rails and Appurtenances

There should be no need for personnel to enter the pump chamber in order to remove or re-install the pumps. Each pump, and attached check valve if provided, should be automatically connected to the discharge piping when lowered into place on a guide (slide) rail system, requiring no bolts, nuts or fasteners to effect leak proof sealing to the discharge connection. The pump, and attached check valve if provided, should be easily removed for inspection or service. A simple linear downward motion of the pump should accomplish sealing of the pumping unit to the discharge connection. The weight of the pump should be supported by and bear solely on the discharge base and not on the guide rails or chamber bottom.

The slide rail system should be of non-sparking design and should be listed for explosion proof service when installed in pump chambers containing septic tank effluent. Slide rails and appurtenances should be manufactured of materials that will have a long-term resistance to corrosion and be of sufficient strength to bear all of the loads imposed on them. Slide rail top and intermediate support brackets should be adjustable to permit accurate vertical alignment of rails. A corrosion resistant lifting chain of adequate strength and length to permit raising and lowering of pumps should be provided. All hardware should be stainless steel.

4. Float Switches

Sealed float-type switches should be supplied for pump control and alarm signals. The switches should be sealed in a leak-proof solid polyurethane float for corrosion and shock resistance. The support cable for each float switch should have a heavy neoprene jacket suitable for the environment in which it will be used and should be equipped with a weight attached to the cable to hold the float in place in the wet well. The weight should be positioned above the float to prevent sharp bends in the cord when the float operates under water.

All float switches should be supported only by cables attached to a wiring support fixture. Three float switches should be provided to control liquid level in the wet well: one for stopping either or both pumps, one for lead pump start, and one for lag pump start. Two additional float switches should be provided for alarms; one for low liquid level alarm and one for high liquid level alarm.

Wiring supports with cable grip holders should be provided near the top of the pump chamber for the pump cables and the control and float switch cables. The supports should be easily accessible from the access hatch of the pump chamber and should provide for easy adjustment of cables to the desired switch operating levels without having to enter the pump chamber.

As an alternative to float switches, pressure transducer type switches designed for service in wastewater systems can also be used.

5. Pump Control Systems

The control system for the wastewater pumps should contain:

- a main circuit breaker designed to disconnect all power conductors,
- individual circuit breakers for each pump,
- magnetic motor starters,
- three pole ambient compensated quick trip overload relays,
- MANUAL-OFF-AUTOMATIC selector switches,
- automatic lead-lag pump alternation, with automatic hold feature to allow continued operation of a single pump if the other pump has failed.
- lead pump start, lag pump start, pump stop, low level alarm and high level alarm level sensing via float switches,
- Pump run lights,
- alarm lights for high liquid level and low liquid level in the pump chamber,
- a seal failure alarm light for each pump,
- an over-temperature light for each pump with manual light reset,
- an elapsed time meter and amp meter for each pump,
- auxiliary contacts for remote high level, low level and loss of power alarms,
- a strip heater to prevent accumulation of condensed moisture within the panel,
- lightning/power surge arrestors, and
- Legend plates, clearly labeled, durable, and securely fastened in position, to identify all switches, meters, run lights and alarm lights.

A main circuit breaker and separate auxiliary circuit breakers should be provided for alarm and control circuits. Control power should be a maximum of 120 volts. Intrinsically safe relays should be provided for operation of all float switches.

The alternation system should provide for alternate pump operation on each successive cycle, and include an override circuit to start both pumps if the level rises in the pump chamber or to start the second pump if one pump fails.

An alarm switch should be provided for on-off, acknowledge, reset and test. Auxiliary dry contacts should be provided to send signals for high liquid level, low liquid level and loss of power alarms to a remote monitoring system via a dialing alarm monitor or other suitable equipment. The remote monitoring system shall be capable of alerting personnel assigned to providing a rapid response to, and correction of, any malfunction at the pump chamber. Seal failure and over-temperature alarms may be indicated by local alarm lights mounted in the control panel if frequent inspections of the pump chamber are conducted.

Audible and visual alarm facilities, suitable for outdoor use, should be provided that will be actuated by any of the alarm indications provided to the remote alarm monitoring system. The alarm facilities should be mounted in a conspicuous location where the alarms can be heard and seen by system users to warn of pump failures. An inside mounted alarm bell and light should also be provided at a conspicuous location within the premises being served, unless an inside mounted pump control panel is similarly located.

6. Pump Control System Enclosures

The pump control enclosure should conform to the NEMA 250 standards for the atmosphere in which it will be located and be of sufficient size to accommodate all pump controls. The selected manufacturer of the enclosure should be advised of the ambient atmosphere to which the enclosure will be exposed.

Pump control enclosures should conform to one of the following NEMA Types.

- Type 1, suitable for indoor use in dry locations.
- Type 2, suitable for indoor use where the enclosure may be subject to dripping and light splashing of liquids.
- Type 3R, suitable for outdoor use, where the enclosure may be subject to rain, sleet and snow and the external formation of ice on the enclosure.
- Type 4, suitable for outdoor use, where the enclosure may be subject to rain, sleet, snow, wind-blown dust, splashing water and hose-directed water and the external formation of ice on the enclosure.
- Type 4X, suitable of indoor and outdoor use, where the enclosure may be subject to the same conditions as a Type 4 enclosure and is also subject to a corrosive atmosphere.
- Type 7, suitable for indoor use in hazardous locations classified as Class I, Division 1, Groups A, B, C, or D as defined in NFPA 70 (National Electric Code).
- Type 8, suitable for indoor and outdoor use in hazardous locations classified as Class I, Division 1, Groups A, B, C, or D as defined in NFPA 70 (National Electric Code).

The control enclosure should be provided with a full dead-front locking type outer door and all switches, lights, meters, etc. should be mounted on a hinged inner door. A schematic wiring diagram and overload heater chart should be displayed on the inside of the panel cover. The control panel and control enclosure should be completely assembled and pre-wired at the place of manufacture and should conform to IEEE, NEMA and National Electric Code design standards and requirements.

All wiring should conform to NEMA wiring standards. Multi-colored circuits should be used within the control panel enclosure to facilitate troubleshooting. Solder-less type terminal blocks, with individually labeled wire terminals, should be provided for connection of power supply, motors and controls. All wiring should be multi-stranded and neatly bundled and color-coded. Both ends of all wires should be clearly numbered with permanent markers. The control panel should include all auxiliary devices such as relays, auxiliary contacts, interlocks, selector switches, etc., required to provide the specified functions.

7. Pump Chamber Structures

Structures (e.g.: rectangular tanks, circular manhole sections) serving as wet wells housing submersible pumps should consist of precast reinforced concrete sections, complying with applicable provisions of ASTM C478, C890 and C913. The concrete should have a minimum compressive strength of 4000 psi at 28 days. Air entrained concrete should be used. Design of the pump chambers should include anti-floatation provisions to prevent dislocation by water pressure due to high water table conditions.

Accurately cast rigid pipe sleeves with integral water stop should be cast into wall sections at the proper locations and elevations for connection of piping and electrical conduit to the pump chamber. The sleeves should be of proper size to accommodate piping and conduit and the materials used to provide a watertight seal of the pipe to the structure.

Guidance for flexible, watertight connection of piping to concrete structures is also provided in ASTM C923-02. Sealing of electrical power and float cables in conduits at Class I, Division I, Group C and D hazardous locations should be made as specified in the National Electric Code (NEC).

All exterior surfaces, except where above grade or in contact with concrete access risers, masonry manhole frame risers, or anti-flotation collars, should receive a damp-proof coating. All joints should be sealed with a self-sealant butyl based rubber gasket or other approved sealant material.

Precast concrete access risers extending from the top of the structure to an elevation at least several inches above finished grade should be provided for removal and maintaining the pumps and for inspection of the pumping chamber. Risers for pump removal shall be of rectangular cross-section equipped with lockable hinged access covers. Other risers provided for chamber inspection and cleaning can be of circular cross-section equipped with manhole frames and covers that can be secured in such manner as to prevent vandalism. They should comply with the same manufacturing and construction requirements set forth above for the main pump chamber.

Each pump access riser should be complete with a cast-in-place corrosion resistant (e.g. aluminum) access hatch frame and cover that will provide ample room for removing and replacing the submersible pumps. Risers should be cast and set level and plumb. All riser joints should be sealed in a manner similar to that used in sealing the pump chamber. Provisions should be made for securely attaching the riser to the top of the pump chamber. When grouting is complete, all grout surfaces should also be sealed with a damp-proof coating.

Access hatch covers and frames should be fabricated of corrosion resistant material (e.g. aluminum) and should consist of single leaf or double leaf doors that open with spring assist to 90° and automatically lock in the open position with stainless steel hold open arms with aluminum release handles. Hatch doors should be designed to support the maximum load likely to be imposed and should be equipped with a lock and removable key and a non-corrodible lifting handle. Hold open arms incorporating enclosed stainless steel compression spring assists should be provided. Doors should be designed to close flush with the frame and rest on a built-in odor reducing neoprene gasket. All lifting handles and hinges shall be completely flush with the hatch covers and frame when in the closed position. Hinges and all fastening hardware should be stainless steel. All access hatch doors should be provided with safety anti-fall nets with stainless steel fastening hooks or other suitable anti-fall safety devices.

The working volume (liquid volume between normal pump start and stop levels) for duplex pumping systems will depend upon the function of the pumping facilities. Where the pumps will be delivering wastewater to downstream pretreatment facilities, and other considerations do not control, the working volume should limit the pump cycles to six or less per hour in order to prevent overheating of the pump motor and excessive wear on the magnetic starter contacts.

In this case, the minimum cycle time for single pump operation occurs when the inflow is exactly half the pumping capacity. Under this condition, the on and off times are equal. The cycle times are greater for the cases when the inflow is larger or smaller than half the pumping capacity.

This leads to the following formula for calculating minimum working volume (Metcalf & Eddy -1981):

Working Volume, V, in gallons, = 0.25 x (minimum pump cycle time*, in minutes) x (pump capacity, in gallons per minute).

* time between successive starts. Thus, if six cycles per hour are desired, the cycle time is 60 min./hour/6 cycles per hr. =10 minutes. Thus, the working volume = 2.5 x pump capacity.

It should be noted that the working volume could be reduced by half where a duplex pumping system contains identical pumps and where alternation is provided. However, consideration should be given to the circumstances that would prevail should one pump fail and an extended period of time is required for repair or replacement of the pump.

In addition to establishing a minimum working volume, additional volume should be provided for proper pump submergence and for incremental settings of the various float switches. Generally, an increment of 6 inches should be provided between successive float actuation levels. Thus, the float switch that causes the lead pump to start should be set at the liquid level elevation established by the total of the working volume and the pump submergence volume. The high water alarm float switch should be set to actuate the alarm when the liquid level rises 6 inches above the lead pump start level. The lag pump switch should be set to start the lag pump when the liquid level rises 6 inches above the high water alarm level. The high water level should be such as not to cause a backwater condition in the wastewater inlet piping.

Additional reserve volume should be provided in the pump chamber for emergency storage in incoming wastewater for a period of time sufficient for operating personnel to respond to a pumping station malfunction. A minimum of 4 hours of reserve volume should be provided. Where pump chamber malfunction is due to a power outage, and an emergency source of power cannot be made available within a 4-hour period, at least 24 hours of reserve volume should be provided. The Department may waive this requirement if it can be shown that the power outage will also result in interruption of wastewater flow from the source being served. Where the required reserve volume cannot reasonably be provided within the pump chamber, an emergency overflow should be provided to a temporary holding tank. Provision should be made for automatic return of the holding tank contents upon restoration of normal pumping operations.

Where the pumping facilities are used for flow equalization, or recirculation, or for pressure distribution of pretreated wastewater to an SWAS, or for wastewater transfer between enhanced pretreatment facilities, the working volume will be dictated by such uses. However, the pump cycle times should always be checked to insure that excessive pump cycling will not occur.

Where the submersible pumps serve to discharge pretreated wastewater to a SWAS, it is good practice to set the pumps on a concrete platform whose top is 12 inches above the bottom of the pump chamber. This will minimize any solids remaining in the wastewater that settle out in the pump chamber from being drawn into the pumped liquid and discharged to the SWAS. (Periodic inspection of the pump chamber and the removal of any accumulated solids deposits should be a regular part of the normal O&M procedures.)

Where the wastewater received at a pumping chamber is corrosive in nature (e.g.: septic tank effluent), it should be discharged into the chamber below the low liquid level via a drop pipe to prevent splashing and release of corrosive gases.

Provisions should also be made for passive ventilation of the pump chambers. Where septic tank effluent is being pumped, provisions should also be made for control of odors emanating from the ventilation piping. Either activated carbon canisters or biofilters can be used for odor control, with subsurface biofilters being preferred if there is adequate space available for their construction.

8. Valves and Valve Chambers

All pump isolation valves should be installed outside of the pump station or chamber. They may be installed underground with valve boxes for access, or they may be installed in a valve chamber located adjacent to the pump station or chamber. Where check valves are not an integral part of the submersible pump lift-out mechanism, they should also be installed in a horizontal position in a valve chamber. Where drain back of the pressure piping after pumping has stopped is required, provisions must be made for draining the piping downstream of the check valves back to the pump chamber. The design and fabrication of the valve chamber and provisions for access to the valves should be equivalent to that of the pump chamber. Design of the valve chambers should include anti-floatation provisions to prevent dislocation by water pressure due to high water table conditions.

Typical details for a pumping chamber with underground valves are shown in Figures XII-3 and XII - 4.

9. Selection of Materials and Equipment

All materials, equipment, and the coatings applied to them, that are selected for use in wastewater conveyance systems should be certified by the manufacturer as being suitable for long-term heavy duty use in the environment in which they will be utilized.

I. Infiltration and Inflow

Every effort must be made to reduce infiltration and inflow into the wastewater conveyance system, as any extraneous liquid can have an adverse and sometimes severe impact on pretreatment facilities and the ability of the SWAS to accomodate the increased discharge.

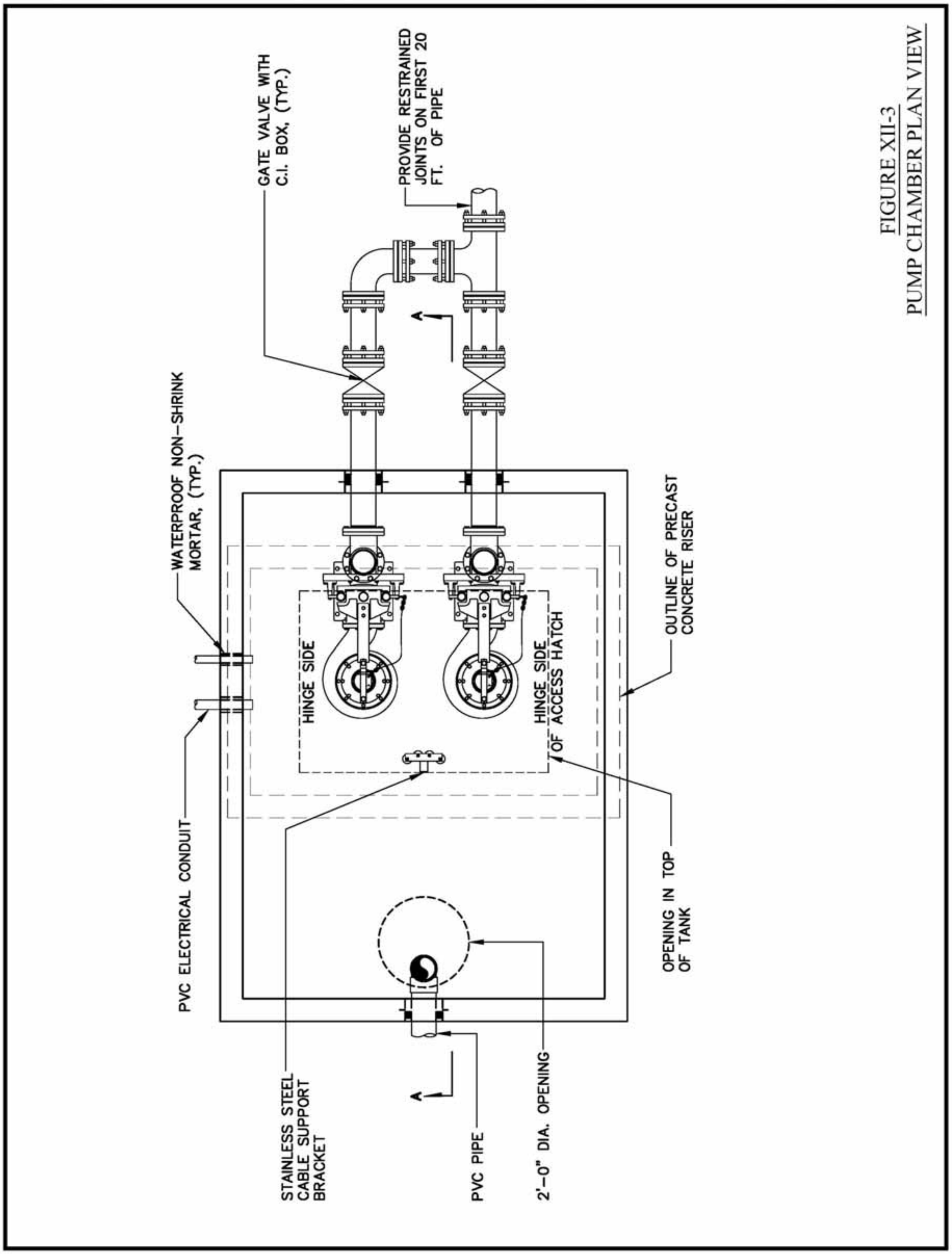

FIGURE XII-3
PUMP CHAMBER PLAN VIEW

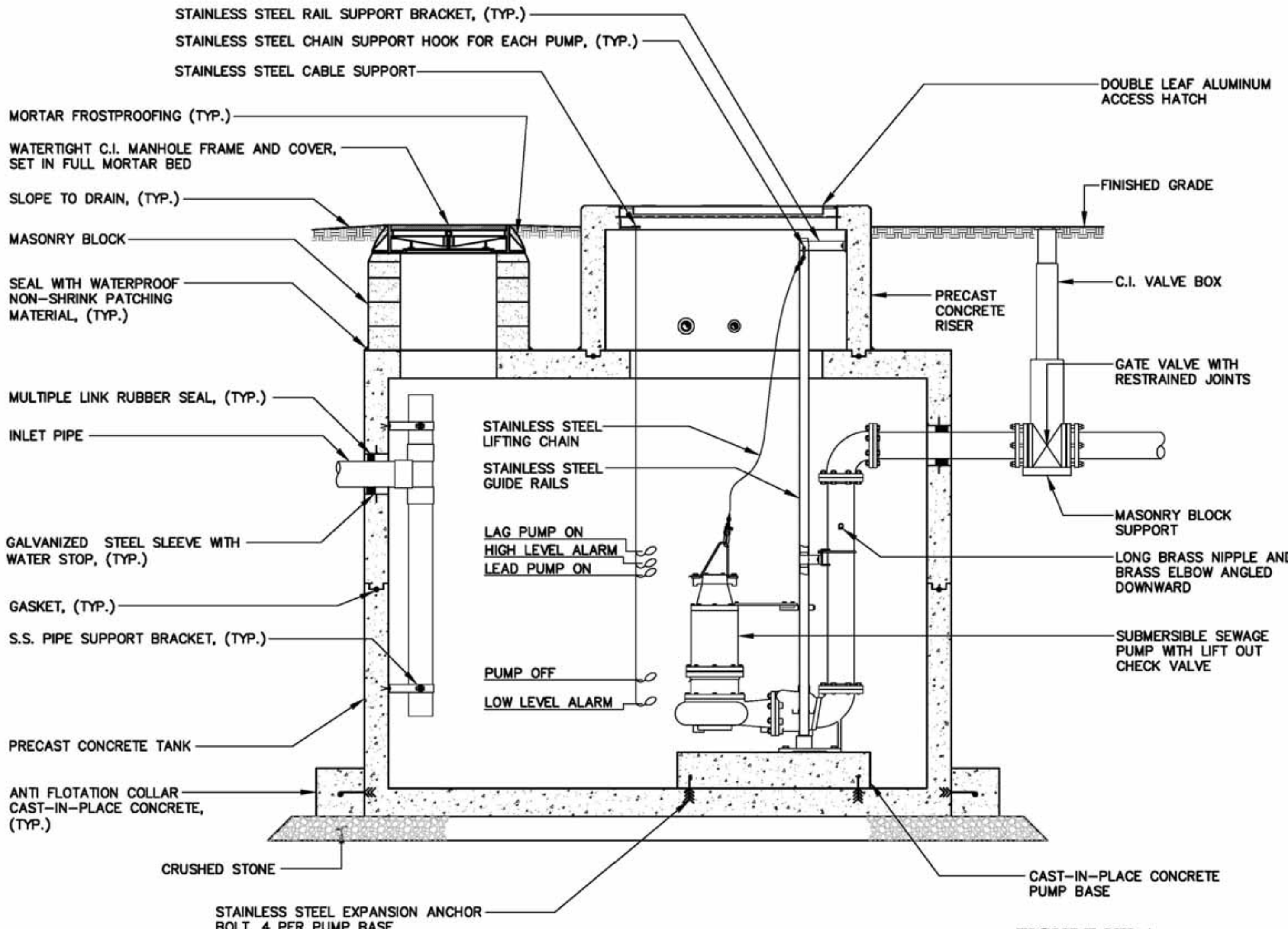

FIGURE XII-4

SECTION THROUGH PUMP CHAMBER

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SECTION XIII QUALITYASSURANCE AND QUALITY CONTROL

TABLE OF CONTENTS

SECTION XIII QUALITY ASSURANCE AND QUALITY CONTROL

A. Introduction

Control of the quality of an onsite wastewater renovation system (OWRS) project begins with the investigations to determine project requirements and site, soil, and ground water conditions. It is not completed until the work of the contractor(s) is completed, all tests are satisfactorily completed, all of the construction work has been finally approved by the engineer(s) responsible for design and construction services and accepted by the Owner, and any contractor and manufacturer guarantee periods have expired. Previous sections of this document have dealt with investigations for determining requirements for and design of various components of an OWRS. This section addresses some of the general considerations that must be given to the overall design of the OWRS facilities and to assuring that the OWRS facilities are constructed in accordance with construction documents (drawings and technical specifications) approved by the Department and will operate as intended.

The quality of construction will depend to a great extent upon the qualifications and experience of the contractor(s) selected to construct the OWRS facilities; equally important, however, is the quality of the construction documents. If these documents are not clear, complete, concise and correct, the contractor(s) will not have the guidance they require to arrive at the quality desired for the completed project. The contractor(s) should not have to guess at the intent of the project designers; it should be clearly and unambiguously set forth in the construction documents. Almost inevitably however, while perfection is sought it is rarely attained and conflicts and omissions will occur despite all the skill, experience, and good faith effort of those that have prepared and reviewed these documents. When such conflicts and omissions arise, they should be rectified promptly and in a decisive manner, so as to leave no doubt as to what is required.

Construction drawings and technical specifications are supportive of each other; carefully prepared drawings will not eliminate the problems that will most certainly arise where equal care has not been paid to the preparation of the written specifications. Good communication with the contractor(s) regarding the project requirements depends upon having complete and fully coordinated construction documents.

B. General Design Considerations.

Design considerations for basic and enhanced pretreatment facilities and subsurface wastewater absorption systems have been presented in previous sections of this document. There are additional considerations that must be given with respect to providing the quality of facilities required for satisfactory long-term service. Some of these considerations are discussed below; however, the designer should consider all aspects of the operation, maintenance, monitoring and security of such facilities.

Aboveground facilities should be designed to present a neat appearance that will engender confidence in the ability of such facilities to carry out their functions. A sloppy appearance will tend to convey to observers a sense of careless operation and maintenance, regardless of how well the O&M is performed. It may also cause a lackadaisical approach to O & M by the operator.

Attractive and functional enclosures for equipment and controls, where not a part of another structure meeting building code requirements, can be obtained as small prefabricated, transportable enclosures, or prefabricated, field-erected enclosures. They can also be constructed on-site using basic building materials. The external surfaces of such enclosures should be clad with long-lasting materials requiring minimum maintenance.

These enclosures should be provided with adequate provisions for heating, ventilation and lighting. The enclosures should be made as vandal-proof as possible. In addition to secure access-ways, any windows should be small enough and high enough above surrounding finished grades to prevent unauthorized entrance and should face in the direction of occupied buildings. The enclosures should conform to the various codes applicable (e.g.: National Electric Code, State and local building and plumbing codes, NFPA recommendations). The enclosures housing equipment and controls should be insulated. All insulation should be covered by rigid, durable wall and ceiling materials that are suitable for washdown or other cleaning methods. The interior walls and ceilings should be coated with materials that will permit ease of cleaning and will protect their surfaces from dampness resulting from cleaning processes or other causes.

A neat and logical placement of equipment and associated piping and electrical conduit within enclosures housing such facilities should also be provided. The facilities and equipment within such enclosures should be laid out in such a manner as to provide for easy access for servicing equipment, delivery of chemicals, pumping of tanks, etc. Adequate space should be provided around electrical control panels as required by the National Electrical Code and protective rubber mats provided wherever a person may stand while operating or maintaining such panels. Spill prevention facilities should be provided in all areas dedicated to storage and feeding of chemicals.

Exposed piping and conduit should run parallel or perpendicular to walls wherever possible. Flexible connectors should be provided between rigid electrical conduit and electrically operated equipment. Air breaks and reduced pressure backflow prevention devices should be provided between potable and non-potable water lines in conformance with the requirements of the Connecticut Department of Public Health. Hose bibs for wash down of structures and equipment should be provided, fed from sources downstream of backflow prevention devices. All exposed piping, conduit, and equipment should be coated with suitable materials that will provide long-lasting corrosion resistance.

The ability to monitor the operations of the various facilities of an OWRS at a remote location during periods when the operator is not present on-site is vital to successful operation of such facilities. Provisions for monitoring and control of pretreatment processes are discussed in subsection N of Section XI of this document and provisions for monitoring pump chambers are discussed in subsection H of Section XII. Facilities for transmitting alarm signals to remote locations for remote monitoring of alarm conditions should be provided. This can be accomplished using a telephone dialing alarm monitor or by more sophisticated radio transmitting alarm equipment. The dialing alarm monitor system has proven to be reliable and robust and is a cost-effect method for monitoring alarms.

Radio alarm transmission equipment is useful, and may be required in certain instances, for sending alarms to central monitoring agencies. Radio alarm transmission equipment has the advantage of not being affected by telephone line outage conditions, and provides essentially the same alarm functions as a telephone dialing alarm monitor.

The purpose of a dialing alarm monitor is to monitor the status of alarms and report any alarm to persons assigned to respond to system malfunctions. When the alarm system for the OWRS facilities detects a fault, a signal is sent to the dialing alarm monitor. The monitor will then begin to place a series of telephone calls, dialing in succession a number of pre-programmed telephone numbers until a number is answered and the alarm message is delivered. Once acknowledged by the receiving party, the system enters a programmable inter-call delay to allow the alarm condition to be corrected before dialing the next telephone number. Communications are transmitted via a standard telephone line dedicated to the dialing alarm monitor.

The dialing alarm monitor should include provisions for voice message recording and an internal speakerphone that will allow authorized personnel to call from a remote location to determine any alarm conditions. A self-contained rechargeable battery power backup supply should be incorporated in the monitor. The memory containing the voice messages and telephone numbers should be sustained regardless of power, battery backup condition or transient conditions that may be experienced by the dialing alarm monitor.

Security fencing should be provided around all aboveground structures and equipment. Where security fencing is not provided, provisions should be made for locking all access hatches and securing manhole covers. Ground water monitoring well access covers should be of a type that can be secured against vandalism.

C. Construction Drawings

The construction drawings depict the dimensions, assembly and relationships of materials and equipment that comprise the OWRS facilities and provide some basic notes and instructions that are best shown on a graphic presentation of the proposed facilities. The drawings should be prepared to appropriate scales that will provide a clear representation of the work involved. In addition to the numerical scales shown, it is good practice to provide a graphic scale on each drawing to provide for situations where the drawings may be printed in reduced size for convenience in the field. All dimensions and elevations should be positioned in conspicuous locations where they will not be obscured by the line work of the drawings. All notes should be concise; where extensive written instructions are required, they should be contained in the technical specifications and the location of these instructions referenced on the drawings.

All drawings should show the date of preparation and the dates of any revisions made to the original drawings. Any revisions should be clearly flagged so as to draw attention to the revisions. A set of construction drawings for OWRS systems should include:

- A Location Plan, clearly showing where the construction project is located. This plan is often included on a title sheet drawing, if included, or on the Site Plan.

- A Site Plan, showing existing and proposed aboveground and underground facilities (including storm drainage, potable water supply, electrical and communication conduits and public utility facilities), existing and proposed contours, and the locations where subsurface investigations were made.
- A Site Layout Plan, showing the horizontal relationship between all proposed structures, piping, electrical, communication and other public utility facilities.
- A detailed Layout Plan of the subsurface wastewater absorption system (SWAS) if the layout cannot clearly be shown on the Site Plan.
- A Process Flow Schematic and Hydraulic Profile.
- Detail Sheets for all proposed structural, mechanical, electrical, communication and piping facilities.
- Elevation Views and Cross-Sections of SWAS
- Detail Sheets for SWAS facilities.
- A Floor Plan of enclosures housing proposed aboveground facilities, showing the relationship between all facilities housed in such structures or rooms.
- Elevation views and sections through enclosures housing proposed aboveground facilities.

Checking of the drawings should be done by persons who are not tasked with their preparation. Any corrections proposed by the checker should be back-checked by the person who prepared the original drawing. Care should be taken to only show dimensions and elevations once, to avoid conflicts that may occur due to changes in dimensions or elevations during the final design process. Such conflicts arise when the corrections are made on one drawing but are not carried over to other drawings that may also contain the same dimensions or elevations.

D. Technical Specifications

Technical specifications should spell out in a clear and concise manner:

- Quality assurance (QA) and quality control (QC) requirements for the work.
- Guarantees required of manufacturers and the contractors.
- Materials and products to be incorporated in the work.
- Requirements for execution of the work, including testing of the completed work.

Construction industry standards for preparation of written technical specifications are available and should be followed whenever possible. One source of such standards is the Construction Specification Institute MasterFormat™ (CSI -1995) developed jointly by CSI and Construction Specifications Canada (CSC) and used throughout North America.

Based on the results of conferences attended by representatives from all sectors of the construction industry in the early 1960s, CSI/CSC developed a 16-division format for technical specifications that has received widespread acceptance as a standard in the construction industry. The 16-division format established broad categories of construction information so that specification sections of a similar nature could be grouped together.

In the 16-division format, specifications in each division are subdivided into a number of sections, each covering one portion of the total work or requirements. CSI/CSC have published a master list of titles and numbers for construction industry technical specifications, with the latest revision being published in 1995. CSI/CSC have also published a number of “master” standard specifications, available in hard copy or electronic format, arranged for ease of editing to adapt them to the specific needs of a particular project.

In 2001, CSI/CSC undertook a complete review of their standard format and it is anticipated that a completely new MasterFormat™ was scheduled to be published in late 2004 that addresses past concerns of the construction industry as well as new technologies. The new MasterFormat will have a much larger number of divisions, with some of the new divisions addressing specific needs of civil and environmental engineers. Only those divisions and sections applicable to a project need to be included in the project specifications. Each division may include a number of individual technical specification sections.

In the CSI format for technical specifications, each specification is comprised of the following three parts:

Part I. General:	Defines the specific administrative and procedural requirements unique to each section.
Part II. Products:	Describes, in detail, the quality of items that are required for incorporation into the project under each section.
Part III. Execution:	Describes, in concise detail, preparatory actions and how the products are to be incorporated into the project.

E. Quality Assurance and Quality Control

General requirements for the contractor’s quality assurance (QA) and quality control (QC) procedures for products and workmanship are covered in Division 1, to be re-designated as Division 01 in the proposed new format. QA and QC requirements in Division 1 can be given in a single “broad scope” specification or in several “narrow scope” sections. These quality control requirements may include:

- Testing Laboratory Services
- Inspection Services
- Field Samples
- Mock-ups (usually for architectural components only)
- Contractor’s Quality Control Procedures
- Manufacturer’s Field Services

QA and QC are also addressed in the individual specification sections. An article on QA is usually provided in Part I of each specification section. Specific QC requirements for the contractor(s) are given in the various articles in Part III of each specification section, which may also include a special article on Field Quality Control.

In Part I of each technical specification section, quality assurance requirements may include such items as:

- Prerequisites, standards, limitations and criteria that establish an overall level of quality for products and workmanship.

  These should include qualifications for manufacturers, fabricators, welders, installers and applicators of products and completed works.

- Regulatory Requirements

  These should include obligations for compliance with specific code requirements and requirements of public authorities having jurisdiction.

- Certifications

  These should include requirements for submitting statements to certify compliance with certain requirements.

- Submission of Samples

- Pre-Installation Conference

  This conference should coordinate materials and techniques and sequence related work for sensitive and complex items.

Quality depends upon the design of a product, the materials used in a product, and the quality of workmanship employed in manufacturing and installing the product. Wherever possible, the specifications should refer to applicable codes and standards, and workmanship recommendations of trade associations, all of which should be clearly identified as to origin and subject matter. It is common practice to list the codes, standards, etc. under "References" in Part I of each specification section. A listing of sources of construction codes, standards and similar information is given in the CSI publication "A Directory of Construction Industry Associations, Societies, and Institutes".

For proof of quality, materials and products should be tested according to applicable standards and the manufacturer should provide certification of such tests to the contractor(s) who should forward them to the Owner's duly authorized representative for approval. Certified records of physical, chemical and other pertinent tests, and/or certified statements from the manufacturer that the materials have been manufactured and tested in conformity with the specifications can be accepted for pipe, cement, steel reinforcement, paint and similar materials that are normally tested in the shop by the manufacturer.

Where such a small quantity of material is required as to make physical tests or chemical analyses impractical, a certificate from the manufacturer stating the results of such tests or analyses on similar materials concurrently produced may be considered as the basis for the acceptance of such materials. Each manufacturer's or supplier's certificate should be endorsed or accompanied by the Contractor's certificate that the material certified by the manufacturer or supplier will be the material incorporated in the work.

Quality of workmanship includes both fabrication of the products and application or installation of the products. Examples of requirements addressing fabrication of a product are:

- The design and fabrication of the product should conform to industry standard practices for the type of product involved.
- All materials and equipment incorporated into the product should be new and of a quality conforming to industry standards for industrial material and equipment.
- The product is to be fabricated by reputable firms that are experienced in the design, production and operation of such products, and only skilled craftsmen should be employed in the fabrication processes.
- The product should be of heavy duty, industrial grade, designed for a long life of trouble free operation in the environment in which it will operate.
- All parts should be so designed and proportioned as to have liberal strength, stability and stiffness and to be especially adapted for the services they will provide.

Information on the materials and design of each product should be provided in shop drawings and technical specifications submitted to the contractor by the manufacturer of each material and product. The contractor in turn should be required to check the manufacturer's shop drawings and technical specifications to determine if they conform to the requirements of the construction contract documents before submitting them to the Applicant's Engineer for review and comment. The Engineer's comments on the manufacturer's shop drawings and technical specifications should normally be confined to determining if they are in compliance with the information given in the construction contract documents. They should not address items that are the responsibility of the contractor (e.g.: dimensions, weights, coordination of trades, or similar items).

An example of a general requirement addressing installation of manufactured items is:

- Manufactured products, materials and equipment should be applied, installed, connected, erected, used, cleaned and conditioned as directed by the manufacturer and in compliance with the construction documents.

An example of addressing the quality of workmanship with respect to installation of a specific product (in this case, pumps) is:

- The contractor must provide at least one person who should be present at all times during the installation of the pumps and who is thoroughly familiar with the pumps being installed and the manufacturer's recommended methods of installation and who should direct all of the installation work.
- The pumps should be installed, connected and tested as directed by the manufacturer and in compliance with the construction documents by experienced workers skilled in the trades required for such installation.
- All piping that is to be connected to the pumps must be thoroughly cleaned before connection.

- After satisfactory tests, pumps should be operated for a period of time sufficient to satisfy the Engineer that each complete unit has been properly installed and aligned and that it runs free from heating, rubbing or vibration.
- Correct direction of impeller rotation has been verified by visual observation.
- Starting and running amps are within the manufacturer's specifications.
- Pumps and piping are free and clear of debris and obstruction.
- The specified discharge is pumped against the specified head.
- The performance of each pump unit is entirely acceptable and meets the requirements of the specifications.

QC procedures for underground construction are particularly important. In general, QC procedures in the technical specification sections for such construction should include materials, workmanship and testing. The QC procedures should cover such items as excavation; dewatering; protection of excavations from inflow of surface water and frost; preparation of acceptable earth foundations for proposed structures, piping and conduit; bedding materials and methods of installation; and procedures for backfilling around and over the structures piping and conduit.

Installation of structures, piping and conduit or placement of fill or backfill on frozen ground should be expressly prohibited. Unless the Engineer gives written permission, work liable to be affected by frost should be suspended during freezing weather. When permission is given to work under such conditions, the Contractor should provide approved facilities for heating the materials and protecting the finished work.

Testing procedures (e.g.: pressure testing, vacuum testing) should be specified with respect to leakage of piping and structures. Procedures to be used for correction of any leakage, and for re-testing, should also be specified.

QC procedures for on-site construction of subsurface and aboveground structures should also be included in the technical specifications. Wherever possible, they should refer to industry standards.

QC procedures also extend to the delivery, unloading, and storage of materials and equipment to insure the preservation of their quality and fitness. Stored materials and equipment that will be incorporated in the OWRS facilities should be located so as to facilitate their prompt inspection. Mechanical and electrical equipment that requires servicing during long-term storage should have complete manufacturer's instructions for servicing accompanying each item, with notice of enclosed instructions shown on the exterior of each package. Materials such as PVC pipe and conduit should be protected against the damaging ultraviolet rays of the sun when stored for long periods of time. Equipment and materials that may be damaged by temperature extremes should be stored where they will not be subjected to such conditions.

F. Field Quality Control by Applicant

The Department may require that the applicant for a OWRS Discharge Permit retain a licensed professional engineer to provide construction services to verify that construction of the OWRS is done in conformance with the construction contract documents approved by the Department. The Department may also require submission of Record Drawings and supporting information upon completion of construction.

The construction services that should be provided by the Engineer include:

- Review of shop drawings and samples for conformance with the design concept and the requirements of the construction contract documents.

  The Engineer and his office staff normally provide these services.

- On-site observations of the work in progress to determine if the work is in general proceeding in accordance with the construction drawings and specifications.

  These services are normally provided by the Engineer's authorized field representative(s). It is particularly important that construction of all underground facilities, and facilities that will be hidden beneath floors, behind walls and above ceilings of aboveground structures, be observed as construction proceeds. This generally involves the full-time presence of the Engineer's field representative during such construction activities. Construction of facilities that remain visible can be observed on a periodic basis and thus may not require the full-time presence of the field representative.

The following tasks are usually assigned to the Engineer's field representative as part of his observations of the work in progress

- Report to the Engineer whenever the representative believes that any work is unsatisfactory, faulty or defective or does not conform to the construction drawings and specifications, or has been damaged, or does not meet the requirements of any inspection, test or approval required to be made; and advise the Engineer of work that the representative believes should be corrected or rejected or should be uncovered for observation, or requires special testing, inspection or approval.
- Consult with Engineer for further instructions should conditions of work or specified requirements conflict with manufacturer's instructions.
- Verify that tests, equipment and systems startups, and operating and maintenance training are conducted in the presence of appropriate personnel, and that Contractor has maintained adequate records thereof; and observing, recording and reporting to the Engineer appropriate details relative to the test procedures and startups.
- Accompany visiting inspectors representing public or other agencies having jurisdiction over the project, record the results of these inspections and report the results to the Engineer.
- Ensure that Contractor maintains accurate project record documents.
- Ensure that O&M instructions are provided to the designated OWRS facility operator by the manufacturers' authorized representative(s).

- Maintain copies of approved shop drawings, field sketches, manufacturers' instructions, etc.
- Assist the Engineer in conducting a semi-final review of the constructed project and preparing a list of items requiring completion or correction by the Contractor.
- Assist the Engineer in conducting a final review of the constructed project.

The Engineer's field representative should not authorize any deviation from the construction documents or substitution of materials or equipment, unless authorized by the Engineer. The field representative should not engage in, or assist, any construction work or other duties that are the responsibility of the Contractor, and should direct any comments, both written and verbal, regarding the quality of the work to the Contractor's on-site supervisor, rather than directly to the construction workers.

Finally, it should be understood that the Engineer has the responsibility to protect the interests of several entities. He has contractual obligations to the Applicant or Owner of the OWRS facilities and must look out for their interests. He also is responsible to the Department for seeing that the completed project conforms to the construction documents approved by the Department. Further, by virtue of his licensing as a Professional Engineer, he has a responsibility to protect the public interest.

G. References

CSI 1995 MasterFormat™. The Construction Specifications Institute. Alexandria, VA.

SECTION XIV SYSTEM OPERATION, MAINTENANCE AND MONITORING

TABLE OF CONTENTS

SECTION XIV SYSTEM OPERATION, MAINTENANCE AND MONITORING

A. Introduction

No onsite wastewater renovation system (OWRS, regardless of the type or size, will operate properly for any length of time without adequate attention being given to operation and maintenance requirements. Inattention to such requirements can soon render properly designed facilities ineffective. These facilities must be operated and maintained so as to meet the required water quality and operating standards established by the Department and any other regulatory agencies having jurisdiction. It must be clearly understood that in undertaking the construction and operation of an OWRS, the owner(s) will be firmly committed to providing sufficient funding, qualified operating personnel and management direction to ensure the satisfactory construction, operation, maintenance and monitoring of such facilities.

The full-time presence of qualified operating personnel at most OWRS facilities is usually not provided or required, and operations should be automated to the extent feasible for the type and size of facilities. However, periodic visits must be made to visually inspect and monitor performance, adjust the operation of the facility equipment and processes as conditions warrant, conduct maintenance functions, maintain performance records, and prepare and submit a periodic discharge monitoring report (DMR) to the Department and any local regulatory agencies having jurisdiction.

In addition, the facilities operator or his assistant must be available on a 24 hour a day, 7 days a week basis to respond to malfunctions of any of the pumping or pretreatment facilities. On-site laboratory facilities and a trained laboratory technician acceptable to the Department are usually not available at OWRS facilities. Therefore, it will be necessary to contract with a state approved laboratory for performing certain testing of the raw and treated wastewater that may be required by the discharge permit issued by the Department. Also, it will be necessary to contract with persons or firms qualified in the mechanical and electrical trades for providing a quick response to any equipment malfunctions that the operator is not capable of correcting.

Periodic cleaning of septic tank(s) and any grease trap(s) will be required. A licensed septage hauler must clean these tanks. Grease traps must be pumped and the contents disposed of separately from the septage pumped from septic tanks, and the contents from both grease traps and septic tanks must be properly disposed of in conformance with Department regulations. The owner of an OWRS must make arrangements with a licensed septage hauler for disposal of the material pumped from these tanks prior to placing the proposed facilities into operation.

An Operation and Maintenance (O&M) Manual for an OWRS incorporating enhanced pretreatment facilities should be submitted to the Department for approval before construction of these facilities is completed and before a permit will be issued for operation of these facilities. The Department may require the approved O& M manual to be updated and revised as necessary during the first year of operation to reflect actual operating experience. The O&M manual should be regarded as a living document and further updating may be required from time to time as experience is gained in the O&M of the facilities. In addition to operation and maintenance instructions, the manual should list the wastewater sampling and testing requirements of the Department as well as such additional sampling and testing required for effective operation and control of the treatment processes.

The manual should also contain the information listed below.

- The name and address of the engineer responsible for design of the facilities.
- The name and address of the contractor(s) who constructed the facilities.
- A list of spare parts to be maintained on site.
- A list of manufacturers' and vendors' who provided equipment and services for the facilities, with addresses, telephone numbers and e-mail addresses of persons to be contacted for assistance.
- A list of tradesmen, including names, addresses and telephone numbers, who have been retained for quick response to any equipment malfunctions that the operator is not capable of correcting.
- A list of manufacturers' O&M manuals should be provided for each piece of equipment and the location where these manuals are filed.
- A list of the construction contract documents (drawings and specifications) used to construct the facilities and the location where these documents are filed.
- A list of approved shop drawings for all of the equipment and piping installed and the location where these are filed.
- A list of Record Drawings, depicting the "as built" facilities, with particular emphasis on the locations of all underground structures, piping and electrical conduit, and the location where these are filed.

These lists should be continually and promptly updated to record any changes in addresses, phone numbers, e-mail addresses, internet addresses, document locations, etc.

A complete description of the OWRS facilities and processes should be provided. Prior to the start-up and operation of the facilities, it is important for the system operator to fully understand the manner in which the various pieces of equipment will be controlled and the functions of all electrical control panels. A discussion should be given for the step-by-step procedure used to place each piece of equipment and each treatment process into operation and for removing them from operation. (i.e. start-up and shutdown procedures) A flow chart should be included, showing each different piece of equipment and each different treatment process, along with the location of each valve on a process piping schematic. Each valve should be given a number, and the position of each valve (open, close, throttling) should be described for each different operating phase. An example of a description of a unit operation (Alarms) is given below.

Description of Alarm System

A malfunction of a system or operating equipment sounds a horn, illuminates an alarm light, and closes a dry set of contacts for the remote automatic dialing alarm monitor. The alarm horn is silenced via an alarm silence button. The alarm light and dry contact stay on until the cause of alarm is corrected and the alarm-reset button is activated. All alarm functions are latched on, however transient. The alarm system is also equipped with a TEST-OFF-AUTO switch. At no time should the alarm system be left in the "OFF" position, as this will prevent an alarm signal from reaching the automatic dialing alarm monitor.

The manual should also contain references to the OSHA Safety and Health Standards referenced in Subsection F of this Section that may be applicable to the operation and maintenance procedures discussed herein.

B. Chemical Handling, Storage, Mixing and Feeding

Instructions for receiving, storing, mixing and feeding of chemicals should be given. Particular attention should be paid to such hazardous chemicals as ethanol, sodium hydroxide, and alum. Operation and adjustment of chemical metering pumps should be described. Material Safety Data Sheets (MSDS) for all chemicals used for treatment and cleaning purposes should be included at one location in the O&M manual.

Based on the anticipated wastewater characteristics and flow rates, the concentration and volume of chemical solutions in the chemical feed tanks should be given. However, only experience with actual treatment of the wastewater will determine how often these solutions will have to be replenished in their respective feed tanks. Therefore, it will be necessary to check these tanks daily during the initial period of operation of the treatment facilities, until a replenishment schedule can be developed. Even after such a schedule is developed, it will have to be modified if changes in wastewater flow rates and characteristics occur.

C. Maintenance

A written description of the maintenance schedule for all facilities and equipment should be provided, with reference to applicable instructions in the manufacturers' O&M manuals. A written report of all maintenance performed should be kept on file in an orderly manner. Forms should be developed to simplify the preparation of such reports.

A description of all inspection and maintenance tasks should be provided. A generalized example of such descriptions is given below. Not all of the tasks described below may be required at a particular facility, while other tasks that are not described below may be required.

1. Inspect Grease Trap(s) (if provided)

Make a visual inspection of the tank(s) and internal baffles. Report any observed deterioration of concrete or inlet and outlet piping. A visual inspection of the grease trap(s) should include a depth measurement of the scum layer and any bottom sludge layer as well as noting any unusual discoloration of the scum layer. Have grease trap(s) cleaned (pumped) by a licensed septic tank cleaning firm whenever (1) the depth of the scum layer exceeds 18 inches or interferes with the operation of the outlet filter, or (2) depth of the bottom sludge layer exceeds 6 inches; or (3) a period of not more than 3 months has elapsed since the tank was last cleaned.

2. Inspect Septic Tank(s)

Make a visual inspection of the tank(s) and baffles. Report any observed deterioration of concrete or inlet and outlet piping. Record depth of sludge and scum accumulation in septic tank(s). Have tank(s)cleaned by a licensed septic tank cleaning firm whenever (1) the depth of the scum layer exceeds 18 inches or interferes with the operation of the outlet filter, or (2) the top of the sludge layer accumulated in the bottom of the tank interferes with the operation of the outlet filter, or (3) a period of 1 year has elapsed since the last time the tanks were cleaned

3. Inspect Gravity Sewers

Check pipes and manholes for solids deposition. Remove any solids accumulated in manholes. Make visual determination of manholes' structural integrity. Be alert to accumulations of grit and to groundwater leakage into the pipes and manholes. Report any accumulations of solids or groundwater leakage in pipes to management. Inspection should be conducted annually.

<u>4. Clean Force Mains</u>

Arrange to have the force mains inspected and, if necessary, cleaned at least once a year by a firm experienced in such work. The need for more frequent cleaning may become evident if the pump hour meters indicate the pumps are running for increasingly longer periods while water use at the building(s) being served has not significantly increased.

<u>5. Inspect Pump Chambers</u>

Check for correct operation of pumps and controls. Record pump running time meter readings. Test high-level alarm by raising alarm float switch.

<u>6. Service Pump Chambers</u>

Pump down liquid in pump chamber to a level no lower than one foot above the bottom, remove accumulations of solids and grease, check each pump for operation. Check all float switches for proper operation. Remove pumps from pump chamber as indicated in the maintenance schedule, clean and check physical condition of each pump and associated piping and fittings, slide rails and slide rail fittings, electrical wiring, and float switches. Clean air vent holes in pump volutes. <u>Do not remove pumps or float switches before padlocking pump disconnect switches in the "off" position!</u> Be sure to unlock pump disconnect switches and turn them to the "on" position immediately after pumps have been replaced in the pump chamber.

<u>7. Exercise Valve Chamber Valves</u>

Turn valves fully closed and then fully open to check their proper functioning. These valves open by turning to the (left)(right) direction depending upon valve furnished. (Generally, valves should open counterclockwise). Indicate the direction used to open the valves. Count and record the number of turns from fully closed to fully opened valve position. Compare number of turns to those recorded previously. If there is a difference of more than 1/2 turn, advise management that further inspection and perhaps maintenance is needed. After valves have been turned to their original position, turn the valves slightly in the opposite direction to prevent them from sticking in position. Check condition of pump check valves. Operate valve levers to ensure that valves operate freely.

<u>8. Inspect Emergency Overflow Tank</u>

Make a visual inspection of the tank. Report any observed deterioration of concrete or inlet and outlet piping. Test high-level alarm by raising alarm float switch. Check for solids deposition. Have tank cleaned of any deposited solids. Exercise any valves in tank or between tank and pump chamber in same manner as described for pump chamber valves. Tank should be maintained in an empty condition between emergency uses.

<u>9. Inspect Flow Equalization Tank</u>

Check for correct operation of pumps and controls. Record pump running time meter readings as indicated in the maintenance schedule. Test high-level alarm by raising alarm float switch. Check condition of concrete, pump slide rail assemblies, piping for signs of corrosion and advise management if re-coating, repair or replacement is required.

Remember that this tank may receive anaerobic liquid from the septic tank. Therefore, conditions in this tank are apt to be such that dangerous gases could be present. Do not enter the tank or put your head down into any openings in the tank without first determining whether such gases are present. Refer to the section on confined spaces for further information concerning entry into such spaces.

10. Service Flow Equalization Tank

Pump out flow equalization tank as indicated in the maintenance schedule, remove accumulations of solids and grease, check each pump for operation. Check all float switches for proper operation.

Remove pumps as indicated in the maintenance schedule, clean and check physical condition of each pump, check valve and associated piping and fittings, slide rails and fittings, electrical wiring, and float switches. Clean any air vent holes in the volutes. Do not remove pumps or float switches before padlocking pump disconnect switches in the "off" position! Be sure to unlock pump disconnect switches and turn them to the "on" position immediately after pumps have been replaced in the pump chamber.

11 Exercise Flow Equalization Tank Valves

Exercise pump isolation gate valves as indicated in the maintenance schedule and in the same manner as given for valve chambers.

12. Inspect Enhanced Pretreatment Facilities

Inspect, clean and maintain enhanced pretreatment facilities, if any, in accordance with manufacturers' instructions. Note any departures from normal operating condition, correct if possible, or report to management if correction cannot be made. Record all observations.

13. Inspect and Maintain Flow Metering Equipment

Inspect flow meter(s) for proper operation. Service in accordance with instructions contained in manufacturers' O&M manuals.

Replace each flow meter chart as needed, making sure that the chart is set to "real time"; that is, that the pen contacts the chart at the true time of day on the right day of the week. Write the date and time that the chart was replaced directly on the chart. If problems at the OWRS facilities are encountered, mark the date, time, totalizer reading, and initials of person making such markings, on the chart at the applicable time position of the chart.

14. Inspect Dosing Tank

Make a visual inspection of the tank. Report any observed deterioration of concrete or inlet or outlet piping. Check for correct operation of pumps and controls. Record pump running time meter readings. Check for proper operation of the float switches. Test the high level alarm by raising the alarm float switch.

15. Service Dosing Tank

Remove pumps as indicated in the maintenance schedule, clean and check physical condition of each pump and associated piping and fittings, slide rails and fittings, electrical wiring, and float switches. Clean any air vent holes in pump volutes. Do not remove pumps or float switches before padlocking pump disconnect switches in the "off" position! Be sure to unlock pump disconnect switches and turn them to the "on" position immediately after pumps have been replaced in the chamber.

16. Exercise Dosing Tank Valves

Exercise the pump isolation gate valves as indicated in the maintenance schedule and in the same manner as given for valve chambers. Check condition of pump check valves. Operate valve levers to ensure that valves operate freely.

17. Inspect the Surface Condition at Location of SWAS

Visually inspect ground surface in the area of the SWAS. Note and record any evidence of surfacing of treated wastewater. Notify management and Owner's authorized agent at once if surfacing of treated wastewater is detected at locations other than at the riprap slope at the downstream end of any lateral sand filters. Check for sinkholes at location of flow distribution manifold valves and fittings. Ensure that grassed area over the SWAS is mowed at least three times per growing season. Remove any woody vegetation growing in the grassed area.

18. Measure Depth of Ponding in Leaching systems

Measure and record the depth of water ponded in the SWAS through the inspection ports and record the time and date of measurement on the forms provided. If an unusual depth of ponding is observed, advise management and Owner's authorized agent at once.

19. Exercise SWAS Distribution Piping Isolation Valves

Exercise the valves used to isolate various portions of the SWAS as indicated in the maintenance schedule in the same manner as described for pump chambers.

20. Inspect and Service Chemical Pumping Equipment

Check and service chemical solution and slurry metering pumps and any flushing systems provided in accordance with manufacturers' instructions. Flush chemical metering pumps and piping at least once per month.

If any explosive or flammable chemicals are used, check all grounding and bonding connections between metering pumps and feed tank. Make sure they are clean and tight.

Check and service mechanical mixers in chemical mixing and feed tanks in accordance with manufacturers instructions.

21. Check Manhole Covers

Check all manhole covers to see that they are in place and properly seated. Check all inside covers on watertight manholes to see that they are properly seated and locked. Verify that the gaskets are properly in place in the watertight manhole frames. Lock all lockable outside covers.

22. Flush Piping Low Points.

Flush piping low points at least once a month to clean out solids that may have settled at low points of the piping. Open valves slowly. After flushing, close valves slowly so as to prevent water hammer. Make sure that valves are closed tightly.

23. Clean Equipment and Control Buildings

Clean all rooms of equipment and control building(s) as needed to maintain them always in a clean condition. Be careful to leave door of explosive and flammable chemical storage and feed rooms open and well ventilated when working inside and do not use tools that could strike sparks. If any items are stored in a loft above the ceiling in a building, be certain that any such items are secure and not of a weight beyond the structural capacity of the ceiling structural members.

24. Inspect Groundwater Monitoring Wells

Measure and record the depth to groundwater from the reference mark on each monitoring well and record the time and date of measurement on forms provided for that purpose. Note and record the condition of each monitoring well, i.e., has there been any damage to the well, is it unobstructed from top to bottom, etc.). It is essential that sterile conditions be maintained during all work relating to the wells, so as to prevent well contamination or cross-contamination between different wells.

25. Inspect and Maintain Emergency Electrical Generation Equipment

Inspect and maintain any emergency electrical generation equipment serving pump stations and other electrically operated facilities in accordance with manufacturers O & M instructions for such equipment. Make sure that such equipment is exercised periodically under load for at least one-half hour. Ensure that an adequate fuel supply is available.

D. Manufacturer and Vendor O & M Manuals

The previously described generalized inspection and maintenance requirements serve to highlight the key operation and maintenance needs for an OWRS employing pump chambers and enhanced pretreatment facilities. In addition to the above, O & M manuals prepared by the vendors should be provided in separate three-ring binders. The operator should be familiar with these manuals and should heed the instructions with regard to operation and maintenance of the various items of equipment.

A listing of the O & M manuals and approved shop drawings should be provided in the main O&M manual. If any information is obtained by telephone from a manufacturer's representative that contradicts the instructions of the manufacturer's O & M manual, a log entry of the precise instructions received by phone should be made, including the representative's name, phone number, date, and reason for change in instructions. A copy of this log entry should be kept in the binder containing the manufacturers’ O&M manuals and in the main O&M manual.

E. Inspection Forms

Forms should be prepared for reporting of inspections required by the Discharge Monitoring Permit for the OWRS. Copies can be used as a weekly or monthly check-off list for all of the inspections required, or similar forms can be prepared. In any case, a full log should be kept of inspections made and any actions taken as a result of such inspections. The person making such inspections should sign the inspection reports and log. An example of an inspection form is given on the following page.

F. OSHA Safety and Health Standards for the Workplace

OSHA (Occupational Safety and Health Administration) is charged with protecting the safety and health of the nation’s workers. Accordingly, it has established standards and periodically inspects workplaces to assure that the requirements of those standards are being met. These standards are contained in Title 29 CFR (Code of Federal Regulations), Part 1910, and the subpart that contains requirements for confined spaces referred to on page 9 is just one of a number of subparts of Part 1910 that pertain to the construction, operation and management of wastewater collection and treatment facilities. The various precautions and procedures discussed in this Section XIV are not all inclusive of the various OSHA requirements. It is the responsibility of the designer, owner and operator(s) of on-site wastewater renovation systems and associated wastewater collection systems to determine and comply with the requirements of these standards.

Example of Inspection Report

Grease Trap #1:

	Inlet	Outlet		
Depth of Scum (inches)			Need Pumping y/n	
Depth of Sludge (inches)			Covers Secured y/n	
			Date Last Pumped	

Grease Trap #2:

	Inlet	Outlet		
Depth of Scum (inches)			Need Pumping y/n	
Depth of Sludge (inches)			Covers Secured y/n	
			Date Last Pumped	

Septic Tank #1:

	Inlet	Outlet		
Depth of Scum (inches)			Need Pumping y/n	
Depth of Sludge (inches)			Covers Secured y/n	
Tees Clear y/n			Baffles ok y/n	

Septic Tank #2:

	Inlet	Outlet		
Depth of Scum (inches)			Need Pumping y/n	
Depth of Sludge (inches)			Covers Secured y/n	
Tees Clear y/n			Baffles ok y/n	
			Date Last Pumped	

Raw Wastewater Pump Station:

Grease Shelf on Walls y/n		Grease Build Up on Floats y/n	
Scum (inches)		Chamber Need Pumping y/n	
Floats Need Cleaning y/n N		Date Chamber last pumped	

Inspect pumps for operation:

	Automatic	Manual		
Pump #1 (ok or comment)			Liquid Level Drop y/n	
Pump #2 (ok or comment)			Liquid Level Drop y/n	

Inspect Float Switches For Operation:

Lead Float y/n		Lag Float y/n		Pump Off Float y/n	
				Confirm High Level Alarm Float Signal is received	
Disconnects Secured y/n		Hatch Cover Secured y/n			

Operator's Signature: ____ Date: ________________

G. Confined Spaces

One of the OSHA Standards (Sub-part J-1910.146) pertains to permits required for entry into confined spaces. Confined spaces at wastewater facilities include, but are not limited to, all manholes, underground tanks and pumping chambers and above ground structures with limited access. Entry into confined spaces must comply with OSHA practices and procedures.

Air normally contains about 21% oxygen by volume; the minimum concentration for safe entry in a confined (enclosed) space is 19.5% oxygen. The presence of sewage in a confined space can result in the formation of "sewer gas" and the presence of this gas can cause an oxygen deficiency. It is important to note that many of the deaths that occur in working around sewers and pump chambers, classed as asphyxiation, are traceable to a deficiency in oxygen. Sewer gas may also contain hydrogen sulfide, a toxic gas. This gas at very low concentrations can cause death in a very few minutes.

H. Safety Considerations

A section should be provided in the O & M manual that discusses steps to be taken to help prevent injury to treatment plant operating and maintenance personnel. Hazards discussed in that section should encompass various broad categories, such as:

- Gas related hazards (toxic, explosive, or oxygen deficiency).
- Entering tanks.
- Physical injuries (from mechanical or electrical sources).
- Bacterial infections.

Emergency phone numbers should be listed (and also posted at telephones). These might include phone numbers for fire departments, ambulance, hospitals, medical caregivers, and State Police.

Some of the items to be discussed under safety considerations might include:

- A general discussion on prevention of physical injuries
- Oxygen Deficiency & Toxic Gases In Confined Spaces
- Explosion & Fire Hazards
- Electrical Hazards
- Mechanical Equipment Hazards
- Bacterial Infection (Health Hazards)
- Suggested Safety Equipment

Examples of such discussions are given below.

1. Prevention of Physical Injuries

The prevention of physical injuries begins with good housekeeping. The facilities, including the grounds, must be kept in good repair and maintained in an orderly manner. Tools must be picked up, manhole and valve box covers promptly replaced, hatchway doors and removable grates kept closed, or properly barricaded when open. Walkways must be kept free of grease, oil, ice or other slippery material. Defective ladders should always be immediately destroyed. Loose or broken ladder rungs must be promptly and adequately repaired or replaced.

Temporary chains, cables, hoists, and ropes should never be left hanging after use, but should be removed, cleaned and properly stored. Hoses should be on reels, hangars, or neatly coiled when not in use. Grease, oil, sludge, scum and other spills should be promptly cleaned up. Reinstallation of all belt, gear or chain guards, etc. should be made when work is completed. The person working on a particular piece of equipment is responsible for such reinstallation.

The safety precautions for oxygen deficiency and toxic gases are similar, as follows:

- Do not enter an enclosed hazardous (confined) space, or allow any contractor to enter a confined space without authorization and without persons entering such spaces having received certified special training and being properly equipped.
- Do not enter an enclosed, poorly ventilated space without testing with portable gas detectors (oxygen, combustible gas, and hydrogen sulfide).
- Use a portable blower to supply ventilation in a suspect area. Blow fresh air into a confined space to provide a safe environment. Do NOT use a blower to exhaust vapors from a confined space unless the blower is completely explosion proof.
- If a hazardous area must be entered because of an emergency or other reason, use a suitable gas mask or air pack.
- When entering a tank or other limited access area, wear a safety harness and have an attendant "on watch" outside the tank.

More detailed information with regard to toxic gases and explosion hazards are presented below.

2. Explosion & Fire Hazards

As previously discussed above, whenever wastewater is present in a poorly ventilated enclosed space (such as a manhole, pump chamber or tank), sewer gas can form. Because of the presence of methane and hydrogen, sewer gas can be explosive. These gases will have a tendency to collect in manholes, sewers, pump chambers, and tanks.

Before entering any enclosed space that contains wastewater, the space should be checked for explosive gases with a portable combustible gas detector (see Suggested Safety Equipment). A portable blower should be used to direct fresh air into the enclosed space to clear out any explosive gases.

Explosive liquids or vapors include gasoline, cleaning materials, and paints and oils. These liquids should be stored in areas specially designed for their storage. Preventive measures that should be followed include:

- Practice good housekeeping.
- Vacuum-clean dust accumulations.
- Forbid smoking and restrict open flames anywhere inside the treatment facilities.

In the event an explosion does occur, it may be accompanied by fire, power outage, or flooding. The following steps should be followed in the event of an explosion:

- Evacuate personnel.
- Notify the fire department, police department, and the Department of Environmental Protection.

- Isolate the affected area by shutting down all electrical equipment and stopping or diverting wastewater flow into the area.
- When it is safe to enter the affected area, assess the damage to the system and determine what has to be done to restore service.

A fire prevention program is recommended. Good housekeeping is an essential part of the program. The removal of oily rags, paper, and other combustible materials will help minimize the chance of a fire by removing the source of fuel. The fire prevention program should emphasize the following items:

- Maintain good housekeeping practices.
- Know the location of all the fire extinguishers, which ones to use for each type of fire, and how to use them.
- Recognize and remove potential fire hazards.
- Be careful in the use of open flames (welder's torch) and volatile materials (paints or solvents).
- Provide proper preventive maintenance to minimize equipment related fires.

In case of a fire, the following procedures should be followed:

- Evacuate personnel.
- Call the fire department, police department and the Department of Environmental Protection.
- Consider the feasibility to combat minor fires with available equipment and personnel.

Recommendations for the safe handling of explosive and flammable liquids should be provided. Some items that should be discussed include:

- Effective employee education on safe handling of explosive and flammable liquids should be provided, and this should be accompanied by adequate supervision.
- Excessive or prolonged breathing of vapors should be avoided.
- All spills should be flushed with water promptly.
- Protection from spark ignition due to static electricity or stray currents during unloading or transfer operations should be effected by grounding and bonding of equipment and piping from the container being unloaded to the container being filled. <u>This should be done before the containers are opened!</u> <u>Similarly, steam or air hoses should be bonded to the tank prior to a purging operation.</u>
- All tools used around open containers should be spark resistant.
- Since grounding does not remove the possibility that sparking can occur on the liquid surface in the container being filled, loading lines or spouts should be extended to the bottom of this container to minimize splash and spray and thus reduce generation of static electricity.
- Tanks, equipment, piping, etc. should be drained and thoroughly cleaned with water and/or steam before being repaired.
- Waste mixtures containing flammable amounts of chemicals should not be permitted to enter drains or sewers where there might be danger of ignition.
- Under emergency conditions, (fire, exposure to hazardous liquid or vapor, spillage), follow instructions given in the MSDS for such liquids.

3. Electrical Hazards

Electrical shock hazards are similar to those that would exist in most modern plants or factories. Examples of safety measures that must be followed are as follows:

- Maintenance or repair work on electrical equipment, switch gear, motors, etc. should only be done by qualified personnel.
- When working on electrical equipment, the main power switch used to disconnect power from the equipment should be locked open and tagged. It is not sufficient to just turn off a manual or automatic control switch.
- Grounding of all equipment is essential. Portable power tools should be equipped with ground wire and special outlet and plug.
- Water and electricity do not "mix". Be sure hands and shoes are dry. A rubber mat should be used where appropriate, particularly around electrical control panels and switchgear.

Other examples of unsafe electrical conditions and corrective measures are listed in the following Table.

EXAMPLE OF UNSAFE ELECTRICAL CONDITIONS

Unsafe Condition	Corrective Measures
Worn insulation on extension and drop cords	Use Underwriter's Laboratories approved materials only. Spliced or worn cords should be removed from service
Unsafe wiring practices, such as using wires too small for the current being carried; open wiring not in conduit; temporary wiring; wiring improperly located.	Comply with recognized electrical code. Remove temporary wiring as soon as it has served its purpose.
Working on "live" low-voltage circuits in the belief that they are not hazardous.	Educate and train personnel in the hazards of low-voltage circuits.
Working on "live" circuits which are thought to be "dead".	Require that main power disconnect switches on all circuits being worked on be locked open and properly tagged. Use protective equipment such as rubber gloves and rubber floor mats.
Replacing fuses by hand on "live" circuits.	Open switch before replacing fuses; use fuse pullers.
Exposed conductors at rear of switch-board.	Enclose rear of switchboard to prevent exposure of unauthorized persons. Provide rubber mats for workers who must enter enclosure.

4. Mechanical Equipment Hazards

Here again, hazards are similar to those that would exist in most modern plants or factories. Examples of safety measures that should be discussed are as follows:

- Maintenance or repair work on rotating machinery or other mechanical equipment should only be done by qualified personnel.
- Belt guards, chain guards, railings, etc. should not be removed except when required for repair or maintenance. Guards, railings, etc. should be replaced as soon as work is completed.
- Switch gear should be locked open and tagged if an electrical drive is involved.
- A safety harness should be worn when working next to an area where a fall is possible.
- Loose clothing should not be worn when working near rotating machinery.
- Proper lifting equipment should be used if heavy objects must be lifted.

5. Pathogen Infection (Health Hazards)

Workers who come into contact with wastewater are exposed to all the potential hazards of water-borne diseases, including typhoid fever, amoebic dysentery, infectious jaundice, hepatitis, and other intestinal infections. Tetanus and skin infections must also be guarded against.

Except for minor injuries, a doctor should treat wounds. No cut or scratch is too minor to receive attention. A disinfectant should be immediately applied to all wounds or cuts.

Rubberized cotton gloves are inexpensive and afford good protection to the hands. In wet places, boots or rubber overshoes protect the feet from dampness and infection. Work clothes or coveralls should be worn in dirty places such as manholes, and should be laundered frequently. For extremely dirty jobs, such as cleaning out a pump chamber, there are rubberized fabric suits with hoods available that can be washed off with a hose. Wear safety goggles to protect against liquids and dust that may harbor pathogens.

Smoking should not be done in or within close proximity to the wastewater collection and pretreatment facilities. It is practically impossible to avoid contamination by wastewater to the ends of pipes, cigars or cigarettes. Smoking is also a potential source of ignition for any flammable vapor present.

"Keeping the hands below one's collar," while at work in tanks, pump chambers or while handling wastewater or sludge is an excellent rule. A majority of infections reach the body by way of the mouth, nose, eyes or ears. Hands should be washed before smoking or eating. Typhoid and hepatitis inoculations are strongly recommended.

6. Suggested Safety Equipment

Consideration should be given to requiring such safety equipment as:

- Gas mask (preferably an air pack).
- Portable fire extinguishers (CO_2-type).
- Face shields and safety goggles.
- Emergency Eye Wash Station
- First aid kit.
- Gas detectors (one each for oxygen, combustible gas, and hydrogen sulfide).
- Safety harness.

- Portable blower and flexible duct.
- Explosion-proof lanterns.
- Heavy neoprene gloves, suitable for working with chemicals.
- Disposal latex gloves.
- Heavy-duty aprons suitable for working with chemicals.

The operator must be familiar with the location and care of all safety equipment and trained in its use.

I. Testing for Monitoring and Control of Treatment Processes.

The periodic testing of influent and effluent wastewater quality required for submission of a Discharge Monitoring Report (DMR) to the Department is normally not sufficient for control of treatment processes because of the long periods of time between such testing.

Outside laboratory testing for various operating parameters is also often insufficient because of the delay (often many days) between obtaining the samples and receiving the test results.

The quickest onsite tests for process control are sensory in nature. Visual and olfactory observations of the effluent from enhanced pretreatment facilities and the operation of process equipment by a trained operator are often all that is needed for a first appraisal of the process operating status and efficiency. If the process equipment is observed and heard to be operating correctly with no unusual noises or vibrations, and the effluent looks sparkling clear and has no appreciable odor, the chances are that the treatment processes are stable and providing a satisfactory effluent. If such observations indicate problems, then additional testing is required.

There are several simple and effective on-site tests that can be conducted in a relatively short time, often no more than 1-2 hours per plant visit, if the necessary equipment and supplies are available. These include:

- Selective Ion Meter, equipped with pH, liquid temperature, dissolved oxygen (D.O.) and ammonia-nitrogen test probes.
- Prepackaged chemical test supplies and associated colorimeters for nitrates, alkalinity, and phosphorus.
- Turbidimeter, for determining clarity of effluent (which can also be calibrated against suspended solids tests so as to provide a surrogate method for rapid determination of the TSS in the plant effluent).
- Settleometer, for Settled Sludge Volume (SSV) tests.*
- High Speed Centrifuge equipped with centrifuge tubes, graduated in % concentration or milliliters, that swing out horizontally while the centrifuge is spinning.*
- Timer 0- 60 minute period.*
- Sludge blanket finder (e.g. Sludge Judge).*
- Secchi Disc.*
- Containers required for the tests indicated above.

* Only needed for suspended growth processes.

The equipment for running these tests can presently be obtained at a total cost on the order of several thousands of dollars and the cost of supplies required will usually be on the order of hundreds of dollars per year. The benefit gained from the test results will usually be greatly in excess of the costs and time involved. Where suspended growth processes are involved, simple methods for evaluation and control are given in Hobson (1993).

J. Facilities Operator

The facilities owner should retain an operator who will be in responsible charge for operation and maintenance of the facility. The certification and classification of the operator and the classification of the facility should conform to the requirements set forth in Section 22a-416 of the Connecticut General Statutes (“Regulations of Department of Environmental Protection concerning Wastewater Facility Operator Certification”).

K. References

Hobson, T. 1993. Activated Sludge. Evaluation & Controlling Your Process. Hobson’s Choice Press. Salina, KA.

GUIDANCE FOR DESIGN
OF
LARGE-SCALE
ON-SITE WASTEWATER RENOVATION SYSTEMS

TABLE OF APPENDICES

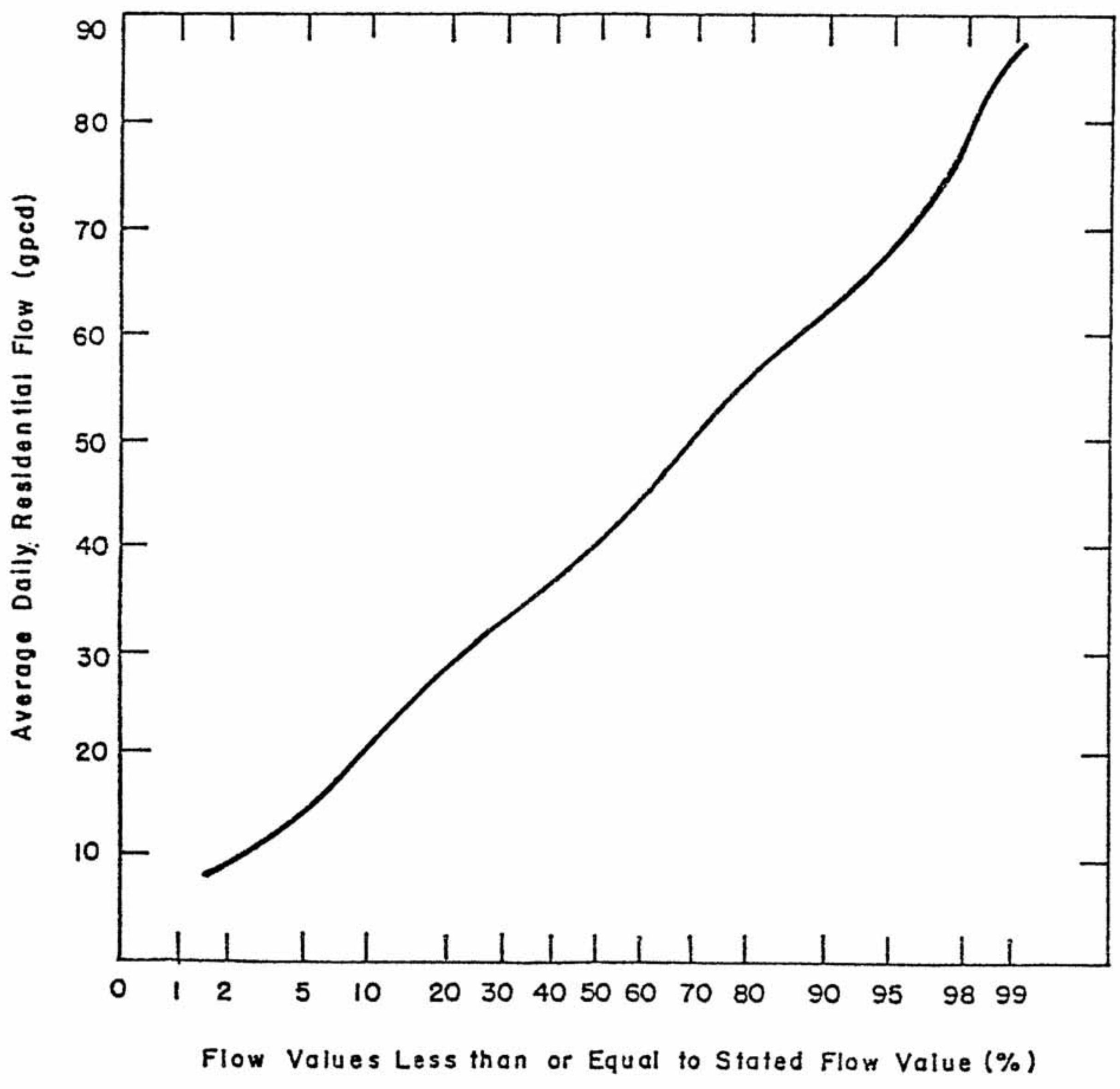
APPENDIX A

WASTEWATER FLOW DATA

From

SEEPAGE AND POLLUTANT RENOVATION
ANALYSIS FOR LAND TREATMENT, SEWAGE DISPOSAL SYSTEMS.
CT. DEPT. OF ENVIRONMENTAL PROTECTION
Healy, K.A. and R. May -1982

Frequency Distribution for Average Daily
Residential Water Use/Waste Flows



From: Healy, K.A. and R. May. 1982. Seepage and Pollutant Renovation - Analysis For Land Treatment, Sewage Disposal Systems. CT. Dept. of Environmental Protection

TABLE 5

Summary of Average Daily Residential Wastewater Flows

Study	No. of Residences	Duration of Study months	Wastewater Flow: Study Average gpcd	Wastewater Flow: Range of Individual Residence Averages gpcd
Linaweaver, et al.	22	-	49	36 - 66
Anderson and Watson	18	4	44	18 - 69
Watson, et al.	3	2-12	53	25 - 65
Cohen and Wallman	8	6	52	37.8 - 101.6
Laak	5	24	41.4	26.3 - 65.4
Bennett and Linstedt	5	0.5	44.5	31.8 - 82.5
Siegrist, et al.	11	1	42.6	25.4 - 56.9
Otis	21	12	36	8 - 71
Duffy	16	12	42.3	-
		Weighted Average	44	

From: Healy, K.A. and R. May 1982. Seepage and Pollutant Renovation - Analysis for Land Treatment, Sewage Disposal System. Connecticut Department of Environmental Protection.

TABLE 7

Residential Water Use by Activity[a]

Activity	Gal/use	Uses/cap/day	gpcd[b]
Toilet Flush	4.3 4.0 - 5.0	3.5 2.3 - 4.1	16.2 9.2 - 20.0
Bathing	24.5 21.4 - 27.2	0.43 0.32 - 0.50	9.2 6.3 - 12.5
Clothes washing	37.4 33.5 - 40.0	.29 0.25 - 0.31	10.0 7.4 - 11.6
Dish washing	8.8 7.0 - 12.5	0.35 0.15 - 0.50	3.2 1.1 - 4.9
Garbage Grinding	2.0 2.0 - 2.1	0.58 0.4 - 0.75	1.2 0.8 - 1.5
Miscellaneous	-	-	6.6 5.7 - 8.0
Total	-	-	45.6 41.4 - 52.0

[a] Mean and ranges
[b] gpcd may not equal gal/use multiplied by uses/cap/day due to difference in the number of study averages used to compute the mean and ranges shown.

From: Healy, K.A. and R. May 1982. Seepage and Pollutant Renovation - Analysis for Land Treatment, Sewage Disposal System. Connecticut Department of Environmental Protection.

TABLE 8

Typical Wastewater Flows from Commercial Sources

Source	Unit	Wastewater Flow Range	Wastewater Flow Typical
		gpd/unit	
Airport	Passenger	2.1 - 4.0	2.6
Automobile Service Station	Vehicle Served	7.9 - 13.2	10.6
	Employee	9.2 - 15.8	13.2
Bar	Customer	1.3 - 5.3	2.1
	Employee	10.6 - 15.8	13.2
Hotel	Guest	39.6 - 58.0	50.1
	Employee	7.9 - 13.2	10.6
Industrial Building (excluding industry and cafeteria)	Employee	7.9 - 17.2	14.5
Laundry (Self Service)	Machine	475 - 686	580
	Wash	47.5 -52.8	50.1
Motel	Person	23.8 - 39.6	31.7
Motel with Kitchen	Person	50.2 - 58.1	52.8
Office	Employee	7.9 - 17.2	14.5
Restaurant	Meal	2.1 - 4.0	2.6
Rooming House	Resident	23.8 - 50.1	39.6
Store, Department	Toilet Room	423 - 634	528
	Employee	7.9 - 13.2	10.6
Shopping Center	Parking Space	0.5 - 2.1	1.1
	Employee	7.9 - 13.2	10.6

From: Healy, K.A. and R. May 1982. Seepage and Pollutant Renovation - Analysis for Land Treatment, Sewage Disposal System. Connecticut Department of Environmental Protection.

TABLE 9

Typical Wastewater Flows from Institutional Sources

Source	Unit	Wastewater Flow Range	Typical
		gpd/unit	
Hospital, Medical	Bed	132.0 - 251.0	172.0
	Employee	5.3 - 15.9	10.6
Hospital, Mental	Bed	79.4 - 172.0	106.0
	Employee	5.3 - 15.9	10.6
Prison	Inmate	79.3 - 159.0	119.0
	Employee	5.3 - 15.9	10.6
Rest Home	Resident	52.8 - 119.0	92.5
	Employee	5.3 - 15.9	10.6
School, Day:			
With Cafeteria, Gym, Showers	Student	15.9 - 30.4	21.1
With Cafeteria Only	Student	10.6 - 21.1	15.9
Without Cafeteria, Gym, Showers	Student	5.3 - 17.2	10.6
School, Boarding	Student	52.8 - 106.0	74.0

From: Healy, K.A. and R. May 1982. Seepage and Pollutant Renovation - Analysis for Land Treatment, Sewage Disposal System. Connecticut Department of Environmental Protection.

TABLE 10

Typical Waste Water Flows from Recreational Sources

Source	Unit	Wastewater Flow Range	Wastewater Flow Typical
		gpd/unit	
Apartment, Resort	Person	52.8	58.1
Cabin, Resort	Person	34.3 - 50.2	42.3
Cafeteria	Customer	1.1 - 2.6	1.6
	Employee	7.9 - 13.2	10.6
Campground	Person	21.1 - 39.6	31.7
Cocktail Lounge	Seat	13.2 - 26.4	19.8
Coffee Shop	Customer	4.0 - 7.9	5.3
	Employee	7.9 - 13.2	10.6
Country Club	Member Present	66.0 - 132.0	106.0
	Employee	10.6 - 15.9	13.2
Day Camp (no meals)	Person	10.6 - 15.9	13.2
Dining Hall	Meal Served	4.0 - 13.2	7.9
Dormitory, Bunkhouse	Person	19.8 - 46.2	39.6
Hotel, resort	Person	39.6 - 63.4	52.8
Laundromat	Machine	476.0 - 687.0	581.0
Store Resort	Customer	1.3 - 5.3	2.6
	Employee	7.9 - 13.2	10.6
Swimming Pool	Customer	5.3 - 13.2	10.6
	Employee	7.9 - 13.2	10.6
Theater	Seat	2.6 - 4.0	2.6
Visitor Center	Visitor	4.0 - 7.9	5.3

From: Healy, K.A. and R. May 1982. Seepage and Pollutant Renovation - Analysis for Land Treatment, Sewage Disposal System. Connecticut Department of Environmental Protection.

APPENDIX B

SOIL TEXTURAL CLASSIFICATION SYSTEM

U.S. Department of Agriculture
Natural Resources Conservation Service

APPENDIX B

U.S.D.A. SOIL TEXTURAL CLASSIFICATION SYSTEM

(1) Soil texture is a term commonly used to designate the proportionate distribution of different sized mineral particles in a soil material. The three basic sizes of soil mineral particles are the sand size, the silt size and the clay size. The sand size class is subdivided further into the subclasses of very coarse sand, coarse sand, medium sand, fine sand, and very fine sand. Individual particles, based on their size, are grouped into separates. These soil separates are classified by size into the groupings shown below:

Soil Separates	Diameter Limit in Millimeters:
Very coarse sand	2.00 - 1.00
Coarse sand	1.00 - 0.50
Medium sand	0.50 - 0.25
Fine sand	0.25 - 0.10
Very fine sand	0.10 - 0.05
Silt	0.05 - 0.002
Clay less than	0.002

(2) Major soil texture classifications and some of the characteristics which can be utilized in the field for identification of these soil texture groups is accomplished primarily by rubbing moist samples of soil material between the fingers and observing how the material feels.

(a) Sand - Sand feels extremely gritty and does not form a ribbon or ball when wet or moist. A sand is loose and single grained. The individual grains can readily be seen or felt.

(b) Loamy sand - Loamy sand feels extremely gritty and forms a weak ball that cannot be handled without breaking.

(c) Sandy loam - A sandy loam feels extremely gritty and slightly sticky. When moist, it forms a cast that will bear careful handling without breaking.

(d) Loam - A loam feels somewhat gritty, yet fairly smooth and slightly plastic. When moist, it forms a cast that may be handled quite freely without breaking. Loam forms only short ribbons about 0.25 inch to 0.50 inches in length.

(e) Silt loam - Silt loam lacks grittiness and feels extremely floury when moist or dry. When dry it may appear cloddy but the lumps can be readily broken. When moist it will form casts that can be freely handled without breaking. It will not form a ribbon but will give a broken appearance.

(f) Silt - Silt lacks grittiness and feels extremely floury when moist or dry. It will not ribbon and forms a weak ball that will tolerate careful handling without breaking.

(g) Sandy clay loam - sandy clay loam feels very gritty and sticky. When moist it forms a firm ball and may form a ribbon of one to two inches before it breaks.

(h) Clay loam - A clay loam feels very sticky with little or no grittiness. When moist it will form a ribbon that is about one to two inches long. The moist soil is plastic and will form a cast or ball that will bear much handling. When kneaded in the hand it does not crumble readily but tends to work into a heavy compact mass.

(i) Sandy clay - Sandy clay feels extremely sticky and very gritty. When moist and forms a firm ball and produces a ribbon that is over two inches in length before breaking.

(j) Silty clay - Silty clay feels both plastic and extremely sticky when moist and lacks any gritty feeling. It forms a firm ball and readily ribbons to over two inches in length before it breaks.

(k) Clay - A clay feels extremely sticky and is neither gritty nor floury. When moist it forms a ribbon over two inches in length before breaking. It will form a hard ball or cast which will not break when handled.

(l) Organic soils - Muck peat, and mucky peat are used in place of textural class names in organic soils. Muck is well-decomposed organic soil material; peat consists of raw un-decomposed organic soil material; and mucky peat designates materials intermediate in decomposition between muck and peat.

(3) Definitions of the soil texture classes according to distribution of size classes of mineral particles ≤ 2 millimeters in diameter are as follows[1]:

(a) Sands - 85 percent or more sand and the percentage of silt plus 1 1/2 times the percentage of clay is 15 or less.

1. Coarse sand - 25 percent or more very coarse and coarse sand and less than 50 percent any other single grade of sand.

2. Sand - 25 percent or more very coarse, coarse and medium sand, but less than 25 percent very coarse and coarse sand, and less than 50 percent either fine sand or very fine sand.

[1] The U.S. Department of Agriculture, Natural Resources Conservation Service "Soil Texture Triangle" shown on the following page provide a graphical means of classifying the fine soil (≤ 2 mm).fractions.

3. Fine sand - 50 percent or more fine sand; or less than 25 percent very coarse, coarse, and medium sand and less than 50 percent very fine sand.

4. Very fine sand - 50 percent or more very fine sand.

(b) Loamy sands - At the upper limit 85 to 90 percent sand and the percentage of silt plus 1 1/2 times the percentage of clay is 15 or more; at the lower limit 70 to 85 percent sand and the percentage of silt plus twice the percentage of clay is 30 or less.

1. Loamy coarse sand - 25 percent or more very coarse and coarse sand and less than 50 percent any other single grade of sand.

2. Loamy sand - 25 percent or more very coarse, coarse, and medium sand and less than 50 percent either fine sand or very fine sand.

3. Loamy fine sand - 50 percent or more fine sand; or less than 50 percent very fine sand and less than 25 percent very coarse, coarse, and medium sand.

4. Loamy very fine sand - 50 percent or more very fine sand.

(c) Sandy loams - 20 percent or less clay and 52 percent or more sand and the percentage of silt plus twice the percentage of clay exceeds 30; or less than 7 percent clay, less than 50 percent silt, and between 43 and 52 percent sand.

1. Coarse sandy loam - 25 percent or more very coarse and coarse sand and less than 50 percent any other single grade of sand.

2. Sandy loam - 30 percent or more very coarse, coarse, and medium sand, but less than 25 percent very coarse and coarse sand, and less than 30 percent either fine sand or very fine sand.

3. Fine sandy loam - 30 percent or more fine sand and less than 30 percent very fine sand; or between 15 and 30 percent very coarse, coarse, and medium sand; or more than 40 percent fine and very fine sand, at least half of which is fine sand, and less than 15 percent very coarse, coarse, and medium sand.

4. Very fine sandy loam - 30 percent or more very fine sand; or more than 40 percent fine and very fine sand, at least half of which is very fine sand, and less than 15 percent very coarse, coarse, and medium sand.

(d) Loam - 7 to 27 percent clay, 28 to 50 percent silt, and less than 52 percent sand.

(e) Silt loam - 50 percent or more silt and 12 to 27 percent clay; or 50 to 80 percent silt and less than 12 percent clay.

(f) Silt - 80 percent or more silt and less than 12 percent clay.

(g) Sandy clay loam - 20 to 35 percent clay, less than 28 percent silt, and 45 percent or more sand.

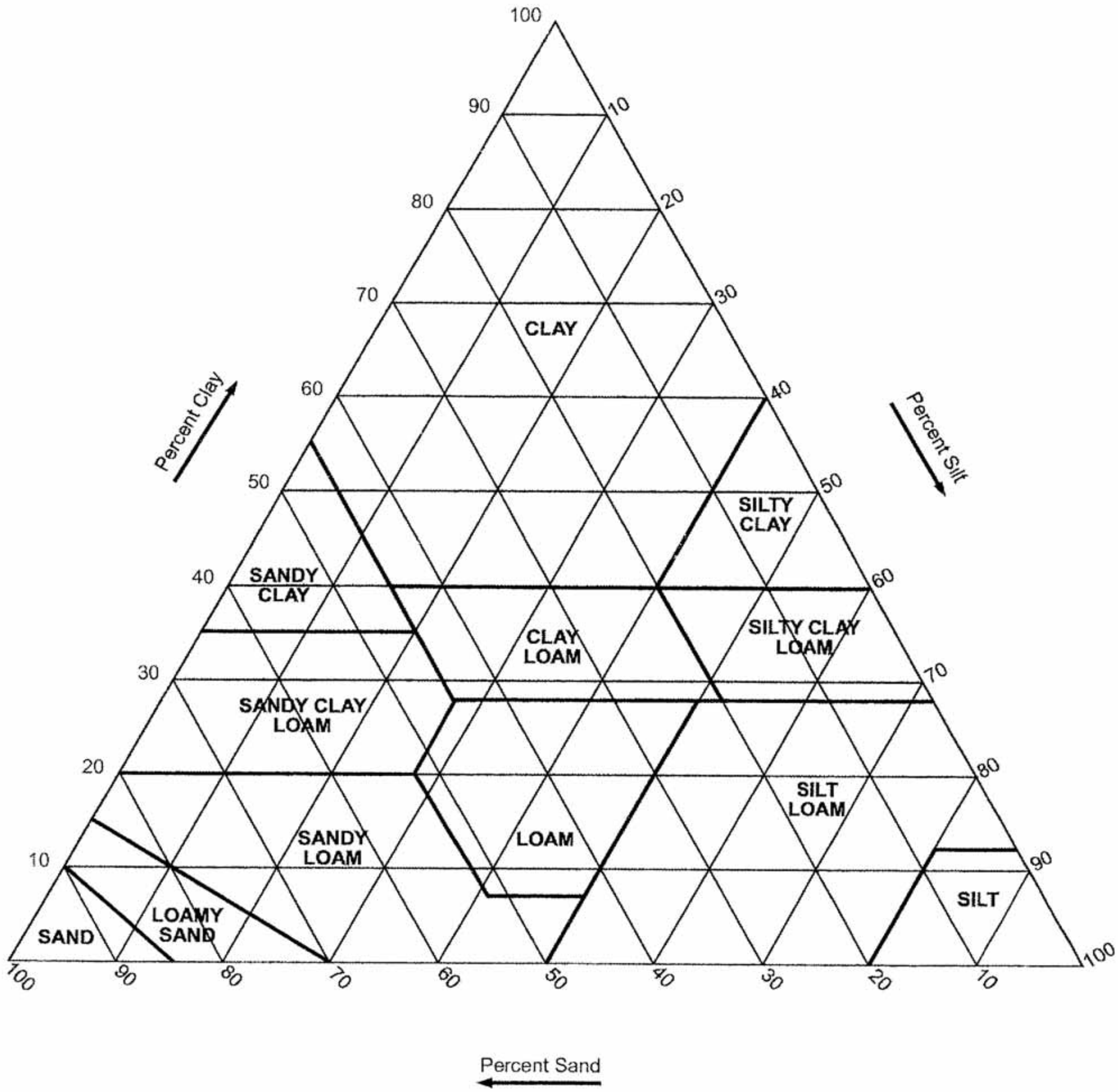
(h) Clay loam - 27 to 40 percent clay and 20 to 45 percent sand.

(i) Silty clay loam - 27 to 40 percent clay and less than 20 percent sand.

(j) Sandy clay - 35 percent or more clay and 45 percent or more sand.

(k) Silty clay - 40 percent or more clay and 40 percent or more silt.

(l) Clay - 40 percent or more clay, less than 45 percent sand, and less than 40 percent silt.



TEXTURE TRIANGLE

(Fine Earth Texture Classes)

From: U.S. Department of Agriculture, Natural Resources Conservation Service, National Soil Survey Center Field Book for Describing and Sampling Soils, Version 1.1 5/13/98

APPENDIX C

SELECTING HYDRAULIC CONDUCTIVITY VALUES FOR DESIGN

APPENDIX C

SELECTING HYDRAULIC CONDUCTIVITY VALUES FOR DESIGN

As stated in Section VI, estimating hydraulic conductivity (K) is a difficult task involving testing and measurement, tempered by reasonable judgement. When performing a hydraulic capacity analysis, or selecting a value for long term acceptance rate (LTAR), a value in the lower 50 percentile of the range of K values should be used and certainly should not exceed the geometric mean hydraulic conductivity (K_g). On the other hand, when computing travel time, a conservative value from the high end of the range of K values should be used.

It is quite likely that the K values, obtained as discussed in Section VI, will differ from the actual K values of the in-situ soils because the K values obtained actually represent a relatively few localized samples in a universe of samples needed to represent the entire soil deposit. For example, the mean value of a set of K sample data points is not equivalent to the mean K of the entire soil deposit under investigation, and it is virtually impossible to physically determine the true mean K value. However, statisticians have developed methods for determining how close the mean of a set of samples approaches the true mean. One such method, as presented in Lin (1986), was used to prepare Figure QC-1. This Figure provides an estimate of the number of samples required in order to have 90% confidence that the calculated mean is within 10% of the true mean. It requires knowledge of the coefficient of variation (C_v) of the sample set. Where C_v is unknown, a reasonable initial value of C_v can be made from results obtained from initial laboratory testing of a relatively small number of samples. Once the initial value of C_v is determined, the number of samples required to provide the 90% confidence level can be obtained from Figure QC-1. The number of samples required for any further testing can then be based on the C_v determined from the preceding sample set.

It should be remembered that errors caused by using values of K that differ significantly from actual in-situ values could be very difficult and costly to remedy once the SWAS is constructed and placed into operation. Therefore, consideration should be given to applying reasonable safety factors to the values of K determined by localized field tests or laboratory tests of soil samples. The magnitude of the safety factor selected should be based on the type of SWAS to be used.

Cedergren (1989) indicates that the K value defined by Darcy's Law is a statistical average factor that represents a definite cross-section of soil. When determined by the careful experimental testing of a given mass of soil, it is representative of the volume of soil tested. To be useful in practical problems it is usually necessary to estimate the K of extensive soil deposits from tests on limited volumes of soil. As a rule, the larger the mass represented by a test, the more reliable the results; however, tests on comparatively minute specimens can be extremely valuable when applied carefully and with the proper regard for limitations inherent to all such determinations. Cedergren (ibid.) further states that " A sound approach to every seepage and drainage situation requires the greatest possible care in assigning permeabilities [hydraulic conductivities] and other physical data as realistically as possible and the use of reasonable care in carrying out the appropriate theoretical or mathematical solution."

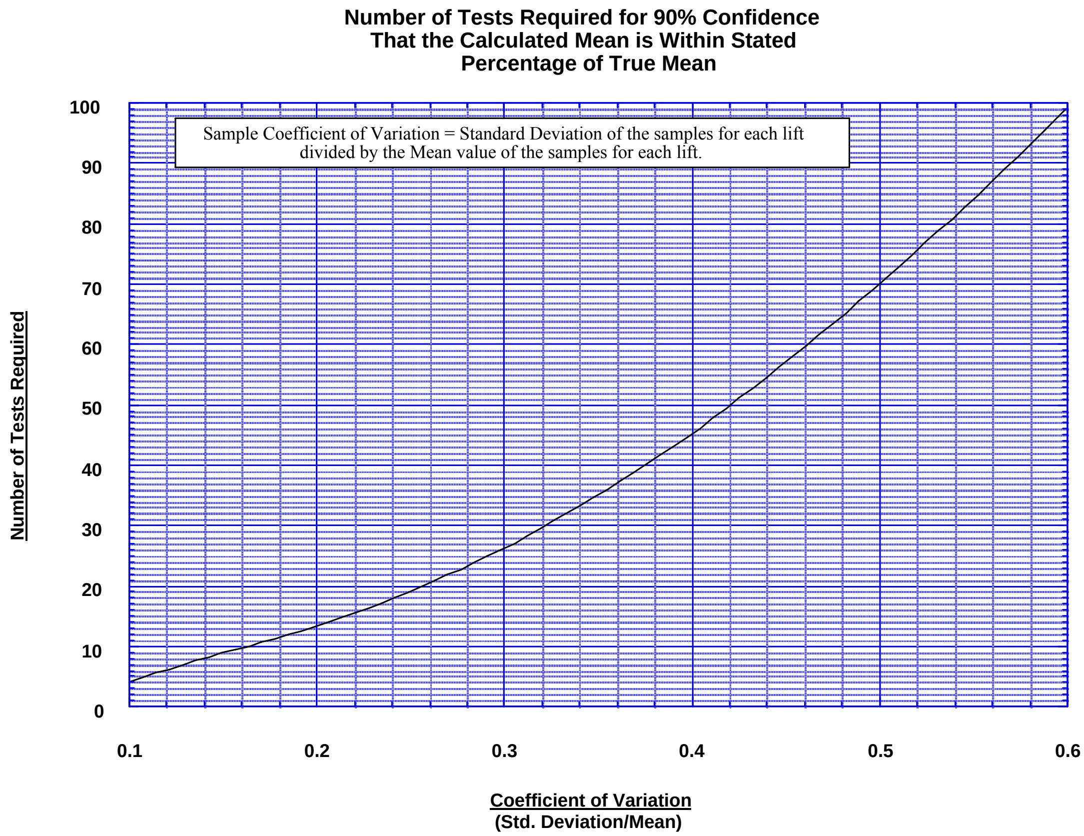


Figure QC-1

Thus, one should make reasonable use of the tools available to assist in assigning realistic K values. One of these tools is the application of simple statistics to the range of K values obtained from tests on soil samples, or from a limited number of field tests.

The use of basic statistical methods has been made somewhat "user friendly" by the availability of numerous dedicated statistical programs for use on desktop and laptop computers. In addition, there are also several spreadsheet and data analysis/graphics program packages available for such computers that are capable of performing statistical analysis. Also, many hand-held electronic calculators have statistical analysis functions programmed into their static memory. Thus, the use of basic statistical methods for analysis of K data and determining the necessary statistical parameters is no longer an involved or difficult task.

When working with transformed variables, as is required when computing the statistics for the values of K determined in the field or laboratory[1], it is necessary to observe the laws of logarithms in making such calculations. Microsoft™Excel™ spreadsheets have been prepared to illustrate the calculation of the statistical parameters such as geometric mean (X_m), standard deviation of geometric mean (s_g), 95% Confidence Interval about the Geometric Mean, and the calculations for determining possible outliers. The computations in these spreadsheets conform to the laws of logarithms.

[1] Recall that K values can be defined by a lognomal distribution.

These laws are:

$Log_a(X * Y) = Log_aX + Log_aY$ *[Note: $Log_a (X+Y) \neq Log_a X + Log_a Y$]*

$Log_a(X/Y) = Log_aX - Log_aY$ *[Note: $Log_aX / Log_aY \neq Log_a (X-Y)$]*

$Log_a X^n = n \, Log_a X$, where n is a real number. *(Note that $Log_a X^n \neq (Log_a X)^n$]*

By combining these laws, we can find the Log of $A^pB^qC^r/D^sE^t$ = p Log A +q Log B + r Log C- s Log D - t Log E. (Spiegel, and Stephens 1999).

The two spreadsheets (Table 1 and Table 2) included in the Appendix make use of natural logarithms because many statistical computer programs use natural logarithms (Ln, = Log to the base e, where e = 2.7183, a mathematical constant) rather than common logarithms (Log_{10} or Log to the base 10). Mathematicians and statisticians normally use natural logs because it simplifies many formulae. (It should be noted that the base used for logarithms affects the values of the logs, but nothing else. Provided the correct anti-log is used to return to the natural (arithmetic) scale, it does not matter which base is used.)

These spreadsheets are provided as an example on how spreadsheets using Excel™, or other spreadsheet and database programs that have statistical functions similar to Excel™, can be used to obtain values for statistical parameters using a lognormal distribution. Tables 1 and 2 show the results before and after removal of the only value determined to be an outlier.

Outliers

Outliers are extreme values of data points that appear to be inconsistent with the trend established by the majority of the data. While extreme values may or may not be outliers, any outliers are always extreme (or relatively extreme) values in the sample set. When no explanations can be found for such inconsistency, outliers should be discarded to eliminate bias from statistical evaluations. Outliers can sometimes be detected by visual examination of plots of the data points, but this can be a subjective decision. There are also procedures available for statistically detecting outliers (Barnett and Lewis-1984; Snedecor and Cochran-1989).

Snedecor and Cochran (1989) state that "A major error in an experiment greatly distorts the mean of (statistical) treatment involved. By inflating the error variance, it affects conclusions about other (statistical) treatments as well. The principal safeguards are vigilance in carrying out the operations, the measurements, and the recording, plus eye examination of the data before analysis. If a figure in the data looks suspicious, an inquiry about this observation should be made. Sometimes the inquiry shows a mistake and also reveals the correct value for the observation, which is then used in the statistical analysis. Sometime it is certain that a gross error was made, but there is no way of finding the correct value. In this event, omit the erroneous observation and use the analysis for data with one missing value. In such cases, check that the source of the gross error did not affect other observations also."

Before omitting any suspicious value (an "outlier") a good faith effort should be made to determine if there is a reasonable explanation of why the value in question differs extremely from other values in the data set. For example, a value for K believed to be an "outlier" may be incorrect because of errors made in selection of sampling locations, sampling procedures, testing procedures, recording of data, or inputting incorrect data into the statistical data processing procedure.

Many statistical procedures are based on the assumption that the data are random in nature. While it is quite difficult to assure that a true random set of samples are obtained, care should be taken not to subjectively select the sampling locations because of the desired end results. A subjective selection might consist of including a sample taken in a small inclusion of soil having a relatively high K value in a body of soil that otherwise can be expected to have a much lower mean K value. A sample knowingly taken from a small, localized deposit of relatively clean sand in a glacial till soil is an example of a subjective selection.

In the case of tube samples, an error might involve using too great a force to push the tube into the soil, thus possibly altering the soil structure and density. Laboratory tests are prone to errors in measurement and technique, even though care is taken in performing the test. Incorrect recording of the laboratory measurement data, such as by misplacement of a decimal point, will contribute an incorrect data point. If such an error is detected by reviewing the measurement data, its correction would provide a valid data point and the test results would not be an outlier. Other errors might occur from a tube sample that contains a worm or decomposed root hole, or a large pebble that would yield a test K value that is not characteristic of the soil deposit.

A common source of error is incorrectly inputting a data point into a hand-held calculator used to derive statistical data from the sample set. If the calculator does not have the ability to recall and display the data points entered, it is difficult to detect such an error unless all of the data are carefully re-entered to check the results of the first data entry. (It is much better to input the data into a computer program capable of statistical calculations that will display, and provide a hard copy of, the data entered so that such errors can be detected visually.)

If all of these possible sources of error have been investigated and discounted, then a method for identifying and dealing with outliers should be used. There are a number of statistical methods available for identifying outliers; many of the more robust methods are given in Barnett and Lewis (ibid). The following method is based on a statistical procedure presented in Barnett and Lewis (ibid) for the case where only the mean and standard deviation of samples are known (i.e. the true mean and standard deviation of the whole universe of values representing the entire soil mass are unknown).

If a set of data is ordered from low to high: X_L, X_2 ... X_H, and the mean (X_m) and standard deviation (s) are calculated, suspected extreme high and low outliers can be tested by the following procedure:

First, calculate the T Score Statistic:

$$\textbf{T score for a high value} = T_H = (X_H - X_m)/s$$
$$\textbf{T score for a low value} = T_L = (X_m - X_L)/s.$$

Second, compare the value of T with the critical value from Figure No. 2 "Critical Values of T at a 5% level of significance" included in this Appendix. If the calculated T is larger than the critical value from Figure No. 2 for the degrees of freedom of the sample set (Degrees of Freedom = N-1, where N = number of sample values in the sample set), then X_H or X_L is an outlier at a 5% level of significance.

This method assumes that the data are normally distributed. However, as previously discussed in Section VI, normal distribution can be assumed by transforming values of K to values of Ln K and calculating the geometric mean and standard deviation from the geometric mean. These values can then be used in calculating the T-score, where the geometric mean = X_m and the standard deviation from the geometric mean = s_g.

[Barnett and Lewis (ibid) state that if X is a log-normal random variable, a test for any outlier in a log-normal sample can be made by taking the logarithms of the observations and apply an appropriate normal sample test to the transformed sample. Davis (1971) states that: "As long as we work with the data in its transformed state, all of the statistical procedures that are appropriate for ordinary variables are applicable to log transformed variables.]

The preceding Barnett and Lewis (ibid) equations can be used for lognormal K data by transforming them to logarithmic equations, as follows:

$$\mathrm{Ln}\, T_H = (\mathrm{Ln}\, K_H - \mathrm{Ln}\, X_m) - \mathrm{Ln}\, s_g$$
$$\mathrm{Ln}\, T_L = (\mathrm{Ln}\, X_m - \mathrm{Ln}\, K_L) - \mathrm{Ln}\, s_g$$

Where:

Ln T_H = Ln of the T score statistic for the highest value of K
Ln T_L = Ln of the T score statistic for the lowest value of K
K_H = Highest value of K
K_L = Lowest value of K
X_m = Geometric Mean K
S_g = Standard Deviation of the Geometric Mean

After obtaining the anti-logs of the Lns of T_H and T_L, the remainder of the procedure is as given on the previous page.

Examples of the calculation for outliers using the method of Barnett and Lewis (ibid.) discussed above are shown in the spreadsheets (Table No. 1 and Table No. 2) included in this Appendix. The approval of the Department should be obtained before using a method for identifying outliers that differs from the cited method.

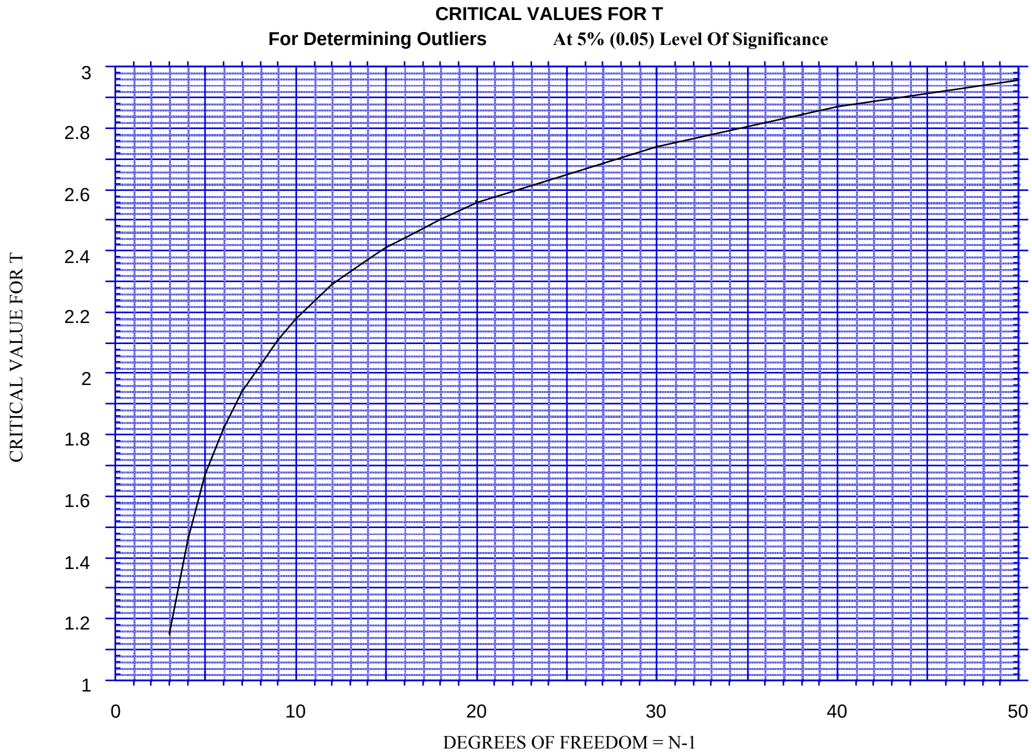


Figure No.2

Selecting K Values for Calculating Hydraulic Capacity and Travel Time

In calculations involving site hydraulic capacity, the mean value of the saturated horizontal hydraulic conductivity of the soil deposit in which the water will travel is needed. However, we cannot determine the mean value of the entire deposit because we have only a limited number of samples that supposedly represent the entire deposit. Thus, we can only estimate the mean value within probable confidence limits determined by the application of statistics.

The procedure recommended by the Department for determining the K value to be used for hydraulic capacity calculations is to calculate the geometric mean of the K samples, determine the 95% confidence interval of that mean, and use the lower interval limit as the design K for hydraulic capacity calculations. A 95% confidence level for a mean value X_m, calculated from a random sample of N values of X, means that we are 95% confident of the probability that the true mean of the variable X (in our case, X = K), will fall within the upper and lower 95% confidence limits (that is, within the 95% confidence interval).

Examples of the calculation for the lower limit of a 95% confidence interval for the geometric mean of lognormal distributed data are shown in the spreadsheets (Table No. 1 and Table No. 2) included in this Appendix. Row 57 in Table No. 1 shows the calculated lower limit of the 95% confidence interval (10.3 ft/day) before discarding the outlier K value of 2.1 ft/day calculated in row 74. Row 56 in Table No. 2 shows the calculated lower limit of the 95% confidence interval (11.3 ft/day) after discarding the outlier K value of 2.1 ft/day. Thus, K = 11.3 ft/day should be used for hydraulic capacity analysis.

The statistical expression for a 95% Confidence Level is as follows:

$$\mathbf{X_m - (t_{(1-\alpha/2)} x\ s_g/N^{0.5}) < True\ Geometric\ Mean < X_m + (t_{(1-\alpha/2)} x\ s_g/N^{0.5})\ where}$$

X_m = Sample Geometric Mean
t = Student's t value for n Degrees of Freedom (N-1),
α = 1- the specified Confidence Level expressed as a decimal (i.e. 1.00-0.95),
$t_{(1-\alpha/2)}$ = the Student's t value at 1-$\alpha/2$ for n degrees of Freedom (two tailed t value)
s_g = Standard Deviation of the Sample Geometric Mean, and,
N = The number of sample values.

Rewriting this expression in logarithmic form, and solving for the high end and low end of the 95% confidence interval

High end of the 95% Confidence Interval = $[Ln (X_m)] + [Ln (t_{(1-\alpha/2)}$ x $Ln (s_g)/N^{0.5})]$

Low end of the 95% Confidence Interval = $[Ln (X_m)] - [Ln (t_{(1-\alpha/2)}$ x $Ln (s_g)/N^{0.5})]$

The antilog of the two intervals will provide the values of K at the high end and low end of the 95% Confidence Interval for the geometric mean K.

Use of the calculated geometric mean K for travel time computations is not appropriate. The travel time will depend upon the ground water velocity and the "horizontal" distance to a point of concern, such as a drinking water well or surface water body. The average linear ground water velocity is calculated from the soil hydraulic conductivity and porosity,[2] and the hydraulic gradient of the water table. However, it should be noted that pathogens might travel faster than the average linear ground water velocity. Those pathogens that have not been adsorbed to a soil particle may be traveling with the ground water through some of the larger micropores and perhaps through macropores, where the ground water velocities are higher than the average velocities. Therefore, the K value used for travel time calculations must be greater than the geometric mean value of K.

Variations in K values of naturally deposited soils will occur depending upon how these deposits were formed. Stratified drift deposits consist of sorted sediments (layers of sand and gravel and lesser amounts of silt and clay) deposited by glacial meltwaters. Thus, for stratified drift aquifers, it is not unlikely that several high values of K, significantly higher than the geometric mean value of a number of sample values of K, may represent a continuous layer of relatively coarse-grained soil lying between other layers of less coarse-grained soils. Ground water flowing in the coarse-grained layer will travel at a faster rate than the ground water in the less coarse-grained layers. In this case, a conservative approach would be to select a K value near the high end of the range of K values as representative of a coarse-grained soil layer when computing horizontal travel times.

[2] It is important that a realistic value of porosity be used in such calculations. Ranges of values of porosities can be found in: Table 3 in Melvin, R. L., V. de Lima, and B. D. Stone. (1992) and Appendix B in Walton, W. C. (1991). It is recommended that the lower end of the range of porosity given in these references be used for travel time computations.

On the other hand, glacial till deposits consist of non-stratified, non-sorted, intermingled clay, silt, sand and boulders transported and deposited by glacial ice. Thus, it is less likely that the highest values of K would represent a continuous layer (continuous path) of coarse-grained soils and a K value between the geometric mean and high-end values of K is considered to be appropriate when computing horizontal travel time through a glacial till soil deposit. However, it should be noted that each soil horizon, or layer, through which the ground water will flow must be analyzed separately for travel time.

In the case of fill type systems, it is likely that each layer of fill will consist of truckloads of granular soil that have been mixed during loading, unloading and placement of the fill material. In this case, the high-end values of K are considered unlikely to represent a continuous layer of coarser-grained soils and each layer can be considered in a manner similar to that of a glacial till soil given above.

Accordingly, it is suggested that a K value equal to the 95th percentile value of K values of coarse-grained strata be used in computing horizontal travel time in deposits of stratified drift, and a K value equal to the third quartile value of K be used in computing horizontal travel time in glacial till. The method of determining the 95th percentile value and third quartile values for K is shown on the spreadsheets included in this Appendix. The spreadsheets also indicate the method of solving for K_m, s_g, and C_v using logarithms.

Where the Applicant's Consultant does not wish to follow the procedures discussed herein for selecting K values for hydraulic capacity and travel time computations, the Department will require that a conservative low value for K be used to calculate hydraulic capacity and a conservative high value for K be used in travel time computations. Thus, the K value for hydraulic capacity should be selected near the low end of the range of K values and the K value for travel time computations should be selected near the high end of the range of the K values for the soil in question.

	A	B	C	D	E	F
1			**TABLE No. 1**			
2			**Statistics for Values of K, ft/d (Before Removal of Outliers)**			
3						
4	**K**	**Ln K**	**EXCEL Statistical**	**Statistical**	**Antilog (EXP)**	**Parameter**
5	ft/day		**Function**	**Parameter**	**of Parameter LN**	**Value**
6	24.1	3.18221	= LN(B6)	Natural Log (LN) of K		
7	26.1	3.26194	.			
8	10.2	2.32239	.			
9	2.1	0.74194	.			
10	22.4	3.10906	.			
11	12.7	2.54160	.			
12	19.2	2.95491	.			
13	13.5	2.60269	.			
14	18.4	2.91235	.			
15	9.2	2.21920	.			
16	35.2	3.56105	.			
17	34.7	3.54674	.			
18	12.0	2.48491	.			
19	4.6	1.52606	.			
20	10.4	2.34181	.			
21	11.2	2.41591	.			
22	24.0	3.17805	.			
23	11.3	2.42480	.			
24	9.0	2.19722	.			
25	31.0	3.43399	.			
26	14.2	2.65324	.			
27	12.8	2.54945	.			
28	6.7	1.90211	.			
29	12.5	2.52573	.			
30	11.5	2.44235	.			
31	46.1	3.83081	.			
32	7.7	2.04122	.			
33	10.5	2.35138	.			
34	5.1	1.62924	.			
35	12.2	2.50144	.			
36	10.6	2.36085	= LN(B36)			
37						
38		2.57247	=AVERAGE(B6:B36)	= Mean of LNs = Geomean = X_m	=EXP(B38)	13.1
39		2.50144	= MEDIAN(B6:B36)	= Median of LNs	=EXP(B39)	12.2
40		0.64923	= STDEV(B6:B36)	= Std. Dev. Of Geomean = s	=EXP(B40)	1.91
41		31	= COUNT(B6:B36)	= The number of values of K		
42						
43						
44		2.33210	QUARTILE (array, quart)= QUARTILE(B6:B36, 1)	=1st Quartile Value of K	= EXP(B44)	10.3
45		2.50144	QUARTILE (array, quart)= QUARTILE(B6:B36, 2)	=2nd Quartile Value of K	= EXP(B45)	12.2
46		3.03199	QUARTILE (array, quart)= QUARTILE(B6:B36, 3)	=3rd Quartile Value of K	= EXP(B46)	**20.7**
47						

48		
49		**CALCULATE 95% CONFIDENCE INTERVAL FOR GEOMETRIC MEAN**
50		
51	31	= B41, which returns the N, the Number of Values
52	5.57	= SQRT(B51), which returns the Square Root of N
53	2.042	= TINV(0.05,30), which returns the value of Student's t for 95% Confidence and N-1 Degrees of Freedom.
54		[Where the input value 0.05 = (100-% Confidence)/100.]
55	13.1	=EXP(B38) =Mid value of interval, which is the Geometric Mean calculated in cell B38
56	**16.6**	= EXP(B38+B53*B40/B52), which returns the high end of the 95% interval
57	**10.3**	= EXP(B38-B53*B40/B52), which returns the Low end of the 95% interval
58		
59		Thus, we are 95% confident that the true geometric mean K lies within the interval 10.3 to 16.6.
60		
61		**CHECK VALUES OF K FOR POSSIBLE OUTLIERS**
62		(Using procedure given in Barnett and Lewis*)
63		
64	3.83081	= MAX(B6:B36), which returns the LN of Highest Value of K_H
65	0.60911	= (B31-B38)-B40, which returns the LN of the T statistic for Highest K Value = (LN(X_H)-LN (X_m))-LN(s)
66	**1.84**	= EXP(B65), which returns the value of the T statistic for Highest K Value
67	0.74194	= MIN(B6:B36), which returns the LN of Lowest Value of K = K_L
68	1.18131	=(B38-B9)-B40, which returns the LN of the T statistic for Lowest K Value = (LN(X_m)-LN(X_L)-LN(s)
69	**3.26**	= EXP(B68), which returns the value of the T statistic for Lowest K Value
70	2.74	= Critical Value of T for N-1 Degrees of Freedom, Input from Figure No. 2
71		
72		
73		Since T statistic for Highest K value of 46.1 is less than Critical Value of T, 46.1is not an outlier
74		Since T statistic for lowest K value of 2.1 is greater than Critical Value of T, 2.1 is an outlier
75		
76		Therefore, the procedure shown in this spreadsheet should be repeated for all values of K except for
77		K = 2.1, which should be discarded.
78		
79		* Reference: Barnett, V. and T. Lewis. 1984. Outliers in Statistical Data. John Wiley & Sons, New York, NY
80		Pages 216-223 and 250-251.
81		
82		**CALCULATE 95% PERCENTILE VALUE OF K**
83		
84	3.55389	= PERCENTILE(B6:B36, 0.95), which returns the LN of the 95 Percentile Value of K
85	**34.9**	= EXP(B84), which returns the 95 Percentile Value of K
86		
87		**CALCULATE COEFFICIENT OF VARIATION, C_v**
88		
89	-1.92324	= B40-B38, which returns the LN of C_v (Cv= Std. Dev.of Geomentric Mean/Geometric Mean)
90	**0.15**	= EXP(B89), which returns the value of the Coefficient of Variation, C_v, expressed as a decimal.
91		

	A	B	C	D	E	F
1			**TABLE No. 2**			
2			**Statistics for Values of K, ft/d (After Removal of Low End Outlier)**			
3						
4	**K**	**Ln K**	**EXCEL Statistical**	**Statistical**	**Antilog (EXP)**	**Parameter**
5	ft/day		**Function**	**Parameter**	**of Parameter LN**	**Value**
6	24.1	3.18221	= LN(B6)	Natural Log (LN) of K		
7	26.1	3.26194	.			
8	10.2	2.32239	.			
9	22.4	3.10906	.			
10	12.7	2.54160	.			
11	19.2	2.95491	.			
12	13.5	2.60269	.			
13	18.4	2.91235	.			
14	9.2	2.21920	.			
15	35.2	3.56105	.			
16	34.7	3.54674	.			
17	12.0	2.48491	.			
18	4.6	1.52606	.			
19	10.4	2.34181	.			
20	11.2	2.41591	.			
21	24.0	3.17805	.			
22	11.3	2.42480	.			
23	9.0	2.19722	.			
24	31.0	3.43399	.			
25	14.2	2.65324	.			
26	12.8	2.54945	.			
27	6.7	1.90211	.			
28	12.5	2.52573	.			
29	11.5	2.44235	.			
30	46.1	3.83081	.			
31	7.7	2.04122	.			
32	10.5	2.35138	.			
33	5.1	1.62924	.			
34	12.2	2.50144	.			
35	10.6	2.36085	= LN(B35)			
36						
37		2.63349	=AVERAGE(B6:B35)	= Mean of LNs = Geomean = X_m	=EXP(B37)	13.9
38		2.51358	= MEDIAN(B6:B35)	= Median of LNs	=EXP(B38)	12.3
39		0.56270	= STDEV(B6:B35)	= Std. Dev. Of Geomean = s	=EXP(B39)	1.76
40		30	= COUNT(B6:B35)	= The number of values of K		
41						
42						
43		2.34420	QUARTILE (array, quart)= QUARTILE(B6:B36, 1)	= 1st Quartile Value of K	= EXP(B43)	10.4
44		2.51358	QUARTILE (array, quart)= QUARTILE(B6:B36, 2)	= 2nd Quartile Value of K	= EXP(B44)	12.3
45		3.07052	QUARTILE (array, quart)= QUARTILE(B6:B36, 3)	= 3rd Quartile Value of K	= EXP(B45)	**21.6**
46						

CALCULATE 95% CONFIDENCE INTERVAL FOR GEOMETRIC MEAN

30 = B40, which returns the N, the Number of Values
5.48 = SQRT(B50), which returns the Square Root of N
2.042 = TINV(0.05,30), which returns the value of Student's t for 95% Confidence and N-1 Degrees of Freedom.
[Where the input value 0.05 = (100-% Confidence)/100.]
13.9 =EXP(B37) =Mid value of interval, which is the Geometric Mean calculated in cell B38
17.2 = EXP(B37+B52*B39/B51), which returns the high end of the 95% interval
11.3 = EXP(B37-B52*B39/B51), which returns the Low end of the 95% interval

Thus, we are 95% confident that the true geometric mean K lies within the interval 10.3 to 16.6.

CHECK VALUES OF K FOR POSSIBLE OUTLIERS

(Using procedure given in Barnett and Lewis*)

3.83081 = MAX(B6:B35), which returns the LN of Highest Value of K_H
0.63462 = (B30-B37)-B39, which returns the LN of the T statistic for Highest K Value = (LN (X_H)-LN (X_m))-LN(s)
1.89 = EXP(B64), which returns the value of the T statistic for Highest K Value
1.52606 = MIN(B6:B35), which returns the LN of Lowest Value of K = K_L
0.54473 =(B37-B66)-B39, which returns the LN of the T statistic for Lowest K Value = (LN(X_m)-LN(X_L)- LN(s)
1.72 = EXP(B67), which returns the value of the T statistic for Lowest K Value
2.70 = Critical Value of T for a 5% level of significance and N-1 Degrees of Freedom (Input from Figure No. 2)

Since T statistic for Highest K value of 46.1 is less than Critical Value of T, 46.1 is not an outlier
Since T statistic for lowest K value of 4.6 is less than Critical Value of T, 2.1 is not an outlier
Therefore, the statistical parameters calculated in this table are OK for use in hydraulic capacity and travel time calculations.

* Reference: Barnett, V. and T. Lewis. 1984. Outliers in Statistical Data. John Wiley & Sons, New York, NY Pages 216-223 and 250-251.

CALCULATE 95% PERCENTILE VALUE OF K

3.55461 = PERCENTILE(B6:B36, 0.95), which returns the LN of the 95 Percentile Value of K
35.0 = EXP(B84), which returns the 95 Percentile Value of K

CALCULATE COEFFICIENT OF VARIATION, C_v

-2.07079 = B40-B38, which returns the LN of C_v (Cv= Std. Dev.of Geomentric Mean/Geometric Mean)
0.13 = EXP(B89), which returns the value of the Coefficient of Variation, C_v, expressed as a decimal.

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APPENDIX D

U.S. --METRIC CONVERSION FACTORS

METRIC (SI*) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS					APPROXIMATE CONVERSIONS FROM SI UNITS				
Symbol	When You Know	Multiply By	To Find	Symbol	Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH					LENGTH		
in	Inches	25.4	millimeters	mm	mm	millimeters	0.039	Inches	in
ft	feet	0.3048	meters	m	m	meters	3.28	feet	ft
yd	yards	0.914	meters	m	m	meters	1.09	yards	yd
mi	Miles (statute)	1.61	kilometers	km	km	kilometers	0.621	Miles (statute)	mi
		AREA					AREA		
in^2	square inches	645.2	millimeters squared	cm^2	mm^2	millimeters squared	0.0016	square inches	in^2
ft^2	square feet	0.0929	meters squared	m^2	m^2	meters squared	10.764	square feet	ft^2
yd^2	square yards	0.8361	meters squared	m^2	m^2	meters squared	1.1960	square feet	ft^2
ac	acres	0.4046	hectare	ha	ha	hectare	2.4711	acres	ac
		MASS (weight)					MASS (weight)		
oz	Ounces (avdp)	28.35	grams	g	g	grams	0.0353	Ounces (avdp)	oz
lb	Pounds (avdp)	0.454	kilograms	kg	kg	kilograms	2.205	Pounds (avdp)	lb
		VOLUME					VOLUME		
fl oz	fluid ounces (US)	29.57	milliliters	mL	mL	milliliters	0.034	fluid ounces (US)	fl oz
gal	Gallons (liq)	3.785	liters	liters	liters	liters	0.264	Gallons (liq)	gal
ft^3	cubic feet	0.0283	meters cubed	m^3	m^3	meters cubed	35.315	cubic feet	ft^3
yd^3	cubic yards	0.765	meters cubed	m^3	m^3	meters cubed	1.308	cubic yards	yd^3
	Note: Volumes greater than 1000 L are shown in m^3								
		VELOCITY					VELOCITY		
ft./s	feet per second	0.3048	meters/second	m/s	m/s	meters/second	3.2808	feet per second	ft./s
ft./d	feet per day	0.3048	meters/day	m/d	m/d	meters/day	3.2808	feet per day	ft./d
ft./d	feet per day	0.35278×10^{-3}	centimeters/s	cm/s	cm/s	centimeters/s	2834.6	feet per day	ft./d
		FLOW RATE					FLOW RATE		
ft^3/s	cubic feet per sec.	2.8317×10^{-2}	cubic meters/sec	m^3/s	m^3/s	cubic meters/sec.	35.3147	cubic feet per sec.	ft^3/s
gal/d	gallons per day	4.3808×10^{-5}	liters per second	L/s	L/s	liters per second	22,831	gallons per day	gal/d
Gal/d	Gallons per day	3.785×10^{-3}	Cubic meters/day	m^3/d	m^3/d	cubic meters /day	264.20	Gallons per day	Gal/d
gal/m	gallons per min.	6.3083×10^{-2}	liters per second	L/s	L/s	liters per second	15.8521	gallons per min.	gal/m
gal/d	gallons per min.	6.3095×10^{-5}	cubic meters/sec.	m^3/s	m^3/s	cubic meters/sec.	15,849	gallons per min.	gal/m
		LOADING RATE					LOADING RATE		
$gal/d/ft^2$	gal/day/sq ft	4.075	centimeters/day	cm/d	cm/d	centimeters/day	0.2454	gal/day/sq ft.	$gal/d/ft^2$
		TEMPERATURE					TEMPERATURE		
^oF	Fahrenheit	$5/9\ (^oF-32)$	Celsius	^oC	^oC	Celsius temperature	$9/5\ ^oC+32$	Fahrenheit	^oF

* SI = Système International (International System [of Units]).

APPENDIX E

GLOSSARY

Note: The Glossary contains definitions and discussions of terms contained in "Guidance for Design of Large-Scale On-Site Wastewater Renovation Systems" as well as terms contained in many of the published articles referenced in the various sections therein.

Abiotic	Pertaining to or characterized by the absence of living organisms.
Ablation Till	Loose, permeable till deposited during the final meltdown of glacial ice. Lenses of crudely sorted sands and gravel are common.
Absorbent	A substance that is capable of absorbing another substance.
Absorption	To take a substance in through pores or interstices. The process by which one substance is taken into and included within another substance, as the absorption of water by soil or nutrients by plants.
Acid	A substance which increases the concentration of hydronium ion in solution. A proton donor.
Actinomycetes	A group of microorganisms with characteristics intermediate between simple bacteria and fungi.
Adsorption	The adhesion of a substance to the surface of a solid or liquid. The increased concentration of molecules or ions at a surface, including exchangeable cations and anions on soil particles.
Adsorption Isotherm	A graph of the quantity of a given chemical species bound to an absorption complex, at a fixed temperature, as a function of the concentration of the species in a solution that is in equilibrium with the complex.
Aerobic Bacteria	Bacteria that require free elemental oxygen for their growth. Strict aerobes utilize the elemental oxygen for their terminal electron acceptor.
Aerobic(Oxic) Conditions -	where molecular oxygen is present at greater than 0.4 mg/L.
Aggregate Soils	Many fine particles held in a single mass or cluster. Natural soil aggregates, such as granules, blocks, or prisms, are called peds. Clods are aggregates produced by tillage or logging.
Ammonification	The decomposition of organic compounds e.g. proteins, by microorganisms with the release of ammonia. Also, the change from organic nitrogen to the ammonium form.
Amorphous	Lacking definite form (e.g.: shapeless; lacking organization; lacking distinct crystalline structure)

Amphoteric	Having the characteristics of an acid and a base and capable of reacting chemically either as an acid or a base.
Anaerobic Bacteria	Strict anaerobes use "chemically bound" oxygen rather than free elemental oxygen, which is toxic to such organisms. They use carbon, nitrogen or sulfur compounds as their terminal electron acceptor.
Anaerobic Conditions	Where molecular oxygen is present at less than 0.4 mg/L and NO_X-N is present at less than 0.2 mg/L.
Anaerobic Digestion	The process of decomposing organic matter in sewage by anaerobic bacteria.
Anaerobic Respiration	Respiration under anaerobic conditions in which a terminal electron acceptor other than oxygen is involved, the more common acceptor molecules being carbonate, sulfate and nitrate.
Anion	An atom that is negatively charged because of a gain in electrons.
Anion Exchange Capacity	The sum total of exchangeable anions that a soil can adsorb. Expressed as milliequivalents per 100 grams of soil or other adsorbing material (such as clay).
Anoxic Zone	An anoxic zone provides the environment necessary for facultative, heterotrophic bacteria to use nitrates in place of oxygen in their respiratory process.
Anoxic Conditions	Where D.O. is less than 0.4 mg/L and NO_X-N is greater than 0.5 mg/L.
Antibiotic	A chemical substance produced by certain molds and bacteria which inhibits the growth of or kills other microorganisms.
Antimicrobial	Capable of destroying or suppressing the growth of microorganisms.
Anthropogenic	of, relating to, or involving the impact of humans on nature.
Antigen	Certain compounds of microbial cells that can incite production of a specific antibody and that can combine with that antibody. Also, any substance that the body regards as foreign or dangerous for which an antibody is produced.

Anisotrophic	A condition where the hydraulic conductivity varies with the direction of measurement at a point in a geologic formation.
Antibodies	specific blood serum proteins, which bind to the antigens of microbial cells. Also, blood protein made in lymphoid tissue in response to the presence of antigens.
Aquifer	A water-bearing stratum of permeable rock, sand, or gravel.
Aquic conditions	Conditions where the soils are saturated and chemically reduced.
Arithmetic Mean	The average, calculated by dividing the sum of all values by the number of values to be averaged.
Artifact	A structure or substance not normally present but produced by an external agent or action.
Assimilation	The conversion of nutritive material into protoplasm.
Asymptomatic	Neither causing nor exhibiting symptoms of disease. Presenting no subjective evidence of disease.
Autoclave	A strong, pressurized, steam-heated vessel, as for laboratory experiments, sterilization, or cooking.
Autotrophic Bacteria	Microorganisms that use inorganic materials (carbon dioxide or carbonates) as a source of nutrients. Autotrophic bacteria utilize inorganic compounds entirely to produce an organic end product. Inorganic salts furnish the building blocks, as well as the energy.
Available Moisture Capacity	The moisture content of the soil in excess of the wilting point that can be taken up by plants at rates significant to their growth.
Bacillus	Any rod shaped bacterium.
Bacteriophage	A submicroscopic virus that can infect and destroy bacterial cells. (Also, see Coliphage)
Bacteria Respiration	The oxidative process occurring within living cells by which the chemical energy of organic molecules is released in a series of metabolic steps involving the consumption of oxygen and the liberation of carbon dioxide and water.

Bar	A unit of pressure; the pressure exerted by the entire atmosphere on one square centimeter is approximately one bar. Normal atmospheric pressure (at sea level) is 1.013 bars or 1,013 millibars, which is equivalent to 29.92 inches or 760 millimeters.
Base	A substance that increases the concentration of hydroxide ion in solution and reacts with an acid to produce a salt and water. A proton acceptor.
Basal Till	Any compact glacial till, deposited beneath the ice, nearly impervious to water.
Bedrock	A solid and continuous body of rock, with or without fractures or faults, and including weathered bedrock overlying solid bedrock.
Bentonite	A chemically altered volcanic ash that consists primarily of the clay mineral montmorillonite. The bentonite has a charge of 70-90 meq/gram.
Biodegradable	capable of being decomposed by biological processes.
Biofilm	a matrix of polysaccharide polymer produced externally by bacteria for attachment to surfaces, protection from environmental attack, and enhanced nutrient capture. It is normally a slimy material that is insoluble in water and most acids. Bacteria produce from 30 to 100 times their own weight in biofilm and, if external pressures are placed on a biofilm, it responds first by tightening or compacting, and shortly thereafter by producing larger amounts of exopolymers as a defense mechanism.
Biomat (Biocrust)	A growth of a biological or zoogleal layer at the soil interface of a subsurface soil absorption system. This growth forms a clogging layer of only a few cm in thickness. The biomat consists of wastewater solids, microorganisms, mineral precipitates, and the detritus remaining after decomposition of organic matter. The permeability and thickness of the biomat depends to a great extent upon the type of soil, wastewater loading rate, and wastewater quality (strength). Also referred to as clogging zone or clogging mat.
Biosolids	The residues of wastewater treatment. Formerly called sewage sludge.
Bioturbation	Disturbances of soils by animals or plants.

BOD (Biochemical Oxygen Demand)	The amount of oxygen required to maintain aerobic conditions during decomposition.
Bulk Density (Soil)	The mass of dry soil per unit bulk volume. The bulk volume is determined before drying to a constant weight at 105°C.
Calcareous Soil	A soil containing enough calcium carbonate (commonly combined with magnesium carbonate) to effervesce visibly when treated with cold, dilute hydrochloric acid.
Capillary attraction	A liquid's movement over or retention by a solid surface due to the interaction of adhesive and cohesive forces.
Capillary fringe	A zone just above the water table that is maintained in an essentially saturated state by capillary forces of lift.
Capsid	protein coat of a virus.
Capsule	Compact layer of polysaccharide exterior to the cell wall in some bacteria.
Cation	An atom that is positively charged because of loss of one or more electrons.
Cation Exchange	The interchange between a cation in solution and another cation on the surface of any surface-active material such as a clay colloid or organic colloid.
Cation Exchange Capacity (CEC)	CEC is a measure of the chemical reactivity of the soil and is generally an indication of the effectiveness of the soil in adsorbing cationic contaminants from wastewater. The adsorption occurs as a result of the attraction of the positively charged cations by negative charges that exist on the surfaces of clay minerals, hydrous aluminum and iron oxides, and organic matter. CEC is the sum total of exchangeable cations that a soil can absorb; sometimes referred to as total-exchange, base-exchange capacity, or cation-absorption capacity. Expressed in milliequivalents per 100 grams of exchanger. (Also defined as the total charge on the surfaces of the soil system.)
Chemisorption	To take up and chemically bind (a substance) onto the surface of a substance.

Chroma	The aspect of color in the Munsell color system by which a sample appears to differ from a gray of the same lightness or brightness and that corresponds to saturation of the perceived color. The relatively purity or saturation of a color, or its intensity of distinctive hue as related to grayness. Chroma is one of the 3 variables of soil color defined within the Munsell system of classification.
Class (Biology)	A taxonomic category ranking below a phylum or division and above an order.
Clay	Soil material that contains 40% or more clay, less than 45% sand, and less than 40% silt. Plastic when wet and hardens when heated. Mineral soil particles less than 0.0002 in dia. consisting primarily of hydrated silicates of aluminum.
Coagulate, coagulation	To cause transformation of dissolved suspended or colloidal matter in solution into a soft, semisolid, or solid mass for the purpose of precipitation (removal) of the matter from the liquid in which it is contained.
COD (Chemical Oxygen Demand)	A measure of the oxygen equivalent of that portion of organic matter that is susceptible to oxidation by a strong chemical agent.
Cocci	Bacteria having a spherical or spheroidal shape.
Cohesion	The force holding a solid or liquid together, owing to the attraction between like molecules.
Coliform	Bacteria that commonly inhabit the intestines of human beings and other vertebrates.
Coliphages	Bacteriophages-viruses that infect the Escherichia Coli (E Coli) bacterium. They resemble human enteric viruses such as poliovirus and Hepatitis A virus in size, shape and composition.
Colloids (colloidal matter)	Particles ranging from molecular dimensions to less than 1μ in diameter.
Confined Space	OSHA defines a confined space as being large enough for a person to enter, having a restricted means of entry or exit, and not designed for human occupancy.
Contaminant (in Water)	An undesirable constituent in the water or wastewater that may directly or indirectly affect human or environmental health.

Cryptosporidium	A protozoan parasite that can live in the intestines of humans and animals. *Cryptosporidium parvum* is a species of *Cryptosporidium* known to infect humans, causing the gastrointestinal disease known as cryptosporidiosis.
Cyst	A small capsule-like sac that encloses certain microorganisms in their dormant or larval stage. A resting stage formed by some bacteria and protozoa in which the whole cell is surrounded by a protective layer. The infectious stage of *Giardia*, and some other protozoan parasites, that has a protective wall, which enables it to survive in water and other environments.
Denaturation	Any process in which the molecular structure of a substance, especially that of a protein or nucleic acid, is artificially altered in order to eliminate or modify one or more of its characteristic chemical, physical, or biological properties. Inactivation of viruses occurs by denaturation of the viral protein coat.
Denitrification	The reduction of nitrates to nitrogen gas. A two-step sequential process that usually takes place only under anoxic conditions. Organic matter must be available to the denitrifying bacteria. In the first step, nitrate is reduced to nitrite. In the second step, the nitrite is reduced to nitrogen gas, N_2, which is released to the atmosphere. For denitrification to occur, nitrification of ammonia must first take place. Denitrification may be carried out heterotrophically by common facultative bacteria, for example species of Pseudonomonas, Alcaligenes, Paracoccus, Bacillus, Propionibacterium, etc. These organisms metabolize compounds for carbon and energy."
Design Flow	The daily flow rate that an on-site wastewater renovation system is designed to accommodate on a sustained basis while satisfying all permit discharge limitations and treatment and operational requirements. The design flow incorporates peaking and safety factors to ensure sustained, reliable operation.
Dessicate, Dessication	To dry out thoroughly, to make dry and lifeless.
Desorption	The process of removal of a previously adsorbed substance.
Diffuse Double Layer	A heterogeneous system that consists of a solid surface having a net electrical charge, together with an ionic swarm

	under the influence of the solid and in a solution phase that is in direct contact with the surface of the solid. The electrical double layer consisting of a charged-particle surface (usually negatively charged) and a surrounding sheath of ions of charge opposite to that of the particle surface.
Disinfection	A reduction in the concentration of pathogens to non-infectious levels.
DNA	Deoxyribonucleic Acid. A nucleic acid that carries the genetic information in the cell and is capable of self-replication and synthesis of RNA. DNA consists of two long chains of nucleotides twisted into a double helix. The sequence of nucleotides determines individual hereditary characteristics. Also see RNA.
D.O.	Dissolved Oxygen
Domestic Wastewater	Wastewater from residential buildings or from non-residential buildings, but not including manufacturing process water, cooling water, wastewater from water softening equipment, commercial laundry wastewater, dry cleaning wastewater, blow-down from heating or cooling equipment, water from cellar or roof drains or surface water from roofs, paved surfaces, or yard drains.
Domestic Well	A self-supplied ground water source for household water.
Domestic Well Water	Untreated ground water collected from domestic wells.
Drumlim	An elongated or oval hill of glacial drift.
Effective porosity	that portion of the total porosity that contributes significantly to fluid flow.
Effective Size, (d_{10})	The percent by weight of the soil particles that pass the No. 10 standard mesh sieve.
Electrical Double Layer	See Diffuse Double Layer.
Electron	A stable, negatively charged subatomic particle.
Electron Acceptor	A substance being reduced. A substance that accepts electrons during an oxidation-reduction reaction; an oxidant.
Electron Donor	A substance being oxidized. A substance that donates electrons in an oxidation-reduction reaction; a reductant.

Eluviation	Leaching. The process of removing nutrients and inorganics out of the A horizon of soils. The removal of soil material in suspension (or in solution, as in leaching) from a layer or layers of soil.
Elution	Extraction of one material from another, as by washing out adsorbed material i.e.: The removal of virus from soil by washing with a liquid.
Endemic	Prevalent in or peculiar to a particular locality, region, or people. Native to or confined to a certain region.
Endogenous	Produced or originating from within.
Energy release	Energy is released by oxidation reactions. Organic matter in wastewaters is stabilized by oxidation. Bacteria and other microorganisms in waste stabilization systems do not oxidize matter by the direct addition of oxygen, but rather by the indirect scheme of hydrogen removal and addition of water. The hydrogen eventually reacts with oxygen, carbon, nitrogen or sulfur.
Enteric bacteria	General term for a group of bacteria that inhabit the intestinal tract of humans and other animals. Among this group are pathogenic bacteria such as *Salmonella* and *Shigella.*
Enterovirus	A virus that infects cells of the intestinal tract.
Enzyme	Protein within or derived from a living organism that functions as a catalyst to promote specific reactions. Complex proteins which act as organic catalysts to speed up the rate of hydrolysis of complex organic compounds and the rate of oxidation of simple compounds. Capable of action outside or inside the cell. Certain of the enzymes are secreted by the cell and are known as extracellular enzymes. Others are associated with the protoplasm of the cell and perform their function within the cell: These are known as intracellular enzymes. Enzymes are proteinaceous in character.
Epidemiology	The branch of medicine that deals with the study of the causes, distribution, and control of disease in populations.
Equivalent Spherical Diameter of Particle	The diameter of a sphere that has a volume equal to the volume of the particle.
Etiology	The branch of medicine that deals with the causes or origins of disease. The cause or origin of a disease or

	disorder as determined by medical diagnosis i.e.: The causal relationship between a virus and the specific disease.
Etiologic Agent	Something that causes disease.
Eukaryote	A single-celled or multicellular organism whose cells contain a distinct membrane-bound nucleus.
Eutrophic	Nutrient -rich.
Evaporation	The change from a liquid state to a vapor state. With respect to soil systems, it is the process by which molecules of water at the surface of moist soils acquire enough energy through solar radiation to escape to the atmosphere.
Evapotranspiration	The process by which water in the land surface, soil and vegetation is converted into the vapor state and returned to the atmosphere. It consists of evaporation from water, soil, vegetative and other surfaces and transpiration by vegetation.
Excystation	The release of the internal contents of cysts or oocysts. The mechanism by which ingested Cryptosporidium oocysts cause human and animal infection.
Exogenous	produced or originating from without.
Extracellular	Located or occurring outside a cell or cells.
Facultative Bacteria	Facultative bacteria can use most aerobic and anaerobic mechanisms, and also can use "chemically bound" oxygen, carbon, nitrogen or sulfur as their hydrogen acceptor. They will use the hydrogen acceptor yielding the greatest energy. Thus, facultative bacteria will not use carbon as their hydrogen acceptor when dissolved oxygen is present.
Failure (SSAS)	When a subsurface soil absorption system (SSAS) does not properly contain or treat wastewater or causes or threatens to cause the discharge of partially treated wastewater onto the ground surface or into adjacent ground water or surface water.
Family (Biology)	A taxonomic category of related organisms ranking below an order and above a genus. A family usually consists of several genera.
Fecal Contamination	Contamination derived from the feces of humans and other animals. Includes bacteria, viruses and parasites.

Fermentation	The metabolic process in which the final electron acceptor is an organic compound.
Field Moisture Capacity (Field Capacity)	The quantity of water held in a soil by capillary action after the gravitational or free water has been allowed to drain. Expressed as a moisture percentage, dry weight basis.
Fill Material	An approved soil material, meeting specific particle size (gradation) requirements, placed on an existing soil horizon and used as part of the soil system to provide treatment of septic tank effluent. Does not include soil material used to contain or cover a subsurface wastewater absorption system (SWAS).
Filtrate	The liquid that has passed through a filter.
Floc	A flocculent mass formed in a fluid through precipitation or aggregation of suspended particles.
Flocculation	Gentle mixing to promote the aggregation of discrete suspended particles of small size into larger particulate matter (“floc”) to enhance removal of particulates by settling in clarification processes.
Formula Weight	The sum of all the atomic weights in the chemical formula under consideration. If the molecular formula is used, the formula weight is called the molecular weight.
Fragipan	a loamy, brittle subsurface horizon low in porosity and content of organic matter and low or moderate in clay but high in silt or very fine sand. A fragipan appears cemented and restricts roots. When dry, it is hard or very hard and has a higher bulk density than the horizon or horizons above. When moist, it tends to rupture suddenly under pressure rather than deform slowly. This horizon is mottled, slowly or very slowly permeable to water, and usually shows occasional or frequently bleached cracks forming polygons.
Genus	A taxonomic category ranking below a family and above a species and generally consisting of a group of species exhibiting similar characteristics. In taxonomic nomenclature the genus name is used, either alone or followed by a Latin adjective or epithet, to form the name of a species.
Geometric Mean	The average of a set of numbers covering a wide range that do not fit a normal distribution. The geometric mean is calculated by converting each of the numbers in the set to a log value, adding up all log values and dividing by the

number of samples. The geometric mean is the anti-log of the sum of the log values. Also referred to as the log mean.

Giardia Lamblia	A single celled flagellated protozoan of the genus Giardia that may be parasitic in the intestines of vertebrates including human beings and most domestic animals. Can cause a gastrointestinal disease called giardiasis.
Glacial Drift	Pulverized and other rock material transported by glacial ice and then deposited. Also the sorted and unsorted material deposited by streams flowing from glaciers.
Glaciofluvial Deposits	Material moved by glaciers and subsequently sorted and deposited by streams flowing from the melting ice. The deposits are stratified and occur as kames, eskers, deltas and outwash plains.
Glacial Outwash	Gravel, sand, and silt, commonly stratified, deposited by melt water as it flows from the glacial ice.
Glacial Till	Unsorted, non-stratified glacial drift consisting of clay, silt, sand and boulders, intermingled in any proportion, transported and deposited by glacial ice.
Gleyed soil	In the B and C soil horizons, a soil matrix with grayish to bluish hues, indicating soils with pores filled with water (saturated) for prolonged periods, caused by leaching of iron from the soil. Usually indicates a poorly or very poorly drained soil.
Graywater	Wastewater generated from non-toilet plumbing fixtures. i.e. discharge from sinks, basins, dishwashers, bathtubs, or showers, etc.
Hardpan	A hardened soil layer, in the lower A or in the B horizon, caused by cementation of soil particles with organic matter or with materials such as silica, sesquioxides or calcium carbonate.
Helminth	A worm, especially a parasitic roundworm or tapeworm.
Heterotroph	a microorganism that is unable to use carbon dioxide as its sole source of carbon and requires one or more organic compounds.
Heterotrophic Bacteria	Those bacteria which can only utilize organic matter for energy. Most microorganisms are heterotrophic.

Horizontal Travel Time	The time that a water volume requires to travel in a horizontal direction through an aquifer from a fecal contamination source to a point of concern.
Hue (Soils)	The dominant spectral color, one of the three variables of soil color defined within the Munsell system of classification.
Humus	The well decomposed, more or less stable part of the organic matter in soil.
Hydration	The physical binding of water molecules to ions, molecules, particles or other matter.
Hydraulic Conductivity	A measure of the ease of flow through a porous media such as soil. The constant of proportionality, K, in the Darcy law of fluid flow through porous media. K is a function of the porous media, the soil moisture tension, and the fluid properties, and has units of velocity. It can also be defined as the one-dimensional flow rate through a unit area at a unit hydraulic gradient.
Hydraulic Head	The elevation of a free surface of water above or below a reference datum. Also see Pump Heads.
Hydrolysis	Decomposition of a chemical compound by reaction with water, such as the dissociation of a dissolved salt or the catalytic conversion of starch to glucose. The chemical reaction of a compound with water, whereupon the anion from the compound combines with the hydrogen from the water and the cation from the compound combines with the hydroxyl from the water to form an acid and a base.
Hydrophobic	Repelling, tending not to combine with, or incapable of dissolving in water.
Illuviation	The process of deposition in an underlying soil layer of colloids, soluble salts, and mineral particles leached out of an overlying soil layer
Immobilization	The conversion of an element from the inorganic to the organic form in microbial or plant tissue, thus rendering the element not readily available to other organisms or plants.
Indicator Microorganisms	viruses and bacteria that may be non-pathogenic but are associated with fecal contamination and are transmitted through the same pathways as pathogenic viruses and bacteria.

Infectious Units	Either a single virus particle or a stable viral clump that is infectious for a living host system.
Infiltration	The entry into soil of water made available at the ground surface. The downward entry of water into the soil.
Infiltrometer	A device by which the rate and amount of water infiltration into the soil is determined.
Ion	An atom or a group of atoms that has acquired a net electric charge by gaining or losing one or more electrons. If an atom loses an electron, it becomes a positively charged ion termed a cation. If an atom gains an electron, it becomes a negatively charged ion termed an anion.
Isoelectric Point	The pH at which the electrical charge of an amphoteric particle becomes neutral.
Isotherm	See Absorption Isotherm.
Isotrophic	A condition where the hydraulic conductivity is independent of the direction of measurement at a point in a geologic formation.
Karst, Karstic	An area of irregular limestone in which erosion has produced fissures, sink holes, underground streams and caverns.
Kjeldahl Nitrogen	A term that reflects the technique used in determining the sum total of organic nitrogen and ammonium.
Labile	Constantly undergoing or likely to undergo change; unstable: a labile compound.
Lipid	Any of a group of organic compounds, including the fats, oils, waxes, sterols, and triglycerides, that are insoluble in water but soluble in common organic solvents, are oily to the touch, and together with carbohydrates and proteins constitute the principal structural material of living cells.
Land Treatment and Disposal	A system which utilizes soil materials for the treatment of domestic sewage and disposes of the effluent by percolation into the underlying soil and mixing with the ground water.
Latent	Present or potential but not evident or active.

Leach (Leaching)	To cause water or another liquid to percolate through something. The removal of materials in solution from the soil.
Limiting Horizon	Any horizon that limits the ability of the soil to provide treatment or disposal of septic tank effluent. Includes seasonal high water table, bedrock, hydraulically restricted or excessively coarse soil horizons and soil substrata.
Loam	As a soil textural class, soil material that contains 7% to 27% clay, 28% to 50% silt, and less than 52% sand.
Loamy	Intermediate in texture and properties between fine-textured and coarse-textured soils.
Loamy Sand	As a soil textural class, soil material that contains 70% to 90% sand, the remainder being silt and clay.
Longitudinal Dispersion	the spreading in the direction of ground water flow.
Lyse	To undergo or cause to undergo lysis.
Lysis	The dissolution or destruction of cells, such as blood cells or bacteria.
Lysimeter	A device for collection of liquid percolating through soil under controlled conditions.
Mass Transport	The distribution of contaminants in the subsurface via advection, dispersion and diffusion processes.
Mesophilic Bacteria	An organism whose optimum temperature for growth falls in an intermediate range of approximately 15° to 40° C.
Macrophyte	A macroscopic plant.
Macropore	Large soil pores, generally having a minimum diameter between 30 and 100 micrometers (μm), from which water drains readily by gravity.
Male Specific Coliphage	Viruses that attack coliform bacteria through the hair-like appendages extending from the cell walls of the bacteria. Designated as MS-Coliphage, or F^+ Coliphage.
Matric Potential	Attractive forces of soil particles for water and water molecules for each other.

Metabolism	The physical and chemical processes occurring within a living cell or organism that are necessary for the maintenance of life. In metabolism some substances are broken down to yield energy for vital processes while other substances, necessary for life, are synthesized.
Methemoglobin	A brownish-red crystalline organic compound formed in the blood when hemoglobin is oxidated either by decomposition of the blood or by the action of various oxidizing drugs or toxic agents. It contains iron in the ferric state and cannot function as an oxygen carrier.
Methemoglobinemia	A condition in which ferrous iron in hemoglobin is oxidized in the presence of nitrites to ferric iron, which converts hemoglobin (the blood pigment that carries oxygen from the lungs to tissue) to methemoglobin which is incapable of binding molecular oxygen. This condition prevents hemoglobin from carrying oxygen throughout the body and is particularly harmful to infants less than 6 months old because their stomachs are not yet acidic enough to prevent the growth of denitrifying bacteria that convert nitrate to nitrite.
Micrometer (µm)	One-millionth of a meter. (1 µm)
Microorganisms	An organism of microscopic or submicroscopic size, especially a bacterium or protozoan. Living organisms to small to be seen by the naked eye (<0.1 mm.); includes microscopic algae, bacteria, fungi, protozoans, and viruses. Also referred to as microbes.
Microfauna	Protozoa, nematodes and arthropods generally <200 micrometers long.
Microflora	Bacteria, fungi, algae, and viruses.
Micropore	Relatively small soil pore, generally found within structural aggregates and having a diameter <30 micrometers.
Mineral	A naturally occurring substance which has definite physical properties and chemical composition.
Mineral soil	A soil consisting predominantly of, and having its properties determined predominantly by, mineral matter. Usually contains less than 20% organic matter, but may contain an organic surface layer up to 30 cm thick.

Mineralize, Mineralization	To convert from an organic to a mineral substance. The conversion of a compound from an organic form to an inorganic form as a result of microbial decomposition.
Moderately-coarse texture	Consisting predominantly of coarse particles. In soil textural classification, it includes all the sandy loams except the very fine sandy loam.
Moderately-fine texture	Consisting predominantly of intermediate-size soil particles or with relatively small amounts of fine or coarse particles. In soil textural classification, it includes clay loam, sandy clay loam, and silty clay loam.
Mole (Chemistry)	An amount of a substance whose mass in grams numerically equals the formula weight of the substance.
Molal Solution	A solution containing one mole of solute in 1,000 grams of solvent.
Molar Solution	A solution that contains one mole of solute per liter of solution.
Molecular Weight	The sum of the atomic weights of all the atoms in a molecule.
Monosaccharide	A carbohydrate that cannot be decomposed by hydrolysis. Also called simple sugar.
Morphology	The branch of biology that deals with the form and structure of organisms without consideration of function. b. The form and structure of an organism or one of its parts. Also used in soil science to describe the form and structure of soils.
Motile, Motility	Moving or having the power to move spontaneously. Movement of a microbe under its own power.
Mottling, mottles	Spots or blotches of different color or shades of color, with both high and low chroma, interspersed with the dominant color of a soil. Redoximorphic features.
Munsell System (soil color)	A system of classifying soil color consisting of an alpha-numeric designation of hue, value and chroma, together with a descriptive color name, such as “strong brown”.
Nanometer (nm)	1×10^{-9} m (0.001 µm) One billionth of a meter.
Nemotode	An unsegmented, usually microscopic roundworm.

Nitrate-Nitrogen	The nitrogen in nitrates. 10 mg/l nitrate-nitrogen = 45 mg/l nitrate.
Nitrification	The transformation of ammonia nitrogen to nitrates.
Nitrogen Fixation -	The formation of ammonia from free atmospheric nitrogen.
NOx-N	Chemical compounds containing oxygen and nitrogen (nitrogen oxides), such as nitrites (NO_2-N), nitrates (NO_3-N), nitric oxide (NO), and nitrous oxide (N_2O).
Nutrient	Any substance that is assimilated by organisms and promotes growth.
Obligate	An adjective referring to an environmental factor (for example, oxygen) that is always required for growth. An organism that can grow and reproduce only by obtaining carbon and other nutrients from a living host.
Obligate Bacteria	Bacteria able to exist or survive only in a particular environment or by assuming a particular role. (Example: Nitrifying bacteria which use inorganic nitrogen compounds, rather than organic compounds, to supply their energy needs. Carbon compounds such as CO_2 are used for cellular synthesis reactions but not for energy producing reactions.
Oligotrophic	Lacking in nutrients, nutrient poor.
Oocyst	The infectious stage of *Cryptosporidium parvum* and some other coccidian parasites. An oocyst has a protective shell-like wall that facilitates its survival in water and other environments. The encysted stage in the life cycle of some protozoa. A metabolically dormant protective phase often exhibited by parasitic protozoa.
Order	A taxonomic category of organisms ranking above a family and below a class.
Organic matter	Chemical substances of basically carbon structure.
Organic soil material	soil containing 12 to 18 percent or more organic carbon by dry weight, depending upon the clay content.
Organic Nitrogen	Nitrogen combined in organic molecules, such as proteins and amino acids.
Ova	A reproductive cell, an egg.

Oxidation-Reduction

A chemical reaction in which one substance is reduced and another is oxidized.

Reduction is a chemical transformation involving a gain of electrons to an atom, molecule or ion. Usually, reduction involves the loss of oxygen; gain of hydrogen; or an increase in the proportion of a metal in a compound. Oxidation is a chemical transformation involving a loss of electrons from an atom, molecule or ion. Usually, oxidation involves the gain of oxygen; loss of hydrogen; or an increase in the proportion of a nonmetal in a compound.

An oxidizing agent oxidizes or acquires electrons from another material and is itself reduced. Reducing agents contribute electrons and are themselves oxidized. Electrons transfer from reducing agents to oxidizing agents. Energy is released by oxidation reactions. The chemical scheme of oxidation is believed to be the same for all microorganisms whether plant or animal. The differences between aerobic, facultative, and anaerobic bacteria lie in their mechanisms of hydrogen oxidation.

Organic matter in wastewaters is stabilized by oxidation. Bacteria and other microorganisms in waste stabilization systems do not oxidize matter by the direct addition of oxygen, but rather by the indirect scheme of hydrogen removal and addition of water. The hydrogen eventually reacts with oxygen, carbon, nitrogen or sulfur.
The growth and survival of microorganisms depends upon their ability to obtain energy from the system. Energy is required for the production of new protoplasm, for motility, and just to remain alive. Microorganisms obtain their energy from the metabolism of organic and inorganic compounds. The energy level of the organic matter being metabolized is reduced while the energy level of the cellular material is increased.

Removal of hydrogen from organic matter results in its oxidation, while addition of hydrogen to organic matter results in its reduction. In aerobic biological systems oxygen is the ultimate hydrogen acceptor. In anaerobic biological systems the hydrogen acceptors include any oxidized organic matter, nitrates, nitrites, sulfates and carbon dioxide.

Oxygen Reduction Potential (ORP)

A metal (gold or platinum) electrode immersed in water with a redox reaction occurring, or at least containing redox reaction products, will develop a potential related to the reaction.

Practical oxidation-reduction potential (ORP) measurement is simply the voltage measured between a metal ORP electrode and a reference electrode. ORP is measured in millivolts (mV), typically ±1000 mV.

Oxidizing chemicals have the ability to accept electrons; reducing chemicals have the ability to donate electrons. When present in a solution, the presence of an oxidizer will raise the ORP (oxidation/reduction potential), while a reducer's presence will lower the ORP.

Pans	Horizons or layers, in soils, that are strongly compacted, hardened, or very high in clay content. A natural subsurface soil layer of low or very low permeability, with a high concentration of small particles, and differing in certain physical and chemical properties from the soil immediately above or below the pan.
Pandemic	Epidemic over a wide geographic area: i.e. pandemic influenza.
Parasite	An organism that lives in or on another organism (the host) and gains benefit at the expense of the host. An organism that grows, feeds, and is sheltered on or in a different organism while contributing nothing to the survival of its host.
Pathogens	Microorganisms that cause disease. They include a few types of bacteria, viruses, protozoa and other organisms.
Pathogenesis	The development of a diseased or morbid condition.
Pathogenicity	Capable of causing disease.
Ped	The arrangement or grouping of individual soil particles into aggregates or clusters. A unit of soil structure such as an aggregate, crumb, prism, block, or granule, formed by natural processes (in contrast with a clod, which is formed artificially)
Pedology	The scientific investigation of soils.
Pedon	The smallest volume of a soil body which displays the normal range of variations in properties of a soil. A three-dimensional body of soil with lateral dimensions large enough to permit the study of horizon shapes and relations.

Percolation	The downward movement of excess water through soil; the water may or may not fill all of the soil pores.
Permeability	A measure of the rate of flow of a liquid or gas through a porous media. Permeability is a function of the porous media only, as compared to hydraulic conductivity, which is a function of both the porous media and the fluid (liquid or gas). Permeability has dimensions of $[L^2]$.
PFU	Plaque forming units. The presence of plaques, in an otherwise dense growth of bacteria susceptible to destruction by the selected virus, indicates that bacteria at the plaque locations have been destroyed and thus the presence of viable viruses.
pH (Soil)	The negative logarithm (base 10), of the hydrogen ion activity of a soil at a specified moisture content or soil-water ratio. The hydrogen ion activity in a soil solution is an index of soil acidity. A soil with a $pH<7$ is an acid soil.
Phosphates	Compounds of phosphorus and oxygen which occur in many forms, often combined with other elements such as sodium, calcium and potassium.
Phreatic Surface	The surface of the ground water, the water table.
Piezometer -	A device so constructed and sealed as to measure the static (hydraulic) head at a point in the subsurface.
Plaques	Small, clear, circular areas on a lawn of growing cells (monolayer) which result from the virus-induced deaths of groups of cells. The number of plaques indicated the concentration of virus particles.
Plinthic	Containing plinthite
Plinthite	An iron-rich, humus poor mixture of clay with quartz and other minerals. It commonly occurs as dark red redox concentrations that usually form platy, polygonal or reticulate patterns.
Polysaccharide	Any of a class of carbohydrates, such as starch and cellulose, consisting of a number of monosaccharides joined together.
Pore space	Portion of soil bulk volume occupied by soil pores.

Porosity	Volume of pores in a soil sample divided by the bulk volume of the sample.
Pump Heads	The head of water on the suction side of the pump is the static suction head. The height to which a pump must raise a liquid is the static discharge head. The head required to overcome the friction and form losses in a piping system at a given flow rate is the system head. The velocity head is the kinetic head imparted to the pumped liquid. The total head on a pump is the sum of all the various heads defined herein, expressed in similar units.
Precipitate	To cause (a solid substance) to be separated from a solution.
Predation	The capture of prey as a means of maintaining life.
Protein.	Any of a group of complex organic molecules that contain carbon, hydrogen, oxygen, nitrogen, and usually sulfur and are composed of one or more chains of amino acids. Proteins are fundamental components of all living cells and include many substances, such as enzymes, hormones, and antibodies, necessary for the proper functioning of an organism, including growth and repair of tissues.
Proton	A stable, positively charged subatomic particle.
Protoplasm	The living substance of a cell. The term usually refers to the substance enclosed by the cytoplasmic membrane, the protoplasm outside the nucleus of a cell.
Protozoa	Microscopic, usually single-celled microorganisms that live in water and are relatively large in comparison to other microorganisms. Protozoa eat bacteria and many are parisitic.
Recalcitrant	Resistant to microbial attack
Receiving layer	The natural soil under and around an effluent disposal area, beyond the biomat interface, which receives and provides additional treatment of the percolating wastewater before it reaches the ground water.
Redox Reactions	See Oxidation-Reduction.

Redoximorphic Features	Features formed by the process of reduction, translocation and/or oxidation of iron and manganese oxides. Can be categorized as redox depletions and redox concentrations (low and high chroma mottles, respectively) and a gleyed matrix.
Reduction	See Oxidation-Reduction.
Refractory	Resistant to treatment.
Regolith	The layer of loose rock resting on bedrock, constituting the surface of most land. Approximately equivalent to the term “soil”.
Respiration	The oxidative process occurring within living cells by which the chemical energy of organic molecules is released in a series of metabolic steps involving the consumption of oxygen and the liberation of carbon dioxide and water. Also, any of various analogous metabolic processes by which certain organisms, such as fungi and anaerobic bacteria, obtain energy from organic molecules.
Respirometer	An instrument for measuring the degree and nature of respiration.
Restrictive layer	A soil horizon that restricts the downward flow of water and is uncharacteristic of the soil layers above and below, such as a layer of soil with a consistence of firm or very firm, cemented horizons, or stratified layers of silt, loam or clay within the soil profile.
Risk	A compound measure describing the probability of an adverse event occurring and the severity of such an event. An evaluation of the health risks posed by the use of on-site wastewater disposal systems requires knowledge of (1) the presence of agents that cause disease, (2) the dose response characteristics of the agent involved, and (3) the manner in which the agent comes into contact with susceptible individuals. The risk of infection is defined as a mathematical probability of infectivity from a given dose or exposure
Risk Assessment	The process of making estimates that particular adverse events will occur in a given time period. There are four steps in a formal risk assessment: (1) hazard identification, (2) dose-response determination, (3) exposure assessment, and (4) risk characterization.

Risk Evaluation	A social value judgement as to what level of risk is acceptable.
RNA	Ribonucleic acid. A polymeric constituent of all living cells and many viruses. The structure and base sequence of RNA are determinants of protein synthesis and the transmission of genetic information.
Salt	A compound produced when an acid reacts with a base. Usually water is also produced. Salts are composed of anions and cations.
Sand	As a soil textural class, soil that contains 85% or more (by weight) of particle sizes between 0.05 and 2.0 mm.
Coarse Sand	Contains 25% or more (by weight) of soil that has a particle size between 0.5 and 2.0 mm.
Fine Sand	Contains 50% or more (by weight) of soil that has a particle size between 0.10 and 0.25 mm.
Sandy Loam	As a soil textural class, soil that contains silt and clay and between 43% and 52% (by weight) of sand.
Saprolite	Highly weathered (rotten) bedrock. Soft, partially decomposed rock rich in clay and remaining in its original place.
Saturated Zone	The soil zone in which all easily drained voids between soil particles are temporarily or permanently filled with water.
Saturated Hydraulic Conductivity	The hydraulic conductivity when all of the soil pores are filled with water.
Seasonal high water table	The upper limit of the seasonally high saturated zone.
Seep	To pass slowly through small openings or pores; ooze.
Self-supplied water	Water withdrawn from a source by a user rather than being obtained from a public water supply system.
Septage	The waste content found in a septic tank.
Serology	The medical science that deals with serums (clear yellowish fluids obtained by separating whole blood into its solid and liquid components).
Serotype	A group of closely related microorganisms distinguished by a characteristic set of antigens.
Sesquioxides	Metallic (e.g. Fe, Al, Mn) oxides in clay minerals or in soils.
Silicate	Any of numerous compounds containing silicon, oxygen, and one or more metals and sometimes hydrogen.

Silt	Soil particle with a diameter between 0.002 and 0.05 mm. As a soil textural class, soil material that contains more than 80& or more silt and less than 12% clay.
Slime layer (Bacterial)	Diffuse layer of polysaccharide exterior to the cell wall of some bacteria.
Smectitic Soils	Soils containing clay minerals that swell on wetting and close the larger pores in the soil system.
Soil Aeration	The process by which air in the soil is replaced by air from the atmosphere. The rate of aeration depends largely on the volume and continuity of air-filled pores within the soil.
Soil aggregates	A group of soil particles cohering so as to behave mechanically as a unit.
Soil Bulk Density	The mass of solids divided by the total volume of solids and voids.
Soil, Silt.	Individual mineral particles that range in diameter from 0.002 mm to 0.05 mm. As a soil textural class, soil that is 80 percent or more silt and less than 12 percent clay.
Soil Catena	Related soils of about the same age, derived from similar parent material and occurring under similar climatic conditions, arranged into a sequence of increasing wetness.
Soil, Clay	Soil particle <0.002 mm in diameter. Naturally occurring inorganic matter, largely of secondary origin. As a soil textural class, soil that is 40 percent or more clay, less than 45 percent sand, and less than 40 percent silt.
Soil, coarse textured	Sand or loamy sand.
Soil, moderately coarse Textured	Sandy loam and fine sandy loam.
Soil, medium textured	Very fine sandy loam, loam, silt loam, or silt.
Soil, fine textured	Sandy clay, silty clay and clay.
Soil Horizon	A distinct layer of soil running approximately parallel to the soil surface, having distinct characteristics produced by soil-forming processes. Alphabetically designated, but not necessarily in alphabetical order, proceeding vertically through the soil profile from the soil surface downward.

O horizon	A soil horizon at the surface, formed from organic litter derived from plants and animals, which overlies mineral soils.
A horizon	A soil horizon formed at or near the surface, but within the mineral soil, having properties that reflect the influence of accumulating organic matter or the removal of soil material in suspension, alone or in combination. Also a plowed surface horizon most of which was originally part of the B-horizon. Commonly referred to as the topsoil.
E horizon	An eluvial, mineral soil horizon in which the main feature is loss off silicate clay, iron, or aluminum,, or some combination of these, leaving a concentration of sand and silt particles.
AB horizon	A transitional horizon between the A and B horizons.
B-horizon	A soil horizon immediately beneath an A, E, or O horizon characterized by a higher colloid (clay or humus) content. Commonly referred to as the subsoil. The alluviated B horizons are layers that are rich in deposited iron, aluminum and other minerals that were leached out of the A horizons.
B2 horizon	That part of the B horizon where the properties on which the B horizon is based are without clearly expressed subordinate characteristics indicating that the horizon is transitional to an adjacent overlying or underlying horizon.
Solum	The upper and most weathered part of the soil profile; the O, E and B horizons. The soil beneath the solum is defined as the substratum.
C-horizon	The unconsolidated mineral soil horizon than normally lies beneath the solum. This horizon is outside the zone of major biological activity and has been influenced only slightly by soil-forming processes. If the C horizon material is similar in properties to the soils in the solum, it is referred to as unaltered or slightly altered parent material.
Cd horizon	A dense, compact, brittle horizon which is nearly impervious to water. Commonly associated with basal till.
Soil Macropores	Large soil pores, generally having a minimum diameter between 30 and 100 micrometers (μm), from which water drains readily by gravity.

Soil, massive	Soil with no structural units, a coherent mass (not necessarily cemented).
Soil matrix	The natural soil material composed of both mineral and organic matter.
Soil matric potential	Portion of the total soil water potential due to the attractive forces between water and soil solids as represented through adsorption and capillarity.
Soil Micropore	Relatively small soil pore, generally found within structural aggregates and having a diameter <30 micrometers.
Soil Moisture	See Water Content.
Soil Moisture Tension	The equivalent negative pressure in the soil water.
Soil Morphology	The physical constitution, particularly the structural properties, of a soil profile as exhibited by the kinds, thickness, and arrangement of the horizons in the profile, and by the texture, structure, consistence and porosity of each horizon.
Soil Particle Size Descriptors	
D	Diameter of soil particles determined by a sieve or hydrometer analysis.
D_{10}	Diameter of particles such that 10%(by weight) of the sample is smaller than that size. Also referred to as the “effective size”.
D_{60}	Diameter of particles such that 60%(by weight) of the sample is smaller than that size.
C_u	Coefficient of Uniformity, the ratio of D_{60}/D_{10}.
Soil Profile	A vertical cross section of the undisturbed soil showing the characteristic soil horizontal layers or soil horizons that have formed as a result of the combined effects of parent material, topography, climate, biological activity, and time.
Soil Series	The basic unit of soil classification, consisting of soils, which are essentially alike in all major profile characteristics, although the texture of the A-horizon may vary somewhat.

Soil solution	Aqueous liquid phase of the soil and its solutes, consisting of ions dissociated from the surfaces of the soil particles and of other soluble materials.
Soil texture	The visual or tactile surface characteristics of soil. The distribution, on a percent by weight basis, of sand, silt, and clay.
Soil structure	The arrangement or grouping of individual soil particles into aggregates or clusters. The principal forms of soil structure are blocky (angular or sub-angular), columnar (prisms with rounded tops), granular, platy (laminated), and prismatic (vertical axis of aggregates longer than horizontal). Structure-less soils are either single grained (each grain by itself) or massive (the particles adhering without any regular cleavage, as in many hardpans).
Soil Water (soil moisture)	Water present in the soil pores in an unsaturated zone above the ground water table.
Soil water potential (total)	The amount of energy that must be expended to extract water from soil. The total potential of soil water consists of gravitational potential, matric potential, and osmotic potential.
Solum	The upper and most weathered part of the soil profile; the A and B horizons.
Solute	A substance dissolved in another substance.
Somatic Coliphage	Viruses that attack fecal coliform bacteria through their cell walls.
Sorb	To take up and hold, as by absorption or adsorption.
Sorption	The transfer of ions (molecules with positive or negative charges) from the solution phase (water) to the soil (solid phase). Sorption actually describes a group of processes, which include adsorption and precipitation reactions.
Species	A fundamental category of taxonomic classification, ranking below a genus or subgenus and consisting of related organisms capable of interbreeding. A collection of closely related strains sufficiently different from all other strains to be recognized as a distinct unit.

Specific Yield	The volume of water that an unconfined aquifer releases from storage per unit surface area of aquifer per unit decline in the water table.
Spore	A dormant, non-reproductive body formed by certain bacteria, protozoa and fungi in response to adverse environmental conditions. A small, usually single-celled, reproductive body that is highly resistant to desiccation and heat and is capable of growing into a new organism under favorable environmental conditions.
Sporocyst	A protective case containing spores of certain protozoans.
Sterilization	Free from live bacteria or other microorganisms.
Strain	A group of organisms of the same species, having distinctive characteristics but not usually considered a separate breed or variety.
Stratified Drift	Sorted sediments (layers of sand and gravel and lesser amounts of silt and clay) deposited by glacial meltwaters.
Streptobacilli	Bacilli arranged in chains of cells.
Streptococci	Cocci that divide in such a way that chains of cells are formed.
SWAS	Subsurface wastewater absorption system. Also referred to as a subsurface soil absorption system (SSAS).
Substrate	The material or substance on which an enzyme acts. Also, a surface on which an organism grows or is attached.
Supernatant	The clear fluid above a sediment or precipitate.
Symbiosis	Close prolonged associations between two or more different organisms of different species that may, but does not necessarily, benefit each member.
Taxonomy	The classification of organisms in an ordered system that indicates natural relationships.
Tensiometer	a device used to measure in situ the negative hydraulic pressure (or tension) with which water is held in the soil; a porous, permeable cup connected through a tube to a manometer or vacuum gage.
Terminal Acceptor	A chemical substance that accepts the hydrogen ion that has been removed from another substance during the

	biological oxidation process. Strict aerobes utilize free dissolved oxygen as their ultimate hydrogen acceptor, while strict anaerobes use chemically bound oxygen, carbon, nitrogen or sulfur as their hydrogen acceptor. The facultative bacteria can use most of the above mechanisms, but will always use the hydrogen acceptor yielding the greatest energy. Also see Electron Acceptor.
Threshold Moisture Content	The minimum moisture condition, measured either in terms of moisture content or moisture stress, at which biological activity just becomes measurable.
Till	See Glacial Till.
Titer	The concentration of a substance in solution. Often expressed as the reciprocal of dilution. A solution diluted to 1:256 is said to have a titer of 256. The concentration of viruses in a given volume of liquid.
Tortuosity	The non-straight nature of soil pores.
Total Nitrogen	The sum total of Kjeldahl nitrogen, nitrites and nitrates.
Total Organic Carbon (TOC)	The organic carbon in water and wastewater that includes those organic compounds that can be oxidized by biological (BOD test) and chemical (COD test) processes and those organic compounds that do not respond to those processes. TOC tests convert organic carbon to CO_2, which is then measured.
Travel Time	The time of travel of a contaminant from the contaminant source to a point of concern.
Transpiration	A process by which plants transfer water from the root zone to the leaf surface, where it eventually evaporates into the atmosphere.
Transverse Dispersion	The spreading in the direction perpendicular to the ground water flow.
Ultraviolet (UV) Light	Electromagnetic radiation with a wavelength between 175 and 350 nm (nanometers)-shorter than visible light. Certain wavelengths absorbed by nucleic acids result in mutation and death.
Unsaturated Zone	The soil zone above the water table in which the soil pores are not all filled with water and the pressure is less than atmospheric.

Uniformity Coefficient	The ratio of the percent by weight of the soil particles that pass the No. 60 standard mesh sieve to the percent by weight of soil particles that pass the No. 10 sieve. (d_{60}/d_{10})
Vadose Zone	The vadose zone is that portion of the soil between the ground surface and the water table and includes the capillary fringe.
Valency	The number of hydrogen atoms that combine with, or replace, one atom of the element. Hydrogen has a valency of one.
Value (soil color)	The relative lightness or intensity of a color, one of the three variables of soil color defined within the Munsell system of classification.
London-Van der Waal Force	An atomic cohesive force, existing between all atoms. Generally considered to operate within distances on the order of atomic dimensions. In the case of colloidal particles, the aggregate effect of the attractive forces is to extend the range of effectiveness to the order of colloidal dimensions.
Virion	A complete viral particle, consisting of RNA or DNA surrounded by a protein shell and constituting the infective form of a virus.
Virus n., pl. viruses.	Any of various simple submicroscopic parasites of plants, animals, and bacteria that often cause disease and that consist essentially of a core of RNA or DNA surrounded by a protein coat. Viruses are unable to replicate without a host cell and are typically not considered living organisms, but are sometimes referred to as being on the "threshold of life".
Virulence (Viral)	The disease: infection ratio.
Volatilization	The evaporation or changing of a substance from a liquid to a vapor.
Wastewater	Water and human excretions or other waterborne wastes incidental to the occupancy of a residential building or a non-residential building but not including manufacturing process water, cooling water, wastewater from water softening equipment, commercial laundry wastewater, blowdown from heating or cooling equipment, water from cellar or floor drains or surface water from roofs, paved surfaces or yard drains.

Water Content (soil)	The unit volume of water in a unit bulk volume of the soil. Expressed as a percentage or decimal fraction. Also referred to as the soil moisture content.
Water Table	The upper surface of the ground water or that level in the ground where the water is at atmospheric pressure. The upper surface of a zone of saturation.
Water Table, perched	The water table of a saturated layer of soil, which is separated from an underlying saturated layer by an unsaturated layer.
Water Year	The period from October 1st of one calendar year to September 30th of the following calendar year.
Wild Viruses	Viruses recovered from the environment or from an infected individual.
Wilting Point	The minimum quantity of water in a given soil necessary to maintain plant growth. When the quantity of moisture falls below the wilting point, the leaves begin to drop and shrivel up.

APPENDIX F

PHOSPHORUS SORPTION ISOTHERM DETERMINATION

Phosphorus Sorption Isotherm Determination

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Introduction:

Phosphorus (P) retention by soils is an important parameter for understanding soil fertility problems, as well as for determining the environmental fate of P. The P adsorption capacity of a soil or sediment is generally determined by batch-type experiments in which soils or sediments are equilibrated with solutions varying in initial concentrations of P. Equations such as the Langmuir, Freundlich and Tempkin models have been used to describe the relationship between the amount of P adsorbed to the P in solution at equilibrium (Berkheiser et al., 1980; Nair et al., 1984).

Advantages of the batch technique include: the soil and solution are easily separated, a large volume of solution is available for analysis, and the methodology can be easily adapted as a routine laboratory procedure. Disadvantages include difficulties in measuring the kinetics of the sorption reaction and optimizing the mixing of solution and soil without particle breakdown (Burgoa et al. 1990). Despite the disadvantages, the batch technique has been, and still is, widely used to describe P sorption in soils and sediments.

Nair et al. (1984) noted that P sorption varies with soil/solution ratio, ionic strength and cation species of the supporting electrolyte, time of equilibration, range of initial P concentrations, volume of soil suspension to head space volume in the equilibration tube, rate and type of shaking, and type and extent of solid/solution separation after equilibration. Although most researchers use a similar basic procedure for measuring P adsorption, there is considerable variation observed among studies with regard to the above parameters. This variation often makes comparisons of results among studies difficult. Thus, Nair et al. (1984) proposed a standard P adsorption procedure that would produce consistent results over a wide range of soils. This procedure was evaluated, revised, tested among laboratories and was eventually proposed as a standardized P adsorption procedure. This procedure as described below is proposed as the standard procedure recommended by the SERA-IEG 17 group.

Equipment:

1. Shaker: End-over-end type
2. Filter Apparatus: Vacuum filter system using 0.45 or 0.2 μm filters
3. Equilibration tubes: 50 mL or other size to provide at least 50% head space
4. Spectrophotometer: Manual or automated system capable of measuring at 880 nm

Reagents:

1. Electrolyte: 0.01 M $CaCl_2$, unbuffered
2. Microbial inhibitor: Chloroform
3. Inorganic P solutions: Selected concentrations as KH_2PO_4 or NaH_2PO_4 (in 0.01 M $CaCl_2$ containing: 20 g/L chloroform)

Procedure:

1. Air-dry soil samples and screen through a 2 mm sieve to remove roots and other debris.
2. Add 0.5 to 1.0 g air-dried soil to a 50 mL equilibration tube.
3. Add sufficient 0.01 M $CaCl_2$ solution containing 0, 0.2, 0.5, 1, 5, and 10 mg P/L as KH_2PO_4 or NaH_2 PO_4, to produce a soil: solution ratio of 1:25. The range of P values could vary from 0 to 100 mg P/L (0, 0.01, 0.1, 5, 10, 25, 50 and 100 mg P/L) and the soil/solution ratio could be as low as 1:10 depending on the sorbing capacity and the P concentrations of the soils in the study.
4. Place equilibration tubes on a mechanical shaker for 24 h at 25 ± 1   C.
5. Allow the soil suspension to settle for an hour and filter the supernatant through a 0.45 μm membrane filter.
6. Analyze the filtrate for soluble reactive P (SRP) on a spectrophotometer at a wavelength of 880 nm.

Calculations and Recommended Presentation of Results:

Two of the often used isotherms are the Langmuir and the Freundlich isotherms; the Langmuir having an advantage over the Freundlich in that it provides valuable information on the P sorption maximum, S_{max} and a constant k, related to the P bonding energy.

The Langmuir equation
The linearized Langmuir adsorption equation is:

$$C/S = 1/kS + C/S_{max}$$

where:

$S = S' + S_o$, the total amount of P retained, mg/kg
S' = P retained by the solid phase, mg/kg
S_o = P originally sorbed on the solid phase (previously adsorbed P), mg/kg
C = concentration of P after 24 h equilibration, mg/L
S_{max} = P sorption maximum, mg/kg, and
k = a constant related to the bonding energy, L/mg P.

The Freundlich equation
The linear form is: $\log S = \log K + n \log C$
where:
K is the adsorption constant, expressed as mg P/kg,
n is a constant expressed as L/kg, and
C and S are as defined previously.

A plot of log S against log C will give a straight line with log K as the intercept, and n as the slope.

Previously adsorbed P (also referred to as native sorbed P)
Adsorption data should be corrected for previously adsorbed P (S_o). For the calculation of previously sorbed P, Nair et al. (1984) used isotopically exchangeable P (Holford et al., 1974) prior to calculations by the Langmuir, Freundlich and Tempkin procedures. Other procedures used to calculate the previously adsorbed P include oxalate-extractable P (Freese et al., 1992; Yuan and Lavkulich, 1994), anion-impregnated membrane (AEM) technology (Cooperband and Logan, 1994) and using the least squares fit method (Graetz and Nair, 1995; Nair et al., 1998; Reddy et al., 1998). Sallade and Sims (1997) used Mehlich 1 extractable P as a measure of previously sorbed P.

Investigations by Villapando (1997) have indicated a good agreement among native sorbed P values estimated by the least squares fit method, oxalate extractions, and the AEM technology. At this point, it appears that selection of the method for determination of native sorbed P would depend on the nature of the soils in the study and reproducibility of the results.

The procedure for calculation of S_o using the least square fit method is based on the linear relationship between S' and C at low equilibrium P concentrations. The relationship can be described by

$$S' = K' C - S_o$$

Where

K'= the linear adsorption coefficient, and all other parameters are as defined earlier. (Note: It is recommended that the linear portion of the isotherm has an r^2 value 0.95 or better).

Equilibrium P Concentration
The “equilibrium P concentration at zero sorption” (EPC_0) represents the P concentration maintained in a solution by a solid phase (soil or sediment) when the rates of P adsorption and desorption are the same (Pierzynski et al., 1994). Values for EPC_0 can be determined graphically from isotherm plots of P sorbed vs. P in solution at equilibrium. From the calculations given above, EPC_0 is the value of C when S' = 0.

Comments:

The above procedure was developed to provide a standardized procedure with a fixed set of conditions that could be followed rigorously by any laboratory. The procedure uses a low and narrow range of dissolved inorganic P concentrations because these are the concentrations likely to be encountered in natural systems and because higher concentrations may result in precipitation of P solid phases. However, higher concentrations of P (up to 100 mg/L) and/or lower soil:solution ratios (1:10) have been used for isotherm determinations on soils and sediments (Mozaffari and Sims, 1994; Sallade and Sims, 1997; Nair et al., 1998; Reddy et al., 1998). A 0.01 M KC1 solution may be used as the background electrolyte to avoid precipitation of Ca in neutral and alkaline soils.

Toluene and chloroform have been shown to increase the dissolved P concentration in the supernatant, apparently due to lysis of microbial cells, and thus, some researchers do not try to inhibit microbial growth (Reddy et al., 1998).

Most adsorption studies are conducted under aerobic conditions, however, with certain studies it is more appropriate to use anaerobic conditions, as they more closely represent the natural environments of the soils or sediments. Reddy et al. (1998) preincubated sediment/soil samples in the dark at 25º C under a N_2 atmosphere, to create anaerobic conditions. Adsorption experiments were then conducted, performing all equilibrations and extractions in an O_2-free atmosphere.

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APPENDIX G

CONNECTICUT PUBLIC HEALTH CODE

REGULATIONS AND TECHNICAL STANDARDS
FOR SUBSURFACE SEWAGE DISPOSAL SYSTEMS

Page 27, Section IV - Design Flows
Pages 28-32, Section V – Septic Tanks

CONNECTICUT PUBLIC HEALTH CODE

Regulations and Technical Standards

for Subsurface Sewage Disposal Systems

Section 19-13-B100a (Building Conversions, Changes in Use, Additions)
Effective August 3, 1998

Section 19-13-B103 (Discharges 5,000 Gallons Per Day or Less)
Effective August 16, 1982

Technical Standards (Pursuant to Section 19-13-B103)
Effective August 16, 1982
Revised January 1, 1986
Revised January 1, 1989
Revised January 1, 1992
Revised January 1, 1994
Revised January 1, 1997
Revised January 1, 2000
Revised January 1, 2004

Section 19-13-B104 (Discharges Greater than 5,000 Gallons Per Day)
Effective August 16, 1982

State of Connecticut
Department of Public Health
Environmental Engineering Program
410 Capitol Avenue - MS #51SEW
P.O. Box 340308
Hartford, Connecticut 06134

www.dph.state.ct.us/BRS/Sewage/sewage_program.htm

January 2004

IV. DESIGN FLOWS

RESIDENTIAL BUILDINGS: 150 Gallons per Day per Bedroom

NON-RESIDENTIAL BUILDINGS and RESIDENTIAL INSTITUTIONS: Table No. 4 shall be used for determining the daily design flow from non-residential buildings and residential institutions unless specific water use data is available for the facility. Design flow based on metered flows must use a minimum 1.5 safety factor applied to all metered average daily water use.

TABLE NO. 4

SCHOOLS, PER PUPIL	**GALLONS PER DAY**
BASE FLOW (EXCLUDES KITCHEN & SHOWERS)	
HIGH SCHOOL	12
JR. HIGH/MIDDLE SCHOOL	9
KINDERGARTEN/ELEMENTARY SCHOOL	8
KITCHEN	3
SHOWERS	3 to 5
RESIDENTIAL	100
DAY CARE CENTER (NO MEALS PREPARED)	10
COMMERCIAL/INDUSTRIAL BUILDINGS, PER EMPLOYEE	
FACTORY (NO SHOWERS)	25
FACTORY (WITH SHOWERS)	35
OFFICE (AVERAGE 200 SQ.FT./PERSON-GROSS AREA)	20
SMALL RETAIL BUILDING-LESS THAN 2,000 SQ.FT.-GROSS AREA	20
LARGE RETAIL/COMMERCIAL BUILDING-SEE MISCELLANEOUS	
CAMPS	
RESIDENTIAL CAMPS (SEMI PERMANENT), PER PERSON	50
CAMPGROUND WITH CENTRAL SANITARY FACILITIES, PER PERSON	35
CAMPGROUND WITH FLUSH TOILETS (NO SHOWERS), PER PERSON	25
CAMPGROUNDS PER CAMP SPACE (WATER AND SEWER HOOK-UPS)	75
DAY CAMPS, PER PERSON	15
LUXURY CAMPS, PER PERSON	75
PICNIC PARKS (TOILET WASTES ONLY), PER PERSON	5
PICNIC PARKS WITH BATHHOUSES, SHOWERS, FLUSH TOILETS, PER PERSON	10
HEALTH CARE FACILITIES	
HOSPITALS, PER BED	250
REST HOMES, PER BED	150
CONVALESCENT HOMES, PER BED	150
INSTITUTIONS, PER RESIDENT	100
GROUP HOME, PER CLIENT (LARGE TUB/ON-SITE LAUNDRYING USE HIGHER FLOW)	100-150
RESTAURANTS	
RESTAURANTS (PUBLIC TOILETS PROVIDED), PER MEAL SERVED	10
TAKE OUT FOOD SERVICE/RESTAURANTS WITH NO PUBLIC TOILETS, PER MEAL SERVED	5
BARS AND COCKTAIL LOUNGES (NO MEALS) PER PATRON	5
RECREATIONAL FACILITIES	
SWIMMING POOLS, PER BATHER	10
INDOOR TENNIS COURTS, PER COURT	400
OUTDOOR TENNIS COURTS, PER COURT	150
THEATERS, SPORTING EVENTS, PER SEAT	3.5
CHURCHES	
WORSHIP SERVICE ONLY, PER SEAT	1
SUNDAY SCHOOL, PER PUPIL	2
SOCIAL EVENTS (MEALS SERVED) PER PERSON	5
MISCELLANEOUS	
AUTO SERVICE STATIONS, PER CARS SERVICED	5
BEAUTY SALON, PER CHAIR	200
BARBER SHOPS, PER CHAIR	50
DENTAL/MEDICAL OFFICES WITH EXAMINATION ROOMS, PER SQ. FT. OF GR. AREA	0.2
KENNEL DOG RUNS, PER RUN, ROOF MUST BE PROVIDED	25
LARGE RETAIL/COMMERCIAL BLDG., PER SQ. FT. OF GROSS AREA	0.1
LAUNDROMATS, PER MACHINE	400
MOTELS, PER ROOM, (NO FOOD SERVICE, KITCHENETTE OR LAUNDRY FACILITIES)	75
MOTELS, PER ROOM, (WITH KINCHENETTE BUT NO LAUNDRY FACILITIES)	100
MARINAS (BATHHOUSE-SHOWERS PROVIDED), PER BOAT SLIP	20

V. SEPTIC TANKS

A. General

1. Septic Tank Standards

All subsurface sewage disposal systems shall be provided with a septic tank. Such septic tank shall be made of concrete or other durable material approved by the Commissioner of Public Health.

a) Concrete Septic Tanks

All concrete septic tanks shall be produced with a minimum 4,000-psi concrete with 4 to 7 percent air entrainment. Concrete tanks must not be shipped until the concrete has reached the 4,000-psi compressive strength. Concrete septic tank construction shall conform to ASTM C 1227 with the following exceptions:

- There shall be no maximum liquid depth.
- The air space above the liquid level shall be a minimum of eight inches.
- Inspection ports over the compartment wall shall be optional.

b) Non-Concrete Septic Tanks

All non-concrete septic tanks shall meet all of the applicable requirements set forth in subsections 2, 3, and 4 of Standard V A regarding tank configuration, tank access, and tank cleaning. Non-concrete tanks shall be marked with the manufacturer's name and tank designation number. Non-concrete septic tanks shall be installed with strict adherence to the manufacturer's installation instructions in order to avoid tank damage or tank deformation. Proper bedding, backfill, and compaction shall be confirmed with each tank installation. Shallow groundwater conditions may prohibit installation of certain tanks due to tank design limitations or warranty restrictions. Tank bottoms located below maximum groundwater levels must be provided with anti buoyancy/floatation provisions (check with manufacturer). Manufacturers of non-concrete septic tanks shall file specifications and technical support documentation with the Commissioner of Public Health. The Commissioner of Public Health shall maintain a list of approved non-concrete septic tanks. The approved list as of the date of this revision has been provided in Appendix D.

2. Tank Configuration

All septic tanks shall contain an inlet baffle submerged for a depth of eight to eighteen inches and an outlet baffle (unless tank is provided with an approved outlet filter) submerged to a depth of at least ten inches, but no lower than 40 percent, of the liquid depth. The inlet baffle shall encompass not more than 48 square inches of liquid surface area. All baffles shall extend a minimum of five inches above the tank's liquid level and an air space of at least a 1/2-inch shall be provided above the baffle. The outlet invert of the septic tank shall be 3 inches lower than the inlet invert. Tanks must be installed with the inlet invert between 2 and 4 inches above the outlet invert. Inlet and outlet piping entering and exiting the septic tank shall be as level as possible with a pitch no greater than 1/4-inch per foot. The outlet invert of the tank shall be set at a higher elevation than the top of all leaching structures (except in a pump systems), or in the case of serial systems higher than the high-level overflow elevation of the upper most trench. All newly installed tanks shall have an approved non-bypass effluent filter at the outlet. The Commissioner of Public Health shall maintain a list of approved outlet filters. The approved list as of the date of this revision has been provided in Appendix B.

All septic tanks (except tanks in series) shall have two compartments with 2/3 of the required capacity in the first compartment (see Figure No. 4). The transfer port must be at mid-depth (opening in middle 25% of liquid depth). Inlet and outlet piping shall be sealed with a polyethylene gasket or rubber boot with stainless steel clamp. All septic tanks shall be manufactured with manhole covers or risers that have been placarded with notification of its two-compartment construction and a warning that "Entrance into the tank could be fatal". The minimum liquid depth of septic tanks shall be thirty-six inches.

Additional septic tank capacity over one thousand gallons may be obtained by utilizing two tanks in series. In no case may more than two septic tanks be placed in series. When two septic tanks are placed in series, each tank shall be of single compartment design; the volume of the first tank shall be twice the volume of the second; mid-depth baffles shall be provided at the connection of the two tanks; an outlet filter shall be provided for the outlet of the second tank (see Figure No. 5).

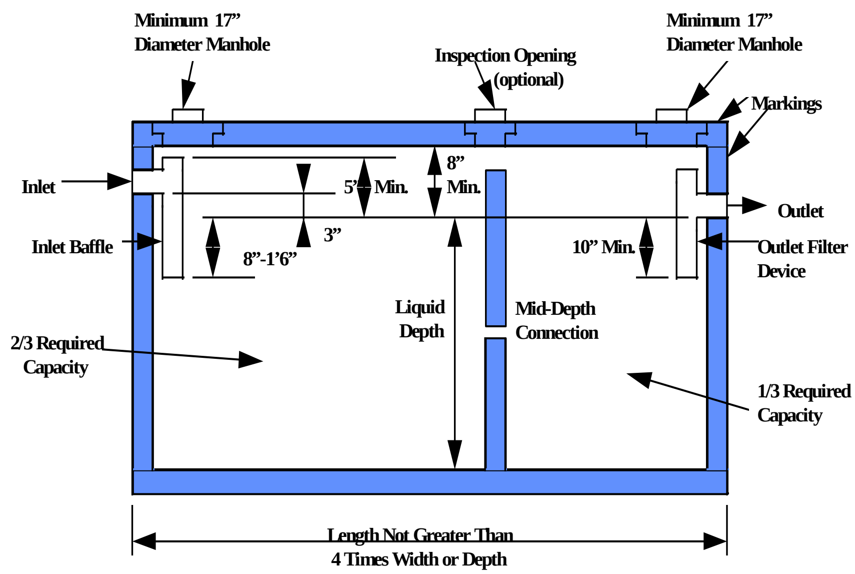


FIGURE NO. 4 - TYPICAL SEPTIC TANK

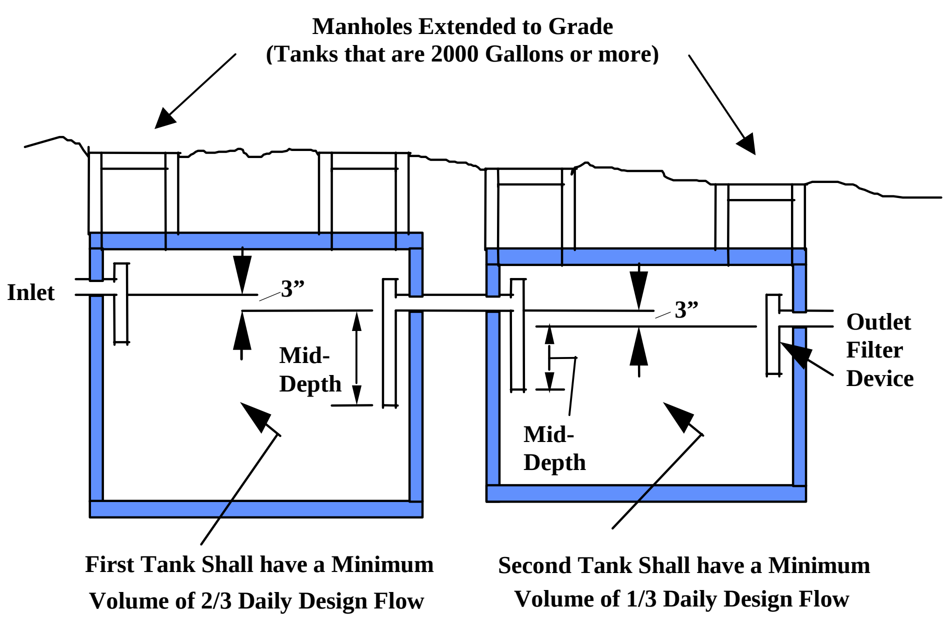
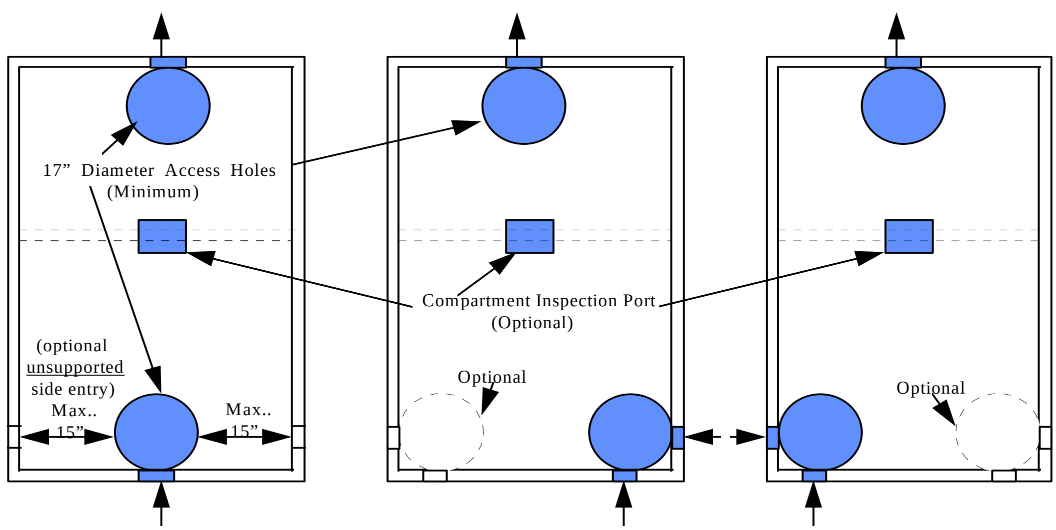


FIGURE NO. 5 - SEPTIC TANKS IN SERIES

3. Septic Tank Access

Septic tanks shall have removable covers or manholes to provide access to the tank for the purposes of inspection and cleaning. Cleanout manholes shall be located at a depth not greater than twelve inches below final grade level. Existing tanks that exceed the 12-inch depth shall be retrofitted with a cleanout riser(s) at the time of tank cleaning. New tanks and existing tanks deeper than 24 inches below finish grade shall be provided with large (24-inch minimum inside diameter) access risers over each manhole opening. Cleanouts shall consist of a minimum 17-inch inside diameter opening and shall be located directly over the inlet baffle and outlet filter. If a tank provides side inlets, the maximum distance between the interior wall surface and the cleanout manhole shall be 15 inches unless the pipe extension from the tank side to the cleanout manhole opening will be supported. Baffle extensions shall not have more than a 1/4-inch per foot pitch. All tank covers shall be stepped and be provided with handles consisting of 3/8-inch coated rebar or approved plastic handles. Below ground plastic handles and plastic riser covers cannot be used unless provisions are made to allow for manhole locating with a metal detector. On septic tanks of two thousand gallons or more, manholes shall extend to grade except for single-family residential buildings. Where covers are flush with or above grade, either the lid must weigh a minimum of 59 pounds or the cover shall be provided with a lock system to prevent unauthorized entrance. Tanks that exceed fifteen feet in length shall provide a minimum of three manholes. In any case, the overall length shall not be greater than four times either the width or the depth.



TYPE “C” TANK
CENTER ENTRY

TYPE “R” TANK
RIGHT ENTRY
(OPTIONAL)

TYPE “L” TANK
LEFT ENTRY
(OPTIONAL)

STANDARDIZED SEPTIC TANK TOP CONFIGURATIONS

4. Septic Tank Cleaning

Septic tanks shall be cleaned as often as necessary to prevent a buildup of sludge, grease and scum which will adversely effect the performance of the leaching system. In a properly functioning subsurface sewage disposal system, effluent should not backflow from the leaching system into the septic tank at the time of pumping. Such conditions indicate the leaching system is surcharged at that time. In these instances, further system evaluation is warranted. Inlet and outlet baffles shall be inspected for damage or clogging at the time of the tank pump out. When provided, outlet filters shall be properly cleaned, at the time of each tank pump out, by washing the filter waste into the septic tank or, if rinse water is not available, exchanged with a clean filter. All contaminated filters shall be treated as sewage and handled properly during the cleaning and/or exchange process.

5. Septic Tank Markings

Tank information (size, date manufactured, name of manufacturer and indication of limit of external loads/cover depths required by Section 13 of ASTM C 1227) shall be located on the top of the tank between the outlet access hole and outlet wall or on the vertical outlet wall between the top of the tank and the top of the outlet opening.

6. Performance Testing

When necessary due to installation concerns, testing for leakage will be performed using either a vacuum test or water-pressure test.

<u>Vacuum Test</u>: Seal the empty tank and apply a vacuum to 4 in. (50 mm) of mercury. The tank is approved if 90% of vacuum is held for 2 minutes.

<u>Water-Pressure Test</u>: Seal the tank, fill with water, and let stand for 24 hours. Refill the tank. The tank is approved if the water level is held for 1 hour.

7. Tank Abandonment

Abandonment of septic tanks, or hollow leaching structures, shall be performed in such a manner as to eliminate the danger of the structure inadvertently collapsing. The responsibility for abandonment lies with the property owner. When hollow structures are abandoned the chamber shall be emptied of all septage wastes, and the structure shall be filled with clean sand and gravel, or the structure shall be crushed and the area backfilled with clean soil.

B. <u>Septic tank capacities</u>

1. The minimum liquid capacity of septic tanks serving residential buildings shall be based on the number of bedrooms in the building. For three bedrooms or less, a 1000-gallon tank is required; and another 250-gallons shall be added for each additional bedroom above three.

2. The minimum liquid capacity of septic tanks serving non-residential buildings and residential institutions shall be equal to the 24-hour design flow (see Table No. 4). In no case shall a septic tank be installed with a liquid capacity of less than one thousand gallons. In cases of non-residential buildings that are subject to high peak sewage flows, the liquid capacity of the septic tank shall provide a minimum detention time of 2 hours under peak flow conditions. The required septic tank capacity shall be increased by a minimum of 50% at food service establishments and restaurants in instances of repairs of existing subsurface sewage disposal systems where it is determined that it is not feasible to install a grease interceptor tank or internal grease recovery unit.

3. Whenever more than 25 percent of the daily design flow from a building served will be pumped into the septic tank, the size of the tank shall be increased 50 percent beyond the minimum capacity required per Standard V B.

4. The liquid capacity of a septic tank shall be increased whenever a residential building contains a garbage grinder or large capacity bathtub in accordance with the following:

<u>Garbage grinder:</u>
Add 250 gallons to required capacity of the septic tank.

<u>Large tub</u>
100 to 200 gallon tub: Add 250 gallons to required capacity of the septic tank
Over 200 gallon tub: Add 500 gallons to required capacity of the septic tank.

C. Grease interceptor tanks

Grease interceptor tanks shall be provided for restaurants and food service establishments with design flows of 500 gallons per day or greater for new construction and repairs of existing subsurface sewage disposal systems where feasible. If it is determined that it is not feasible to install a grease interceptor tank on a food service/restaurant system repair, a mechanical grease recovery unit (GRU) is recommended to be retrofitted on the internal wastewater piping in the kitchen. If a grease interceptor tank or an internal GRU is not included in a food service/restaurant septic system repair, then the required septic tank capacity shall be increased by a minimum of 50% (see Standard V B).

Grease interceptor tanks shall receive wastewater from the kitchen waste lines only. Effluent discharged from the grease interceptor tank shall be directed to the inlet end of the septic tank. The capacity of grease interceptor tanks shall be a minimum of 1000 gallons and shall meet or surpass the 24-hour design flow. For restaurants and food service establishments with design flows of 2,000 gallons per day or greater, two grease interceptor tanks in series shall be provided. Such grease interceptor tanks shall have a combined liquid volume meeting or surpassing the 24-hour design flow. Grease interceptor tanks shall have inlet and outlet baffles that extend to a depth of six to twelve inches above the tank bottom (see Figure No. 6) and extend at least five inches above the liquid level. All manholes and cleanouts on grease interceptor tanks shall be extended to grade to facilitate cleaning. Grease interceptor tanks shall be provided with manhole covers that have been placarded with notification as to the danger of entering the tank due to noxious gases.

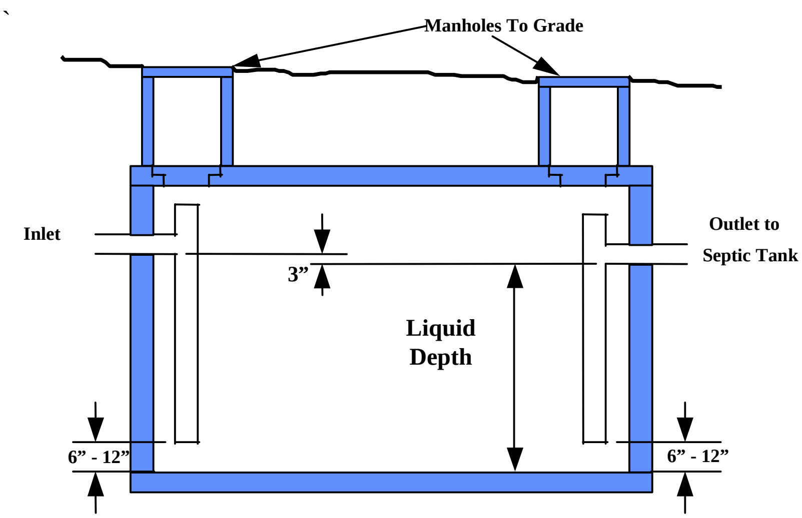


FIGURE NO. 6 - GREASE INTERCEPTOR TANK

Grease interceptor tanks can be single or two compartment tanks and shall be constructed out of concrete or other durable material approved by the Commissioner of Public Health. Concrete grease interceptor tanks shall meet all structural and access requirements for concrete septic tanks. This includes applicable configuration (pipe seals, inlet/outlet differential, etc) and access (riser sizes, stepped covers, etc) requirements consistent with the requirements for concrete septic tanks. Concrete grease interceptor tanks shall be marked with tank information (size, name of manufacturer, date manufactured, loading limits), and be subject to other applicable septic tank provisions (performance testing, cleaning, tank abandonment, etc). Non-concrete grease interceptor tanks shall also meet all of the requirements for concrete grease interceptor tanks excluding the structural and marking requirements. Non-concrete grease interceptor tanks must be approved by the Commissioner of Public Health. Non-concrete grease interceptor tanks shall be marked with the manufacturer’s name and tank designation number.