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# Manual

# Wastewater Treatment/Disposal for Small Communities

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## *Acknowledgments*

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# CHAPTER 1
# Introduction

## 1.1 Background

Over the past decade, changes in federal policies have forced states to play a larger role in financing and administering public works programs and compelled local governments to do more for themselves. A 1990 report by the U.S. Congress, Office of Technology Assessment identified several national wastewater treatment problems common to small communities:

- Absence of economies of scale and low per capita incomes
- Low level of technical expertise of many operating personnel
- Limited access to existing advanced technologies

Many small communities are without access to the engineering expertise that would enable them to resolve the technical problems related to assessing needs, evaluating technologies, siting facilities, and deciding on action plans to meet regulations. Furthermore, small, low-income communities have few alternatives to raising user fees substantially to cover operating and maintenance costs and to pay debt service.

In many cases, traditional wastewater treatment strategies have been shown to be inappropriate for the physical and economic characteristics of the small community. In the past, when public sewer systems were not available, the only practical alternative was to install individual onsite wastewater systems that used traditional septic tank–soil absorption treatment. While these individual systems still represent a viable wastewater management option for many small communities, not all situations or community planning strategies are suited for this type of disposal system. The current trends in wastewater treatment technology and the adoption of innovative management strategies have provided new alternatives and options for small communities. When carefully evaluated against actual community needs and available resources, these alternatives can result in a final selection and implementation of a wastewater management system that is responsive to the needs of each community by providing a balanced approach to cost allocation and operational responsibilities.

The 1977 Clean Water Act (CWA) and subsequent amendments provided the first federal recognition that costs are a major problem in the national program to address water pollution. This is especially true for established small communities where failing onsite systems and growing rural population densities necessitate the development of wastewater management programs to protect public health. Conventional options of providing gravity sewers and activated sludge treatment are often excessively expensive and require significant management costs that lead to unacceptably high burdens for small communities. As a result, the emerging focus for small community systems has shifted to small-scale systems that are designed to fit the specific needs of the community, rather than to provide a standard solution for all situations. Thus small communities have found that development of specialized wastewater systems calls for a well thought-out strategy in the early stages of problem definition and planning, the generation and evaluation of options pertaining to system selection and costs, and, finally, the selection of a management approach that meets the existing and future needs of all the identified small community user groups. The improved levels of treatment and long-term economic savings of well-designed small community systems provide engineers and planners with increasing confidence for reevaluating wastewater management strategies for existing and developing communities and give hope to local officials who formerly despaired when considering the cost of complying with environmental regulations.

Historically, although much of the initiative for financing small community systems had been assumed by the Federal Construction Grants Program, which provided up to 85 percent of the funding for construction of wastewater systems, most small communities were unable to obtain funding due to the reliance on priority lists related to population. Incentives for development of wastewater systems were provided primarily through the allocation of state "set-aside" funds for implementation of small community wastewater system construction grants. However, for many communities, the impact of the amendments to the CWA, the changes in the construction grant program instituted after 1984, and the implementation of the State Revolving Fund Program have restricted the availability of grant assistance and increased the local share of project costs. These changes have placed increased pressure on small communities to reduce project costs. This is usually accomplished through greater use of easy-to-manage, low-cost technologies and implementation of effective management strategies with appropriate planning and operational functions.

## 1.2 Types of Small Community Systems

Wastewater treatment alternatives for small communities can be broadly defined under three category groupings that represent the basic approaches to wastewater conveyance, treatment, and/or disposal.

- **Natural Systems**—that utilize soil as a treatment and disposal medium, including land application, constructed wetlands, and subsurface infiltration. Some sludge and septage handling systems, such as sand drying beds, land spreading, and lagoons, are included.
- **Alternative Collection Systems**—that use lightweight plastic pipe buried at shallow depths, with fewer pipe joints and less-complex access structures when compared to conventional gravity sewers. These include pressure, vacuum, and small-diameter gravity sewer systems.
- **Mechanical Systems**—that utilize a combination of biological and physical processes, employ tanks, pumps, blowers, rotating mechanisms, and/or other mechanical components as part of the overall system. These include suspended growth, fixed growth, and combinations of the two. This category also includes some sludge and septage management alternatives, such as digestion, dewatering, and composting systems and appropriate disposal information.

## 1.3 Use of Small Community Systems

The appropriate selection and use of the various small community wastewater management system alternatives will depend on both the physical characteristics of the site and the configuration of the service community. There are many technical alternatives from which small communities may choose in deciding how to collect and treat wastewater. Each of the various types of wastewater systems will be affected by the requirements of the service community and wastewater treatment system objectives.

Water conservation systems can significantly reduce the amount of wastewater generated and thus affect the feasibility and cost of different wastewater alternatives. Reducing water use can also lower the day-to-day operating costs for treatment chemicals and other utilities, such as the drinking water supply system. In some cases, effective water conservation programs can permit the use of smaller, less-expensive treatment facilities.

As a preliminary screening tool for system evaluation, it is often advantageous to consider under which circumstances systems are appropriate and eliminate other treatment strategies on the basis of inability to meet community or physical requirements for effective wastewater treatment. This can be accomplished simply by developing a generalized list of advantages and disadvantages for each alternative to assist in the preliminary decision-making process and to facilitate understanding of applications of treatment alternatives. To a degree, this manual assists in this process by listing only technologies that are useful in small community wastewater systems; it generally avoids discussion of technologies that are rarely appropriate for those applications.

The appropriateness of a given technical alternative will depend largely on the physical site constraints and the management capabilities of the community. Knowing when and where to apply a particular technology will also involve consideration of the regulatory requirements for each collection and treatment system and the capability of the community to support a management program tailored to the needs of the affected service groups.

## 1.4 Importance of Proper Planning/Management

Many issues and considerations affect the management approach for small community systems. These considerations and discussions of the planning and management alternatives for wastewater systems are presented in detail in Chapter 2. There are, however, several overriding concerns that will play a role in the evaluation of the management options and the selection of a technically sound treatment alternative. These include careful evaluation of all feasible technical and management alternatives against the concerns of the community, treatment objectives, economic capability, and management entities and regulatory environments that provide the enabling legislation to conduct the required functions and activities of the system management. The underlying theme that must be addressed here is to integrate the selection criteria with all the relevant components that affect an efficiently functioning wastewater management program. The management approach must not assume that innovative treatment alternatives are necessarily more environmentally sound or easier to operate, nor that a centralized system will necessarily prove easier to regulate and manage. The resolution of these issues is extremely site-specific and community-specific, and can be addressed only through comprehensive evaluation of all components of the small community wastewater management program.

## 1.5 Organization and Use of the Manual

This manual is designed to guide planners and designers through the required steps for developing well-conceived small community wastewater management systems and to highlight the specific characteristics of the system design and management functions that lead to the successful implementation of the selected system. The focus of the manual is to present information about small-scale wastewater treatment systems that are appropriate for the growing suburban and rural fringe-treatment areas that characterize the environmental needs of small communities. For the purpose of this manual, *small communities* generally refers to rural communities of fewer than 3,500 people; however, the population of small communities referred to can be as high as 10,000.

One of the major goals of this manual is to highlight the importance of planning and management considerations in choosing the appropriate system. This information is presented so that planners, consultants, local elected officials, or other organizational groups charged with providing wastewater management can better evaluate a broad range of appropriate options for small communities and understand the mechanisms for available management alternatives. This information is included to assist those charged with implementation of small community wastewater management systems in understanding the basic working principles and practical limitations of the various alternatives. It also is intended to facilitate the planning and the final selection process.

The manual is organized into five chapters. The successive chapters span the technical considerations, the concept and planning stages of alternative wastewater systems evaluation, and the implementation of comprehensive wastewater management programs. Chapter 2 discusses the planning and implementation of small community systems, placing special emphasis on roles and responsibilities and the regulatory approvals required to initiate wastewater programs. Site suitability and evaluation are discussed in Chapter 3, linking the characteristics of the physical and chemical conditions of site soils and hydrogeology to treatment processes. The specific characteristics of wastewater (e.g., flow, quality) and the capability of the wastewater treatment designs in addressing these characteristics are presented in Chapter 4. Chapter 5 comprises a technical presentation of various alternatives for wastewater treatment and residuals management systems and provides a discussion of the applications, factors affecting performance, features of system design and construction, and the operation and maintenance requirements.

The information presented herein is intended as technical guidance reflective of sound professional practice. Before any system is designed and constructed, local and state authorities should be contacted to determine the local design requirements for a particular system.

## 1.6 General References

Many technical manuals and guidelines, as well as specific guidance from state and federal programs, are available and should be consulted to assist in the evaluation and selection of a wastewater management system that is suited to the needs of a particular community. Several useful resources are listed below; other more specific guidance documents are provided in the reference sections within chapters.

When an NTIS number is cited in a reference, that reference is available from:

National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650


- 1990 Preliminary draft strategy for municipal wastewater treatment, Office of Water and Waste Management, Washington, DC, January 1981.
- A strategy for small community alternative wastewater systems, Office of Waste Program Operations, Washington, DC, December 1980.
- Planning wastewater management facilities for small community, EPA Office of Research and Development, EPA/600/8-80-030, NTIS No. PB91-111064, August 1980.
- Design manual; on-site wastewater treatment and disposal systems, EPA Office of Research and Development, EPA/625/1-80-012, NTIS No. PB83-219907, October 1980.
- Management of on-site and small community wastewater systems, EPA/600/8-82-009, NTIS No. PB82-2C0829, EPA Office of Research and Development, July 1982.
- Reference handbook on small-scale wastewater technology, US Department of Housing and Urban Development, Office of Policy Development and Research, Washington, DC, April 1985.
- Rebuilding the foundations: state and local public works financing and management, OTA-SET-447, Washington, DC, Government Printing Office, March 1990.
- Handbook of septage treatment and disposal, EPA Office of Research and Development, EPA/625/6-84-009, Cincinnati, OH, October 1984.

# CHAPTER 2
## Planning and Management of a Small Community Wastewater Project

### 2.1 Introduction

The planning and management activities of a wastewater treatment project are so closely intertwined as to make them at times inseparable and indistinguishable. The purpose of this chapter is to acquaint the reader with the planning and management work necessary to complete the six steps in the typical wastewater treatment project and to discuss approaches that may be particularly useful in small communities. The six distinct phases or steps in a wastewater project are:

- Problem recognition and mobilization
- Planning
- Design
- Construction
- Startup
- Operation

### 2.2 Problem Recognition and Mobilization

The driving force behind a wastewater treatment project is usually one or several of the following:

- A public or private sector individual or group recognizes the need for action and rallies a significant segment (not necessarily a majority) of the community to support the need for a project.
- The courts resolve an action to abate pollution and order remedial measures.
- State and/or federal agencies enforce pollution laws and/or order remedial measures.

The individual, group, judge, or agency responsible for creating the political or legal pressure that gets the project off the ground and who supports it through its evolution can be called the project's champion. The need for a champion is apparent when one considers the potential impediments to any wastewater project, which include the following:

- Not all community members contribute to or share the problem. Those that don't may oppose the project.
- Several community members may see the absence of wastewater collection or treatment facilities as a method of growth control.
- The cost of the project and the resultant annual fees may draw substantial opposition.
- The locations available for a treatment site may draw neighborhood opposition.
- The point of discharge and the perceived negative effects downstream may draw opposition from within and outside of the service area.

There is no formal management and planning scenario to follow in this phase of the project. Judicial and administrative rulings evolve out of other processes, and private sector efforts are directed toward raising community awareness of the problem and developing a public outcry for action. Nonetheless, the energy or the authority of the project champion is key for driving the project through this phase. The momentum generated and then sustained by the champion through the design phase in most cases makes the difference between a successful and unsuccessful project.

### 2.3 Planning

The formal planning and management of a project begins at the point where a committee is formed or a governing body is authorized to study the problem, explore options, and estimate costs. Management actually precedes planning in the sense that a formal plan to allocate resources—personnel, technical expertise, time, and funds—and a mechanism for measuring progress is required prior to proceeding.

The objective of the planning phase is to generate a recommendation to the community that covers the type of wastewater facilities required, with cost estimates for design, construction, and operation; a plan for managing the design, construction, startup, and operation of the facilities; and a plan for financing facilities development and operation. If a comprehensive facility recommendation cannot be generated, then alternative plans and cost estimates can be offered; but no more than three alternatives should be put forth. The choice of the number of alternatives is arbitrary but the reason for limiting the choices is that each option possesses a myriad of impacts for community members to consider. More than a very few choices will lead to confusion.

At the beginning of the planning phase, three key management actions must be carried out:

1. Appoint a person or, more typically, a group to carry out the planning effort.
2. Establish a deadline acceptable to the group and community for submitting recommendations.
3. Establish a preliminary planning budget.

Once formed, the group assumes management responsibility and establishes intermediate tasks, which may be in the following sequence:

1. Assemble or determine the whereabouts of all documentation pertaining to the reasons for the project and the general area to be served.
2. Familiarize all group members with the wastewater management problem and the reasons for the project.
3. Determine the need for consultant services.
4. Solicit and engage consultant services, if needed.
5. Arrange for public information/feedback.
6. Document the regulations and laws pertinent to the project.
7. Identify and evaluate technical alternatives.
8. Compare the costs of the most suitable technical alternatives.
9. Prepare a financing plan.
10. Prepare draft recommendation and a report for public and regulatory agency comment.
11. Prepare the final recommendation and report.

With the goals, schedule, staffing, and budget established, the process of bringing about the small community wastewater treatment system can commence. Accepting as a given the major goal of the planning process—the development of a recommended wastewater collection and treatment plan for the community—and leaving the appointment of the planning group's members to local judgment, the remaining discussion in this chapter focuses on the 11 intermediate tasks.

### 2.3.1 Assembly of Documentation

The gathering of pertinent documentation concerning the wastewater treatment project, which can be performed by the planning group or by a consultant, is the foundation of any successful planning effort. The information collected must formally document the community's perception of the problem, how the project was conceived, and who was involved. Supplemental information should include:

- U.S. Geological Survey quadrangles, tax assessment records, flood plain data, and any other maps of the planning area
- Geological studies of the planning area (especially those concerning local soils and those that include aerial photography)
- Government (state and local) studies of sanitary conditions in the planning area
- Local climatological reports and data
- Studies of local surface-water and ground-water quality and quantity
- Reports on the history and archeology of the area
- State, county, and local planning and development studies of the area
- Any other data or information that could have an impact on the project

All such information is necessary to assist in documenting the existence and extent of wastewater management problems, to evaluate alternative solutions, and to identify essential information that is lacking. The data will be used to establish the layout of sewers and the location of treatment facilities; to identify areas of historical importance and flood plains that should be avoided; to characterize the soil and hydrogeological features of the area to determine potential sites for land-based systems (i.e., cluster systems); to establish the proximity of future roads and subdivisions; to plan for treatment and discharge systems; to identify streams with the greatest flow and the most appropriate stream class designation; to make preliminary evaluations of the cost and feasibility of acquiring sewer easements; and most important, to establish the project's justification. Legal counsel should be involved in gathering documentation on the enviromental impacts of each alternative treatment method considered so that complete findings can be reported to state and federal regulatory agencies.

### 2.3.2 Familiarization of Planning Group Members with the Nature of the Wastewater Management Problem and the Reasons for the Project

Although the subject of wastewater problems and potential solutions can be very technical, the typical small community wastewater planner is not expected to have had special training. A general understanding of the problems and principles involved is all that is necessary. This is easily acquired if the group receives assistance from a regulatory expert or a consultant. Unassisted groups will require training or will have to be composed of technically competent individuals.

Familiarization begins during the data assembly process wherein all members are expected to read the various documents and reports. Technical reports may require interpretation by knowledgeable governmental officials or consultants. Planning groups with consultant assistance will usually receive a summation of the existing data in nontechnical terms. Ultimately it is important that all group members have a firm understanding of existing conditions to facilitate their review of the implications and impacts of remedial alternatives.

### 2.3.3 Determination of the Need for a Consultant and the Extent of Services

Not all wastewater projects require consultants during the planning stages. Common determinants of the need for a consultant are as follows:

- Project size (i.e., the larger the project the more likely that consultant services will be necessary)
- Project complexity (e.g., does the problem definition or solution require a hydrogeologist and/or do the only technically feasible treatment methods require some type of treatment plant, etc.)
- Project schedule (e.g., consultants working full time on a project are more efficient than part-time volunteers)
- Availability of volunteer experts as committee members
- Capability of existing management agency, if available

It is difficult to establish rules of thumb regarding the necessity for professional consulting services during the planning phase. Advice on the matter can be secured from state, local, and county regulatory agencies that have an interest in the project. Communities that can afford to hire consultants will usually benefit from the assistance. However, planning committees must not abdicate their responsibilities to a consultant. The group is ultimately responsible for making critical decisions concerning the project, and a consultant should serve only to facilitate the work of the group.

Use of a consultant during the planning stages eliminates potential liability for the performance of the recommended plan. More specifically, if an unassisted nonprofessional group develops a recommended plan that is subsequently used as the basis for a system design and the system doesn't work, or it costs substantially more than anticipated, who is responsible? The use of consultants for final planning and design may provide protection for the community against the possibility of charges of substantial system nonperformance, failure, or excessive cost of the preliminary plan.

### 2.3.4 Soliciting and Engaging Consultant Services

Of all the intermediate tasks in the planning process, retaining a consultant is the most difficult for small communities to master, primarily because project group members will often lack experience in contracting for professional services. The process is fraught with pitfalls. For instance, it can be difficult to define the work to be done so that all proposals can be evaluated using a common measure. And it may not be easy to identify the firms with the most experience in technologies pertinent to a community's wastewater problem.

There is no quick resolution to such issues. The task requires diligent effort by all members of the planning group or consultant selection subcommittee. Where federal grants or loans are involved, federal procurement regulations should be followed closely. Similarly, special requirements may apply where state funds are used. In general, five steps should be followed:

1. Prepare a request for proposals (RFP)
2. Advertise or distribute the RFP to selected firms (e.g., firms can be selected by issuing a request for qualifications (RFQ))
3. Collect and review proposals to create a short list of firms to be considered
4. Interview selected consultants from the "short list"
5. Choose a consultant

Establishing a price for consultant services can be approached in several ways. The fee may be a required component of the proposal, a separate simultaneous submittal, or an issue negotiated following consultant selection, or it may be established by a combination of these methods. Since the legal wording of a contract for services will vary in form from state to state and locality to locality, the discussion below concerns the technical selection process, with limited reference to the fees and contract language.

#### *2.3.4.1 The Request for Proposals*

A request for proposals (RFP) generally begins with a brief statement of the problem affecting the community and the events that led the community to initiate the wastewater planning process. The statement of the problem and background is followed by a description of the work that would be performed by the consultant.

Drawing up an RFP requires the planning group to have become as knowledgeable as possible about community wastewater problems. The description of work also requires some knowledge of the possible courses of action. Information can be acquired by consulting regulatory agencies, studying the responses to a request for qualifications from interested engineers, interviewing officials from nearby communities, and conducting library research. In general the RFP should request that the consultant:

1. Search out and review existing data on the wastewater project area and the problem
2. Define the problem using available data and perform additional analyses as necessary
3. Identify alternative technical solutions and prepare cost estimates

4. Evaluate and compare the various technical and management alternatives and costs, and prepare draft recommendations
5. Conduct a public hearing to solicit community, regulatory agency, and planning group comments on the draft proposals
6. Prepare a final report and recommendation incorporating the comments received

The statement concerning work to be performed should also include instructions regarding interactions between the consultant and planning group. For instance, regular meetings should be scheduled for the consultant to report on progress and findings. A formal report should be made to the committee upon completion of each of the tasks listed above. These early progress reports, which can be incorporated in whole or in part into the final text, will give the planning committee opportunities to provide input, and allow it to perform a quality assurance review of the work prior to the final document preparation. The RFP should also request or establish a schedule for the work. The remainder of the RFP should contain a description of other required submittal contents, such as information on insurance, liability, and contractual requirements. The form of the RFP, as well as insurance, liability, and other legal issues, should be reviewed by the community lawyer before the solicitation is issued. Such a review should be relatively brief and inexpensive if the section of the solicitation on the scope of the work to be performed is well thought out.

#### *2.3.4.2 Proposal Evaluations*

In general, responses to requests for professional services use the following as a table of contents:

- Description of the problem (from the RFP and research)
- The proposer's approach to solving the problem
- Detailed scope of services to be provided
- Schedule of work
- Consulting firm's documented experience with similar projects
- Proposed staffing for the project, with résumés
- Staff organization for the project
- Categories of staff to be assigned to the project and the estimated time each will expend on the project (e.g., accounting, engineering, hydrogeology)
- Proposed fee or fee schedule (optional)

In describing an approach to the problem, the consultant should be allowed considerable latitude. In reviewing and comparing approaches to the project, the reviewers should look specifically for clues to the consultants' understanding of the project needs. Often consultants have their marketing staff produce promotional statements about the company and staff's abilities. The reviewers, however, should look for specific language indicating that the consultant has made an effort to understand the community's wastewater problems, and determine whether the suggested approach seems reasonable and demonstrates the consultant's competence. The thoroughness of understanding and completeness of the suggested approach should be reflected in the proposed scope of services, which is a detailed presentation of the consultant's plan of action.

When comparing the scope of services section of proposals, reviewers should consider the level of effort proposed for various stages of the work. Any emphases proposed should be noted to see if plans correspond to the project group's expectations.

The proposed scope of services section, as well as the approach section, should also be compared with the section describing the firm's experience with similar projects. Does the proposing firm have experience to support its statements in the approach and scope-of-services sections? Do the résumés of proposed team members reflect the firm's experience with similar projects? It is necessary with proposals from multilocation firms to determine the work location of proposed team members. Are they available to perform the work, and how will their location impact the ability to participate as required? Modern communication technologies reduce, but do not eliminate, the need for all team members to be located at sites convenient to the project. Finally, the proposed schedule should be evaluated against the planning group's deadlines and the need to study the various portions of the completed work prior to taking action.

#### *2.3.4.3 Consultant Interviews*

The selection committee generally chooses several of the best proposals to follow up on with further consideration and consultant interviews. Interviews are carefully scheduled affairs to help differentiate the best proposal from the others. All interviewees are given the same amount of time. The interview itself usually begins with a presentation by the consultant during which he or she is asked to embellish and expand on the written proposal contents. The time allowed for presentations should be relayed to consultants with the short list announcement. Following the consultant's presentation, the selection committee asks questions of the consultant to help it distinguish the one best suited to the committee's needs.

Following the interview process a consultant is selected. If a fee quote was requested with the proposal and the fee is acceptable the process can proceed directly to contract execution. If a price quotation was not required by the solicitation, price negotiation and contract execution follow the technical selection process.

### 2.3.5 Arrange for Public Involvement and Feedback

In the past the EPA grants program called this aspect of the planning process *public participation*. Because the primary responsibility of the individual or group designated to manage the project's development involves planning, the public involvement process consists mainly of providing information and getting feedback. That is, the public advises the planners of its knowledge about the wastewater problem and provides commentary or feedback about various proposals as they are developed and finalized.

It is important to involve the public during the planning process in order to improve the chance a majority of people in the community will accept the planning group's final recommendations. A plan to encourage public involvement must be prepared in the early stages of the planning process and managed throughout. The wastewater planners must establish lines of communication with the public. Relying on the local newspapers and radio stations and flyers is not enough. The committee must provide forums that will draw out the public and encourage comment.

Successful public involvement programs include the following:

1. Scheduled public meetings that mark important project milestones and provide an opportunity for making informational presentations by the planning committee or its consultants and for soliciting public comment. A typical schedule includes advertised meetings at the following points:
   - At project kickoff
   - At completion of draft recommendations
   - At completion of final plan recommendations
   - At design-phase kickoff
   - At completion of design
   - At construction contract award
   - At any and all major changes in the projects direction of the project
2. Public outreach to draw diverse individuals or groups into the planning process
3. Regular news releases or flyers describing the work and progress of the committees

Public involvement must be managed. Since the planning committee is charged with developing a recommended plan for wastewater management, various community groups both for and against the project will try to ensure that their concerns will be addressed fully and that their viewpoints will be given prominence. Meetings must be conducted in such a manner that all factions are given a hearing. Arrangements for day-to-day contacts with the local citizenry must be made such that the public is heard and comments acknowledged. Special interest groups require additional attention to ensure that members are given every opportunity to express their concerns and that their concerns are thoroughly addressed. Special interest groups with a stake in the project that do not become actively engaged in the planning process dialogue should be approached directly and encouraged to make concerns known. The degree of difficulty and level of effort associated with this task will depend on the level of controversy generated by the project. A consultant is often assigned the responsibility of planning public involvement activities and managing the program.

### 2.3.6 Document the Regulations and Laws Pertinent to the Project

The documentation of pertinent laws and regulations is usually the responsibility of the consultant, when one has been retained. The information that is required early on in the planning effort is important for determining the technical feasibility, a schedule, and the estimated cost of the project. This task includes the collection of all permit applications required and regulatory agency guidance documents. A listing of subjects and issues likely to fall under regulatory control are listed in Table 2-1.

The list in Table 2-1 is not meant to be all-inclusive and should only be used as a guide to get the regulation-gathering process started. The regulation of wastewater treatment varies from state to state and locale to locale. Planners should inquire about the existence of regulations at all levels of government.

Using information on applicable regulations, a flow diagram can be constructed illustrating the steps that need to be taken to carry out the project. A schedule for the completion of the various tasks can be prepared based upon the requirements for the various approval applications and the need to perform studies and engineering assessments as supporting documentation. Accurate completion of this subtask is important for establishing design requirements for the next stage of the project.

### 2.3.7 Identify and Evaluate Technical Alternatives

With the first six tasks completed or near completion, the actual planning of the project can begin. The initial array of options available to the small community is more inclusive than it is for the larger cities. However, a realistic list of technologies that can be operated by small communities is somewhat shorter and includes onsite systems, cluster approaches, and community-wide options. Even with this more select variety of available options, small communities can expend valuable time and money pursuing and evaluating inappropriate alternatives. Some communities have dealt with this problem by only considering only few options with which they are familiar. While this approach may produce a solution, it may not be one that is best suited for the long-term interests of the

**Table 2-1. Aspects of Wastewater Treatment Management Typically Regulated by the Government**

| Issue | Usual Level of Government |
|---|---|
| National Pollutant Discharge Elimination System permit | federal, state |
| Individual septic system regulations | state, local |
| Ground-water discharge standards | state, local |
| Receiving stream classification and discharge restrictions | state |
| Wetland regulation (sewers and treatment plant siting may impact wetlands) | federal, state |
| Historical preservation (many areas require historical and archeological evaluations of proposed public works sites) | state, local |
| Highway construction requirements (the regulation or the prohibition of road excavation can have enormous impacts on the feasibility and cost of sewers. Driveway curb cuts for treatment facility locations is a lesser problem) | state, local |
| Flood control (construction and siting requirements in or adjacent to flood plains can affect feasibility and cost of projects) | federal, state, local |
| Erosion and sedimentation control regulations | state, local |
| Navigable waterway construction (the Army Corps of Engineers regulates all construction within the waters of the United States) | federal |
| Planning and zoning impacts | local |
| Procurement standards | federal, state, local |

Note: Local level may include county, township, or parish.

community. Suggested criteria are presented below to assist planning groups to narrow down the list of options.

### *2.3.7.1 Small Community Systems Must Be Simple to Operate*

Ease of operation is the single most important criterion for small community wastewater treatment systems. Numerous projects have failed to perform and have cost the communities they serve large sums expended to correct problems because planners have overlooked this requirement. This criteria is particularly important because persons with the training required to operate the more complex or delicately balanced systems are not readily available in the general population. Moreover, once trained, small community wastewater system operators are difficult to retain at rural and suburban pay scales. Small communities that are situated close to larger cities and towns may be able to employ on a part-time basis the operators of larger, more complex facilities. In addition, service companies with suitable expertise are more likely to be available to communities located in a metropolitan area.

The simplest of all systems is the individual on site septic system, and its simplicity is a reason for its attractiveness as a wastewater management option. Persons with the skills necessary for onsite system maintenance are generally available even for the most innovative of the modern systems.

Large subsurface, sand filter, and wetland-based systems are examples of larger systems that are simple to operate. Members from the local farming, electrical contracting, plumbing contracting, and excavating contracting communities collectively have the skills necessary to construct and operate these systems.

Biological and physical/chemical systems require more than a knowledge of the systems' electronics and mechanics to achieve and maintain optimum performance of all treatment elements. If systems based on these principles are to be considered, then equal consideration must be given to the availability and affordability of the required operations staff.

### *2.3.7.2 Small Community Systems Must Be Reliable*

All discharging systems and large subsurface systems must operate within performance standards that are established by regulation and are specified in the operating permit. Violations of these standards may expose the community to fines or legal action. Metropolitan operations with seven-day-week, round-the-clock staffing are monitored constantly to detect operating aberrations that could lead to a permit violation. In contrast, small community systems tend to be monitored on an intermittent basis and therefore must be designed to be resistant to upsets and to provide days or weeks of advanced warning when a problem is developing.

The type of warning of impending performance deficiencies provided by a system is somewhat related to the system's complexity. Onsite systems that are properly monitored can provide advanced notice of elevated contaminant levels in ground water or of absorption system failure months or years in advance. For instance, sand-filter ponding, which indicates potential problems, occurs weeks or months before critical conditions occur. Wetland systems provide days of advanced notice of performance failure. Biological and physical/chemical systems provide advanced notice of only several hours, at the most, of potential problems.

Reliability is also an important consideration in the selection of alternative wastewater collection systems. Many of the early designs for alternative sewer systems failed to provide a level of reliability equal to the reliability of more conventional systems. To a degree, the early problems were due to a general unfamiliarity with the weaker links in these systems. Problems also resulted from efforts to shave costs. Also, a consensus has yet to be reached among the states on corrosion-resistance standards for electrical components in septic tank effluent pump systems. Although the atmosphere in these systems is extremely corrosive, vulnerable metal components are still in use. The use of such materials in pressure and small diameter gravity sewers can result in premature failure of system components and necessitate difficult repairs. Reliability in terms of corrosion protection, structural durability, ease and access of maintenance are therefore crucial concerns.

#### *2.3.7.3 Small Community Systems Should Be Economical to Construct and Operate*

For years onsite systems have been proposed as a low-cost, reliable, and simple method of treating wastewater problems. Experience has shown that the lower cost is dependent on the perspective of the evaluator. In a community of 50 homes, for instance, the total capital cost to replace 10 or even 20 onsite systems is likely to be lower than to provide collection and treatment facilities for all 50. However, on an individual basis there may be no difference in capital cost. That is, the homeowner contracting for an onsite repair or a replacement may incur as much capital cost as he or she would have incurred were a central system constructed. On the other hand, onsite systems have shown themselves to have lower O&M costs than community-wide systems with central collection and treatment facilities.

Sand filter and wetland systems have also been promoted as low-capital-cost treatment options. Unfortunately the cost estimates for these systems are a vestige of the light-duty designs used during their technological development. The design standards for working versions of these systems are those used for public works that require rugged and durable facilities intended for years of trouble-free service. They must be equipped with liners to separate them from the surrounding environment and with dozens of performance, operation, and flow monitors. These added requirements cause these systems to cost as much if not more to construct than complex biological or physical/chemical systems; however, O&M costs are generally low because of the simplicity of operation and reliability of these systems.

Many of the more complex biological- or physical/chemical-based systems are available to small communities as complete prepackaged units. Prefabrication renders these systems very cost-competitive with those of lesser complexity. The O&M costs for these systems are generally higher than the lower technology systems because they require daily monitoring to check the balance and performance of the various subsystems. The personnel performing these tasks are required to be more highly trained than operators of simpler systems, thus salaries for personnel with the required skills are generally higher.

Collection systems should not be left out of any discussion of system capital costs. In unsewered areas, they can account for up to 75 percent of the total system's capital cost, especially if development density is low and the length of sewer per user is high.

Their attractiveness has been largely attributed to their eligibility for supplemental funding as alternative collection systems under the EPA grant program. They remain attractive in the post-grant era because they offer potential cost savings in that they require shallower excavations for pipes, smaller pumps, and simpler pump stations than more conventional systems. The lower cost of construction because of shallow excavations can provide savings of up to 50 percent depending on the soil, rock, and ground-water conditions in the sewered area. Savings are the result of reduced requirements for shoring, dewatering, and blasting. The small diameter of these sewers may permit their installation using emerging methods such as plowing, trenching, and directional drilling that provide even greater potential construction cost savings. There are few direct comparisons regarding the maintenance cost differential between conventional and small diameter gravity sewers (SGD). However, available data indicate that O&M costs can vary from less to more than conventional sewer technology depending on local conditions (e.g., O&M costs for a single conventional pump station can equal those of an SGD system.)

Thus, simplicity of operation, reliability, construction and operating costs are the considerations to weigh when selecting potential wastewater management alternatives. A preliminary screening of alternatives will produce a short list of options deserving of in-depth consideration. The planning group consultant will generally perform this task and report to the committee. Unassisted planners can seek the assistance of state and federal regulators.

Planning groups should be prepared for potential problems when turning to consultants or regulators for advice. Very few advisors have had experience with all technological approaches. As a result, they are prone to recommend familiar processes while questioning the cost and performance of processes with which they are less familiar. No method guarantees an open-minded assessment by all evaluators. The planners must be critical in their consideration of recommendations and representations to ensure a fair evaluation. The goal of the alternative review is to produce a list, in order of preference, of the technically appropriate wastewater management systems.

### 2.3.8 Compare the Costs and Environmental Impact Assessments of the Most Suitable Technical Alternatives

Making system cost comparisons requires that conceptual designs be prepared for all viable alternatives. The level of detail must be sufficient to permit preparation of both a cost estimate and an environmental impact assessment (EIA) for the construction as well as the operation of each system. Cost estimates and EIAs at this stage are best prepared in concert with public works staff, consulting engineers, or other experienced professionals. The EIAs will also require input from state and federal environmental management staffs familiar with stream designations and existing environmental regulations that may restrict the discharge location or even the use of certain alternatives.

The conventional way of reporting and comparing costs is in terms of the project's present worth. This approach involves calculating in today's dollars the value of the project's engineering cost, construction cost, financing cost, operating cost, and salvage value for a given planning period (e.g., 20 years). Planners are also advised to consider the actual construction cost, engineering costs, and annual operating cost separately. The ranking of the various options by present-worth cost is likely to generate a list different from the one generated by a technical evaluation.

The objective of the cost comparison is to generate a preliminary financing plan with projected user costs to connect to the system, to amortize long-term debt, and to pay for yearly O&M. The financing plan will be of great interest to the potential users of the system and will have a significant effect on the public acceptability of the planners' recommendations.

### 2.3.9 Prepare a Financing Plan

#### *2.3.9.1 Overview of Financial Management*

Sound financial management is crucial to the efficient operation of wastewater systems. It involves estimating expenses and needed revenues, keeping comprehensive records, and planning for the future. It is not enough for small community wastewater systems to break even. Successful systems charge enough so that funds can be set aside for the system's future needs, such as equipment replacement, line repairs, and emergencies.

Poor financial management of wastewater systems can lead to:

- Deferred maintenance or repair
- Inability to replace key system components
- Inability to operate the system so that it performs as designed
- Eventual operational or financial crises
- Violations of wastewater regulations

The components of financial management include:

- General utility planning (anticipating capital and operating needs)
- Financial planning (meeting capital and operating needs)
- Budgeting
- Cost recovery
- Accounting and information services

Several of these components are discussed in more detail below.

**Financial Planning.** Financial planning covers two areas of concern for the small community wastewater system: capital improvements and operational expenses. The costs of capital improvements include not only the costs of materials and construction, but also legal, engineering, and certain administrative costs, as well as other costs, such as land acquisition.

The second area of concern of financial planning for small communities is financing operational expenses, which are the annual expenses associated with the operation of the facility. These include:

- Operations, maintenance, and most administrative costs
- Annual debt service expenses
- Financial reserves

The goal of financing operational expenses is to make the wastewater system self-supporting. Reliable sources of income should be found that will match existing and projected expenses for operation, maintenance, equipment replacement, and loan repayment costs.

**Budgeting.** Most utility budgets are prepared on an annual basis. The budget should reflect the major activities of the utility, including detailed information regarding the revenues and expenses of the facility.

*Typical Revenue Accounts*

- User service charges
- Hookup/impact fees
- Taxes/assessments
- Interest earnings

*Typical Expense Accounts*

- Administration
- Wages
- Benefits
- Electricity
- Chemicals
- Fuel and utilities
- Parts
- Equipment replacement fund
- Principal and interest payments

The budgetary needs of each utility are unique. The information contained in the budget should be representative of the information contained in the general utility plan and the financial plan. Financial statements should be prepared on a monthly basis and compared to the budget. The budget should be revised if the projected costs and revenues are significantly different from the actual costs and revenues.

**Cost Recovery.** In order to maintain the facility as a self-supporting entity, the operating revenues must be sufficient to meet all system costs. The following steps are necessary to develop a cost-recovery system:

- Identify all system costs
- Compile system data as a basis for allocating costs among users
- Classify system users, individually or in groups, to allocate costs in proportion to their use
- Estimate annual revenue needs
- Establish rates to meet annual revenue needs
- Develop billing procedures according to use, and for collection of user charges

A healthy system will have a positive cash flow as shown on the annual income statement. To stay financially healthy, a wastewater system must:

- Identify and recover all expenses
- Institute and maintain an effective financial management system that includes:
  - A utility planning process
  - A financial planning process
  - Effective annual budgeting
  - Effective cost recovery
- Complete annual audits and act upon the recommendations of the auditor

### *2.3.9.2 Financing Alternatives*

Various financing alternatives exist for small communities, including:

- Community system reserves
- Farmers Home Administration loan/grant programs
- Community development block grants (CDBGs)
- Municipal facilities revolving loan funds (also referred to as State Revolving Fund loans)
- Bond issues
- Local assessments and local option sales taxes
- Rate increases

When evaluating these financing resources, the impacts of the financing option upon the system user and future budgets should be explored. A selection of these financing mechanisms are discussed below.

**User Service Charges.** The user charge system is the central and most important component of a revenue plan, since it usually accounts for 80 to 90 percent of total revenues. The user charge system has two components: setting the user rates and collecting them. The checklist that follows lists the type of user information that should be established.

Key revenue criteria for EPA-funded wastewater treatment facilities include:

- Charge each user in proportion to the quantity and quality of the discharge
- Notify the user of rates annually
- Impose surcharges for industrial or commercial wastewaters that require additional treatment
- Establish a financial management system to account for revenues and expenses

| User Service Charges Checklist | Is This Done at Your Utility? | | |
|---|---|---|---|
| | Yes | No | Unsure |
| All costs are identified | | | |
| Costs are allocated proportionately based on use | | | |
| Flow characteristics are known for each customer class | | | |
| Each customer's use is known or fairly estimated | | | |
| Customers are billed in proportion to use | | | |
| Billing cycle provides timely revenues | | | |
| Established procedures ensure collection of delinquent bills | | | |

**Federal Grants and Loans.** The Farmers Home Administration (FmHA) provides loans and grants to rural communities of up to 10,000 people for wastewater treatment facilities. These loans and grants can be used to build, repair, improve, or change a facility according to community needs. The loans have a maximum term of 40 years (or a shorter term if specified by state law). Interest rates may be as low as 5 percent for borrowers that meet specific criteria (EPA, 1983; EPA, 1984).

FmHA loans are intended for communities that are financially sound but cannot obtain funds from other sources at reasonable rates (EPA, 1983). Grants covering up to 75 percent of a project's cost may be made to help the most financially needy communities reduce the repayment burden on the system's users (EPA, 1984). Eligible applicants include municipal and county governments, public service districts and authorities, other nonfederal public bodies, Native American tribal organizations, and broad-based community nonprofit corporations. Priority is given to local public bodies providing service to communities of 5,500 or less or to projects meeting certain other criteria establishing urgent need.

FmHA may participate with other federal or public service agencies in jointly funding projects. Community facility loans are administered by FmHA district offices, but application information is available at local FmHA offices. Other federal agencies such as the Economic Development Administration and the Department of Housing and Urban Development provide grants and/or loans to communities for various purposes, including wastewater systems. County extension agents or regional planning agencies are good sources of information on such programs.

**State Revolving Fund (SRF) Loan Program.** SRFs are available to towns for purposes such as constructing wastewater treatment facilities. Loan repayments go directly back into the fund to be loaned to other communities (EPA, 1988).

Under the Federal Water Quality Act of 1987, EPA provides each state with startup money to establish a revolving loan fund or with money to add to an existing loan fund for wastewater facilities. As of January 1992 all states had established SRF programs.

Each state's revolving fund program is slightly different. Some programs limit assistance to communities with poor or no credit ratings. Others base their assistance on such factors as the affordability of the project, public health benefits, and the potential for economic development. Programs also vary according to maximum loan amount, percentage of total project cost eligible for a loan, interest rate, and duration of the loan. Some states simply fund projects on a first-come, first-served basis, relying exclusively on the community's ability to repay. Thus the state must be contacted to see how its revolving loan fund works (EPA, 1988). Each state must establish legal mechanisms to ensure that the SRF and all repayments are used in a manner consistent with the Federal Water Quality Act (EPA, 1989).

There are several advantages and disadvantages associated with SRFs. Perhaps the biggest advantage is that, if implemented properly, the program can become self-sufficient using loan repayments as a source for future loans. This reduces future dependence on state funds and bonds as a funding source. In addition, the use of loan repayments as a funding source can make a state program independent of less-reliable revenue streams, such as federal grants and annual state appropriations for which potential problems exist. The state interest-rate ceiling, however, may limit the interest charged to borrowers, potentially limiting levels of future loans. Also, loan repayments may not flow back in sufficient amounts to meet program needs in a timely fashion (EPA, 1984).

**Bond Issues.** Bonds are the most common means of securing funds for the construction of wastewater treatment facilities. Like a home mortgage, bonds extend payments for new facilities over a period of 10 to 30 years. Bond proceeds are primarily used as a source of funds for bond banks or direct loan programs. They have been used for leveraging monies in revolving loan funds or providing grants and interest rate subsidies. Leveraging is a technique by which bonds are issued to be repaid from other financing sources in order to provide project funding sooner than would otherwise be possible (EPA, 1984).

The source of bond payments to investors depends on the type of bond that has been issued (another term for bond payments to investors is *debt service*, which simply means the interest payments on the bonds and bond principal as bonds are retired). The three common bond types are:

- *General obligation bonds*, which are secured by the state's full faith and credit and taxing ability; payments are made directly from the state's general fund.
- *Revenue bonds*, which are secured by the revenues generated by project operation. Thus debt service is paid from project revenues. In the case of traditional revenue bonds issued by communities to build single projects, project revenues are made up of user charges. In the case of a state program that funds many different projects, project revenues may include loan repayments from communities and special revenues such as dedicated taxes and EPA capitalization grants.
- *Double-barreled bonds*, where debt service payments may also come from a combination of sources: taxes and project revenues. In this case, the bonds are secured first by project revenues, then by the full faith and credit of the state, should project revenues not be sufficient to repay the bonds. This type of bond is generally not available to small communities.

Regardless of the bond type used, bonds always work in combination with another funding source that will be dedicated to repayment.

General obligation and revenue bonds are traditionally used by state governments to obtain long-term funds for the construction of capital facilities such as wastewater projects because they provide statewide benefits for present and future users. Bonds may not be the appropriate source for program startup or operating funds, however, because the need for repayment diverts funds from direct financial assistance and because these costs cannot be recovered to make future debt service payments (EPA, 1984).

**State Bond Banks.** Government bonds are backed either by the general taxing power of the issuing government (i.e., general obligation bonds) or by continuing sources of revenue such as sewer user charges (revenue bonds) (EPA, 1983).

Most state bond programs act as go-betweens for municipalities and the national bond market. These programs help many small towns that cannot issue bonds, or if they can, they must pay high interest rates to attract investors. A state's typically high credit rating allows it to issue bonds at relatively low interest rates. Bond banks act in two ways: they may either guarantee local bond issues or they may actually buy local bond issues.

Not all states have bond banks and those that do have a variety of alternative arrangements. State offices must be contacted for additional information about eligibility, how to apply, and terms.

### 2.3.10 Prepare Draft Recommendation and a Report for Public and Regulatory Agency Comment

Completion of the preceding steps permits the selection of a wastewater management approach to recommend to the community. All the previously collected data and evaluations should be assembled in a document known as a *facility plan* that is released for public and regulatory agency review. Comments from all parties are solicited. The usual forum for public comment is a public hearing.

Of the several junctures in the wastewater system planning process where formal public interaction takes place, the public hearing on the recommended plan of action is the most critical. The effort put into preparing for this meeting will be reflected in the ease in which the project proceeds. Insufficient preparation can create mistrust and doubt among the public and lead to a lengthy and potentially costly effort to restore support and confidence. Aspects of the planning process that should be brought out in the facility plan include:

- Problem definition
- Thoroughness in the number of alternatives evaluated
- Accuracy in the development of conceptual plans
- Accuracy of cost estimates
- Justification of all charges
- Environmental impacts of the proposed system (e.g., benefits and potential problems for all media)

If the plan's foundation is thoroughly established, the recommendations usually can be modified or changed readily in accordance with feedback from the public and from agency representatives.

### 2.3.11 Prepare Final Recommendation and a Report

Following a well-prepared public hearing on the facility plan, public and regulatory agency comments are addressed and a final plan prepared. This is then released for public review. A meeting of the representatives of the affected community or the sponsoring community is subsequently held for the express purpose of voting on acceptance or rejection of the proposed plan. If the public hearing on the draft plan was well attended and the plan well prepared, it is likely that public comment at this meeting will be limited.

### 2.3.12 Summary

The planning scenario presented is subject to modification depending on the state and locality in which the wastewater project is intended to be built. For instance, the facility plan may be put to a referendum vote at a polling station in some communities. Nevertheless, some expression of community acceptance is necessary to complete all planning phases. If public participation/liaison has been effective, the likelihood of community support is greatly increased.

Many projects never get beyond a majority vote in opposition to the plan. If the planning process has imparted enough momentum to the project, it is likely to get into the next phase. At this point the energy of the project champion is often essential.

The major reason for a project stalling after the vote is inadequate funding to carry on. This impediment looms larger now that the federal grant program has been curtailed.

## 2.4 Design

The first step in the implementation of planning recommendations is the preparation of a formal engineering design. From this point in the process on, paid consultants and contractors are generally used because of the expertise required to prepare the technical plans, specifications, and legal papers required to secure regulatory approval and for use as construction documents.

The primary work of the community during this phase is the management of consultant selection, work progress, and budgets. Although the formal planning phase will have been completed, during this and subsequent phases the project group must continue to plan for the eventual startup and operation of the facility.

The group given responsibility for these activities may be made up of the same or most of the same people who participated in the planning effort or may be an entirely different group depending on the management recommendations developed in the planning phase. The design-phase manager is expected to complete the following tasks in achieving the major goal, the production of plans and specifications:

- Consultant selection
- Budget management
- Schedule management
- Work quality management

These tasks are discussed below. Planning for operations is a component of the work quality management task and will be dealt with in that discussion.

### 2.4.1 Consultant Selection

The consultant solicitation process for this phase is similar to the process followed in the planning phase. However, the request for proposals and the response should be much more focused. Specific areas on which consultant's proposals will be evaluated are:

- Experience — What is the firm's experience with the type of project and technology being proposed, and what is the experience of employees proposed for the work?
- Approach — This narrative should provide insight into the depth of a consultant's knowledge of the subject and whether there is a sharing of the community's philosophy.
- Scope of Work — This is the consultant's specific work proposal. Does it answer the RFP and cover all the known issues thoroughly? Does it suggest activities that may have been omitted in the RFP?
- Schedule — Does it take into account RFP-imposed deadlines? Does it allow for community review at critical points? Does it anticipate reasonable times for community and regulatory review?
- Budget and Billing — Is the price considered fair for the scope of work proposed? How is progress to be measured and billed?

Other nontechnical requirements such as indemnities, liability insurance, bonds, and guarantees should be established with advice of the community's counsel.

Proposal review and final selection are the same as in the planning phase.

### 2.4.2 Budget Management

Management of the project's budget is a relatively straightforward task, although special emphasis should be put on scrupulous recordkeeping. Recordkeeping to the specifications of funding agencies is particularly important.

Assessing the value of the consultant's work and reviewing bills is an important fiduciary responsibility of the management team. The manager should be thorough in evaluating bills and statements of work. He should bear in mind that the relationship with the consultant is defined by the contract.

Requests for change orders from the consultant should be compared with the contract to confirm that the work was not originally included in the contract.

### 2.4.3 Schedule Management

Most project milestones will have been established by the section in the RFP on the scope of work. However, practical steps can be taken to ensure that deadlines are met. The most effective action is the scheduling of regular project meetings for monitoring progress and maintaining momentum. Such meetings should be held at least monthly but may need to be held more frequently on smaller or fast-paced projects. The minutes of these meetings will serve as a formal record of all parties' activities and document discussions and statements, should a record be needed for use in audits or dispute negotiations.

### 2.4.4 Work Quality Management

Although drawing up construction contract documents is the responsibility of the consultant, the community project managers have a responsibility to monitor the quality and quantity of the work. Monitoring, however, is not easily accomplished by a group that may have limited technical expertise and practical experience with wastewater systems. There are two reviews that can be performed by the project managers, with the help of consultants, to ensure that the principal consultant's work meets the intent of the design and additional charges are kept to a minimum. These reviews are:

- Peer Engineering Review
- Constructability Review

These reviews are generally performed by outside professionals, at an added cost to the project. The potential long-term savings in dollars and the benefits in terms of peace of mind are more than worth the cost. Potential consultants for the peer engineering review can be identified by questioning state and local regulators about the recognized experts in the technologies proposed for the project.

The objectives of the peer review are to answer the following questions:

- Is the design capable of performing as intended (i.e., consider present vs. design population analyses)?
- Has the engineer omitted required treatment steps?
- Are the design parameters accurate?
- Are there redundant or needless treatment steps or equipment?
- Does any of the equipment selected lack sufficient durability to serve the community adequately?
- Does the equipment or process require special or costly maintenance?
- Can the same treatment objectives be met in a less costly manner?

Even community wastewater management systems employing unsophisticated technologies can benefit from peer review. Consider, for example, a proposal to repair or replace individual septic systems in a community. Questions the reviewer may address for a simple septic tank alone are:

- How are watertight pipe connections to be made?

- How are tank sections to be sealed?
- Is the cleanout access sufficient for proper maintenance?
- Is the chosen tank material the best for the conditions in the locality? Would plastic, fiberglass, or concrete be better?
- Are any protective coatings required inside or outside the tank?
- Are the materials specified for gas baffles, tees, risers, or covers durable and not easily corroded by septic sewage?
- Will the tank be susceptible to flotation at any time?
- Are the tank backfill requirements suitable?
- What provisions have been made to prevent settling of the inlet and outlet pipes.
- Is the tank properly vented?

There are many questions to be asked about the other septic system components as well. Large or more complex systems have an even greater need for scrutiny.

Such questions may not be obvious to all design consultants. Since money is less available for wastewater systems for small communities and since lower system costs are achieved by using packaged systems and locally available or innovative equipment, financial constraints promote the selection of lower cost systems.

There are few validated national design standards for small package systems or for equipment, such as septic tanks, commonly used in small community systems. Many package systems employ patented components for which performance data are available only from the manufacturer. Without third-party verification, such data should be treated with caution.

The septic tank example illustrates the point regarding design standards. Septic tanks represent one of the oldest wastewater treatment technologies. They are manufactured throughout the country, primarily using concrete. Most states lack a structural standard for concrete septic tank design. While most sewer piping used today is plastic, plastic pipe does not bond to concrete and few standards for watertight tank-to-pipe connections are known. Other septic tank materials also have incomplete structural design requirements. Polyethylene and fiberglass tank designs, for instance, have had numerous problems, but thus far no state has adopted a structural standard.

Peer engineering review then is a matter of prudence and insurance to reduce the risk to the small community of using less-expensive and untested technologies.

The constructability review may be performed as part of the peer review task or separately. It is recommended, however, that separate advice from experienced contractors be secured for maximum benefit.

The object of a constructability review is to determine if the system plans or specifications contain requirements that are impossible or too costly to implement and for which there are lower-cost or more feasible substitutes. An example of the kind of issue addressed by a constructability review is the appropriateness of chosen septic tank construction materials. Assume, for instance, that the bid date and completion date for an onsite system repair or septic tank effluent collection system project require work through the spring or wet season. Assume also that the soils in the community have a large percentage of silt and clay and that the water table is high during the wet season. Finally, assume that concrete septic tanks are specified. The constructability reviewer in this case might find the following:

- Most yards are likely to be too soft during the wet season to support standard septic tank delivery trucks.
- The construction methods required to overcome this problem are likely to be more expensive than the methods required during drier seasons.
- Construction during the wet season is likely to cause greater collateral damage to properties, thus increasing the cost of restoration.

The constructability reviewer might recommend a longer contract period with a planned shutdown to overcome the problem and associated increased costs, or might recommend the use of lighter, more easily handled fiberglass or polyethylene tanks.

The potential return on the cost of peer and constructability reviews varies from project to project. In general, however, such reviews reduce the potential risk and liability that construction bidders associate with projects involving unfamiliar technologies and conditions. For example, consider the bid prices of septic tank effluent systems when they were first introduced. The specifications usually required that the private property on which the tanks and service laterals were placed be restored to their original condition or better upon completion. Contractors bidding the projects were faced with the dilemma of entering into a contract with one party (the community) to perform work on the land of another party (the homeowner). The contract requirements were vague in stating that restoration was to equal preexisting conditions; the quality of preexisting conditions, however, was a subjective requirement and subjected the contractor's work to the judgment of a homeowner with whom the contractor has no contract. Further, the contracts carried no indemnification of the contractor from suits resulting from disputes over the adequacy of landscape restoration. There were other shortcomings in the early contracts, but these two glaring omissions resulted in inflated bids to cover the cost of the risk the contractors assumed or felt that they were assuming. As contractor experience grew with successive biddings and contracts were more carefully crafted, the bid prices of septic tank effluent system construction contracts dropped dramatically.

Had these early projects been peer and constructability reviewed many of the contract and design provisions that contributed to contractor apprehension would have been removed.

Concurrent with the management of the design phase, it is important that the community management group begin planning for the operation of the facility during this phase. Although the consultant will be expected to prepare an acceptable O&M manual for the system, input and review by the community is important for the following reasons:

- Wastewater facility operating costs will continue to be incurred for years after construction is completed. The design and its effect on these costs should be reviewed constantly to ensure that what is planned as an environmental enhancement does not lead to a chronic fiscal problem.
- Many operating activities can be designed out of facilities. The level of operator attention required affects operating costs. The community representatives should have input into this decisionmaking.
- The community must develop an understanding of the skills and number of individuals required to operate and maintain the facility in order to prepare accurate operating cost projections.

Specific areas that the management committee should concern itself with are:

- Durability of materials, equipment, finishes
- Availability/access to spare parts
- Access to all components
- Safety provisions and requirements
- Energy requirements
- Life expectancy of equipment
- Requirement for special tools
- Potential disasters and effects
- Emergency operations
- Monitoring and controls

Special knowledge is not required to address these issues. Questions relative to these issues and about potential alternatives to the design should be asked, the cost impact evaluated, and guidance given to the designer so that the constructed facility is appropriate given the needs, desires, and finances of the community.

## 2.5 Construction

The construction phase of the project involves more groups and individuals and more money over a short period than all the other phases combined. The participants are:

- The community representative
- The design consultant
- The construction manager (optional, function may be carried out by design consultant)
- The contractor(s)
- The subcontractors
- Funding agencies
- Regulatory agencies

Management of this group would be difficult enough were they all linked contractually. Unfortunately, they are linked in various ways that could allow for chaos if the group is not carefully managed. A typical organization chart is presented in Figure 2-1.

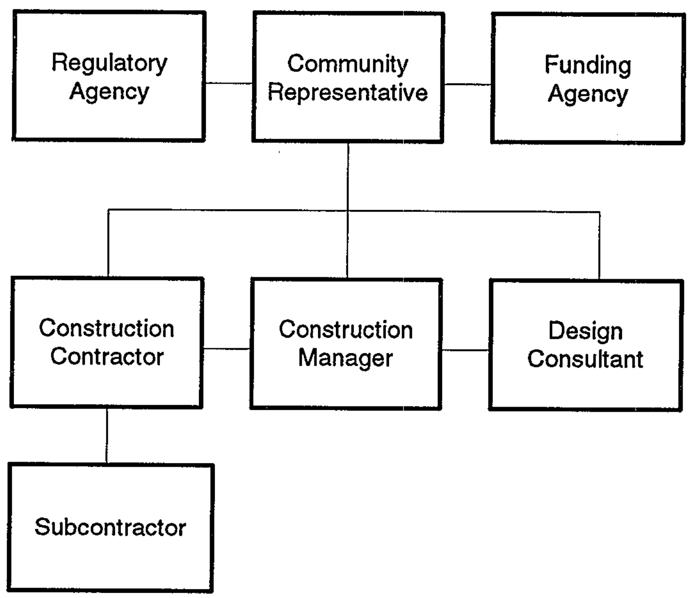


**Figure 2-1. Typical Management Organization for Construction of a Small Community Wastewater Project.**

The management functions during this phase include:

- Paying the project bills
- Maintaining records of funds expended
- Ensuring that money will be available when needed
- Functioning as a public liaison
- Reviewing and approving change orders
- Making cash-flow projections

- Monitoring the schedule and status of the work to be sure that work complies with the funding agencies' conditions
- Monitoring the work to ensure that the terms and conditions of regulatory approvals are met
- Preparing progress reports as may be required by funding and regulatory agencies
- Monitoring the quantity and quality of the work
- Resolving disputes as they arise

Even though much of the detailed owner responsibility may be delegated to a construction manager, the construction period requires substantial effort on the part of the community representative. The primary demands are processing payment requests, attending periodic (monthly) job meetings, reviewing change orders, functioning as a public liaison, facilitating dispute resolution, and carring out operational planning.

### 2.5.1 Functioning as a Public Liaison

Public liaison responsibilities of the community representative include conducting a preconstruction community meeting and responding to questions from the public throughout the construction period. The use of all available local media for providing progress updates and notification of public hearings is vital to success.

The preconstruction public meeting can often be combined with the design completion meeting. In relative terms, the preconstruction public meeting is the more important of the two meetings, especially when a sewer system is to be constructed or upgraded as part of the project. Sewer construction brings the public into direct contact with the construction contractor since it involves working on public roads. Sewer construction tends to slow traffic and leaves roads in disrepair for extended periods. Projects may also dry up wells and interrupt cable-TV service, electrical power, or water. Construction also may require that individuals make arrangements with a contractor for connection to the wastewater collection system.

Preconstruction public meetings for treatment systems primarily serve to inform the public of project activity and to maintain a high level of awareness.

The public meeting should include the following elements:

1. Summation of project history
2. Description of the planned facilities and their expected performance
3. Description of the construction process, schedule, and anticipated effects on the community
4. Introduction of project participants, including the contractor, construction manager, and community representative
5. Designation of contact representatives for the various participants. Designation of the types of complaints and concerns for which each participant is responsible. Distribution of worksite address and phone numbers for responsible participant representatives and after-hours emergency phone numbers.

The matter of jobsite contacts for the public deserves additional comment. Many communities designate one person or entity as the point of contact to which the public is directed to refer all complaints and inquiries. This is recommended, if only because it permits the establishment and maintenance of a tracking system that ensures a response to all public contacts.

### 2.5.2 Change Order Review

It is rare that a project will be executed precisely as planned or as bid, and project changes always have a cost associated with them. Therefore the evaluation and negotiation of change orders are an important management function of the community representative. If the development of change orders is delegated to a construction manager, then review of the manager's recommendation is an important function.

Although a detailed discussion of change order procedures is beyond the scope of this volume, a few major concepts include the following:

- If a change is unavoidable, don't delay approval. Some communities consume valuable time trying to associate the change with a regulatory requirement or design oversight in order to avoid the cost by transferring liability. This process only delays the contractor and can add more cost to the contract than the value of the change because of the costs of delay. The job requirements should be addressed quickly. Liability or other concerns should be addressed separately.
- Get expert help when negotiating costs. Contractors generally know their actual costs since their business depends on it. Small communities don't contract for wastewater systems everyday and may be at a disadvantage at the negotiating table. Therefore the community is advised to employ expert assistance when valuing changes to the work. A construction manager usually provides this service. If the manager recommends a price for a change, be sure backup calculations and an explanation are provided.
- Don't let haggling over the price delay the work. The community should keep in mind that time is money and any delay to the project adds cost. When the delay is caused by the owner, the cost is passed on by the contractor.

Finally, well-designed and specified projects will have fewer unforeseen problems. If the total value of change orders exceeds 5 percent of the construction budget, the planners and designers have not performed well overall.

### 2.5.3 Processing Payment Requests

The prompt review and payment of contractor requisitions is the single and simplest action the community representative can take to ensure a smooth construction project. Cutoff dates for contractor billings should be coordinated with community finance board meetings to ensure quick payment. The community must never transfer its cash-flow problems to the contractor by delaying his payment; for instance, when loan payments, grant payments, or other funding has not been received. The community must update cash outflow and inflow plans regularly and take necessary steps to ensure timely contractor payments.

### 2.5.4 Attendance at Progress Meetings

The job progress meeting is a formal procedure established to bring all project participants together on a regular basis to discuss all aspects of the work project. It is the main mechanism by which the community and its representatives keep up with the details of the project.

The meeting should have a formal agenda and detailed minutes should be kept. Attendees should include:

- The community representative (the owner)
- The contractor
- The design consultant(s)
- The construction manager
- The funding agency representative (if possible)
- Regulatory agency representatives (if possible)
- Utility representatives
- Traffic control representatives
- Police
- Interested citizens

A typical agenda might include:

**Acceptance of previous meeting minutes**

**Schedule review**

1. Describe work completed since previous meeting
2. Compare total work to date against contractor schedule
3. Compare contractor payments to date against projected cash flow
4. Agree what action if any is necessary to correct incongruities in the proposed versus actual work and cash-flow schedules
5. Agree on revised schedules and cash-flow projections
6. Describe and discuss in detail activities that will occur in the next period

**Shop drawing review**

All projects have a list of detailed equipment descriptions and component configurations that are required to be submitted and approved before the contractor proceeds with certain work. The list of submittals and their approval status is compared to the completed and proposed work. Problems relative to the adequacy of the contractor's submissions and the timeliness of the review by the design consultant or construction manager are resolved.

**Change order review**

The change orders issued, both pending and anticipated, are reviewed for their content and cost. Although the monthly meeting is not the proper forum for negotiation, it can be used to move intractable issues because all the interests involved are generally represented at the table. Change order review allows the owner to keep a current total of the cost of project extras.

**Review of jobsite visitations**

All projects have a constant stream of official and unofficial visitors and inspectors. These visitors often converse with and question the inspectors, foremen, laborers, and operators on the job. These informal contacts may result in important findings and directives being lost or not communicated to the responsible individuals. This review attempts to overcome problems caused by lack of communication. One of the most important sources of comment is complaints or questions from the citizenry.

**Review of jobsite problems**

This review addresses past and anticipated construction problems and is an advantageous forum for their resolution because of the array of interests present. The discussion can save the owner many dollars otherwise lost to lack of productivity and in potential change orders. However, even more important than the construction-related problems, this segment of the meeting permits exposure of work problems associated with friction among the project participants so that they can be dealt with before they turn into full-blown disputes.

### 2.5.5 Facilitating Dispute Resolution

Disputes arise in all construction projects. The rapidity and skill with which they are addressed have a major impact on the cost and outcome of the project. All construction contracts have a dispute resolution clause. These clauses, however, should be reserved for the most egregious disputes, after all efforts to reach a compromise and conciliation have failed.

Disputes can usually be attributed to some omission, inconsistency, error, or vagueness in the plans or specifications for the project. They can also arise from friction between project participants.

When the community representative becomes aware of a dispute he or she must take action immediately and per-severe until a resolution is achieved. While the art of dispute settlement is beyond the scope of this work, there are a few pertinent points to consider.

1. Put the disputed issues in writing and get the opposing parties to agree on the points. This single act will facilitate dispute resolution by allowing the issues to stand out.
2. Get the disputants to talk with each other to resolve their differences.
3. Set deadlines for action. Don't let the dispute fester for a long period.
4. Get actively involved in the negotiations only when the impasse is judged to be permanent.
5. Take action. In almost all cases the least costly resolution is readily available.
6. Document the actions taken and the reasons for them.

### 2.5.6 Operational Planning

Operational planning for small community systems continues through the construction phase. This effort has two main components:

1. The assembly of a thorough and usable O&M manual.
2. The preparation of a thorough startup plan.

The technical preparation required for an O&M manual is usually performed by the design consultant. The construction contractor supplies the O&M manuals for the facility's individual components, such as valves, flow meters, and generators. It is the responsibility of the community representative to review and evaluate these documents for their accuracy, clarity, and completeness. Technical assistance is available from regulatory agencies and operations consultants. It should not be left to the system operations staff to discover O&M manual shortcomings long after the consultants and contractor are gone.

Startup planning is an area that is often overlooked. Most small community systems are designed to serve previously unsewered areas. As such, flow to the treatment plant has usually not begun upon completion of the project. As the house-by-house connections to the system are made, flow increases from zero to approach design levels. This scenario presents two startup problems:

1. How is the finished treatment system to be evaluated for compliance with design requirements without flow?
2. Since most treatment systems are designed to operate within certain flow and contaminant concentration limits, how will performance—compliance with operating permits—be achieved with flows and contaminant loads outside the design ranges?

Answers to these questions are as varied as the number of collection and treatment options. The design consultant should be charged with developing a startup plan, acceptance criteria, and an interim operating plan for the connection period. Assistance in evaluating the plan is available from regulatory agencies, other utilities, and operating consultants.

The plan for operational startup must address the issues of performance and permit compliance during what may be a protracted period. Physical/biological systems, such as sand filters, have a relatively short startup period and a broad operating range. Biological systems using activated sludge processes have somewhat longer startup periods; they are dependent upon a supply of wastewater nutrients to perform properly and have a much narrower operating range. Wetland systems, which are coming into frequent use in small communities' facilities, have a very broad operating range but may require a year or more to develop the necessary plant community. Sewer systems with small flows may have such long detention times in pump station wetwells that offensive odors are produced.

The design consultant's plan to address such startup issues might be included in the O&M manual or, since it may be a one-time process, in a separate document. It is the community representative's responsibility to be aware of the requirements for startup and to make the necessary staffing and resource commitments to carry out the plan.

In preparation for conditional acceptance and startup, the community representative should begin to recruit operating staff early in the construction period. The number of staff persons and the training requirements should be established in a draft of the O&M manual by the design consultant prior to recruitment.

Once identified, the principal operator should be engaged, placed on the community's payroll, and assigned to work along with the construction manager and engineers through the end of construction to become familiar with the physical equipment and the O&M manual. The earlier the chief operator can be brought onto the project, the more readily the operator will be able to deal with the requirements of startup. The need for early engagement of the operator is dependent on the complexity of the system and equipment. Each community must determine its own needs.

## 2.6 Startup

The construction phase of wastewater management projects and the startup phase overlap, with *acceptance startup* commencing at the announcement that the contractor has completed work and is ready to transfer possession of the facility to the community. At this point all subcomponents will have been operationally checked. The acceptance startup will check the operation of all systems as an integrated unit. Operational startup officially commences on the day that conditional acceptance of the facility is established, but actually commences with the recruitment of competent staff, about midway through the construction period.

Acceptance startup is the responsibility of the community representative overseeing construction. Assuming that a reasonable plan for startup testing has been developed, this phase of the startup should be a straightforward exercise. The activity should conclude with the preparation for the community representative's approval of a startup and testing report, which compares, on a mechanical performance basis, the requirements of the design report with the measured performance of the completed facility. Tank sizes, pump sizes, pump output, blower sizes, blower output, weir overflow levels are a few of the items compared. The performance of biological systems is usually not compared at this time because of the need for seeding and a steady flow of wastewater to encourage the ecosystem.

The results of testing should be presented in as simple a form as possible for the community representative to evaluate prior to acceptance. All deviations from design requirements should be explained and an acceptance recommendation secured from both the design consultant and construction manager. Prior to acceptance, the community representative should read the document and understand the reasons for any variation from the design. The community representative must be aware that acceptance of the facility releases much of the financial leverage that he possesses to ensure compliance with the design requirements—bonds and guarantees notwithstanding.

Operational startup commences on the date of conditional acceptance of the completed facility. Conditional acceptance is the point at which the facility is completed and ready to be used for its intended purpose with only minor items (e.g., touchup painting, grass growth) remaining.

Public information and feedback during the startup phase are usually limited to media announcements about the startup of the facility or a formal open house. Although not crucial to the continuance of the project, the open house provides an appropriate setting to acknowledge the efforts of the various groups and individuals involved in the project and to display the fruits of the community's efforts.

## 2.7 Operation

The operation phase is the long-term continuation of operation after startup. Late in the construction phase or during startup, the permanent community organization charged with managing the operation of the facility begins its work.

Management responsibilities include:

- Establishment of annual operating budgets
- Establishment of a preventive maintenance plan
- Establishment of a long-term capital replacement plan
- Monitoring of permit compliance and reporting
- Monitoring of normal O&M
- Monitoring of staff workloads and needs

Planning responsibilities include:

- Regular review and updating of emergency operations plan
- Preparation and maintenance of capital expansion plan

Daily operations are carried out by the full- or part-time operations staff.

The entity responsible for system management should establish a formal work plan and schedule. Many small, and even some large, wastewater systems have failed to perform or survive through their intended life cycle because of inadequate management attention or the delegation of responsibilities to the operations staff.

It is often difficult for managers to sustain their attention to well-designed, simple, and reliable systems because during the early years, after stable operations are established, they demand little attention. Management alertness can be maintained by scheduling, at a minimum, quarterly quality-control and budget reviews. Larger, more complex systems will require monthly evaluations. Sample agenda items are listed below.

**A. Facility Performance**

1. Discharge standards versus discharge performance
2. Establish reasons for noncompliance
3. Establish a corrective-action plan
4. Review trends in facility performance (charts or other graphics are useful for presenting trends)
   a. seasonal
   b. long term
   c. corrective-action plans
5. Review scheduled maintenance activities
6. Review major unscheduled maintenance activities

**B. Staff**

1. Actual versus planned labor hours
2. Review training plans
   a. operations
   b. safety
3. Review licensing status

**C. Budget**

1. Planned versus actual cash flow

    a. maintenance appropriations

    b. labor cost

    c. capital expenditures

2. Prepare requests for supplemental appropriations

**D. Planning**

1. Annual budget
2. Capital improvements
3. Staff training
4. System expansion
5. User charge system review (annually)

The operations management group should retain the design consultant for guidance at least until it is comfortable with its task. When the EPA grants program was still in effect, consultants were required to be retained for a one-year period following startup for the purpose of certifying the successful performance of the project. This arrangement was useful in that it maintained a formal communications link between the community and the designer that helped resolve operations and management problems.

In summary, planning for system operations begins during the preliminary planning phase of the project, months, if not years, before operations commence. The guiding principles are that small community systems must be simple to operate, reliable, and economical. Addressing these concerns is then followed by a design effort during which specific choices are made to implement the project goals. During construction, an O&M manual is prepared and operators engaged and trained. Most operating procedures will have been tested during startup, as operators are gaining experience. The attention paid and time devoted to these tasks will greatly enhance the ability to manage the completed wastewater system.

## 2.8 References

When an NTIS number is cited in a reference, that reference is available from:

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5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650

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# CHAPTER 3

## Site Evaluation and Construction Considerations for Land Application Systems

### 3.1 Introduction

#### 3.1.1 Soil as a Treatment and Disposal Medium

Soil frequently is used to provide both treatment and disposal of wastewaters. It has a large capacity to retain, transform, and recycle many of the pollutants found in municipal wastewater. As the wastewater percolates through the soil to the ground water, physical, chemical, and biological processes occur to provide a high level of treatment consistently and reliably. Where properly applied and operated, soil-based or land application treatment systems have been shown to have relatively low capital and operating costs. Today several thousand such systems are being used successfully across the United States.

Various designs exist for land application systems. They may apply wastewater either on or below the land surface. Surface application systems include slow rate, rapid infiltration, and overland flow. Subsurface application systems include septic tank systems and subsurface infiltration systems designed for clusters of homes. The selection of a design depends on the nature of the wastewater to be treated, the characteristics of the site, and regulatory requirements. Except for overland flow, these designs are "zero discharge" systems, where ultimate disposal of the treated wastewater is to the ground water, rather than a point discharge to surface water. Therefore, to protect ground-water quality, the land application site must be carefully selected and the system design appropriately adapted.

#### 3.1.2 Importance of Site Evaluation in System Performance

Hydraulic and treatment performance of land application systems are related directly to the soil and site characteristics and the applied wastewater quality. Performance is measured by the ability of the system to accept and adequately treat the applied wastewater within the defined system boundaries. The system is commonly defined to include the unsaturated soil below the infiltrative surface, the permanent ground-water table, and a portion of the ground water horizontally extending to the property line. Typically, drinking water standards must be met for many pollutants before the wastewater/ground-water mixture leaves the system boundaries.

Site evaluation is the most critical factor in successful performance of land application systems. It must provide sufficient information to predict the capacity of the soil to accept and treat the projected wastewater loading and to assess how the soil and ground water will respond to that loading. Design selection, infiltrative surface area, elevation, geometry, and method of system operation are all dependent on information gathered during the site evaluation. Most failures of land application systems can be attributed to inadequate site evaluation and interpretation of the data collected. It is necessary that the engineer recognize the importance of a thorough site evaluation by experienced, qualified professionals.

#### 3.1.3 Importance of Construction Procedures in System Performance

Soil is a complex physical, chemical, and biological system that functions well only when the soil pores remain relatively open and continuous. Porosity and pore continuity is essential for the free movement of liquids and gases within the soil. Construction activities will destroy many of the pores through compaction, smearing, and puddling, all of which reduce the hydraulic and treatment capacity of the soil. Proper construction procedures performed under the appropriate soil moisture conditions will minimize the damage that can occur.

Soil that has been excessively damaged during construction cannot be easily repaired. Usually the system must be redesigned to account for the reduced hydraulic and treatment capacity of the soil or to avoid use of the damaged soil. Often the site is no longer suitable and must be abandoned. Therefore, the engineer must consider construction procedures during design and limit the soil moisture conditions under which a facility can be built.

### 3.2 Treatment and Disposal of Wastewater in Soil

#### 3.2.1 Soil as a Three-Phase System

Soil is a complex, heterogeneous, porous system of particulate mineral solids and organic matter in which the interfacial surface area is enormously large. It provides a three-phase treatment system: the solid phase consists of soil particles; the liquid phase consists of water (containing dissolved substances); and the gaseous phase consists primarily of soil air (and other gases).

The solid matrix of the soil consists of particles differing in chemical and mineralogical composition as well as in size, shape, and orientation. The mutual arrangement of these particles determines the characteristics of the soil pores in which water and air are transmitted or retained. The interaction of these phases in the soil directly impacts the capacity of the soil to accept and treat applied wastewater.

### 3.2.2 Water Movement in the Soil System

Water moves in soil through the soil pores in response to a potential energy gradient. The rate of flow is proportional to the permeability and the potential gradient. Soil permeability, a measure of the capability of soil to transmit fluids, is determined by the size, shape, and continuity of the pores rather than the porosity of the soil. For example, clay soil is more porous than sandy soil, but the sandy soil will transmit much more water under saturated conditions because it has larger, more continuous pores than the clay soil.

The soil moisture potential consists of several components. The primary components of interest in land application systems are the gravitational, hydrostatic (positive pressure), and the matric (negative pressure) potentials. Water will move in the direction of decreasing potential; however, its path may not be direct because of the available flow paths in the porous media.

The *gravitational potential* is the mass of the water times the acceleration due to gravity, or the weight of the water. At any point, it is determined by the elevation of the point relative to an arbitrary reference level.

The *hydrostatic potential* is a positive pressure potential that occurs below the ground-water surface. It is greater than atmospheric pressure and increases with depth below the surface due to the weight of water above.

The *matric potential* is a negative pressure potential created by the physical affinity of water for the soil particle surfaces and capillary pores. Water attempts to wet the particle surfaces due to adhesive forces between the water molecule and the surface. Cohesive force between water molecules pulls other molecules from the bulk liquid—thus a negative pressure, or suction, is created, a phenomenon referred to as *capillarity*. The movement of water stops when the column of water that is being pulled into a pore is greater than the surface tension at the air/water surface. Therefore water is pulled further and held tighter in small pores than large pores, such that large pores are the first to empty upon draining.

Under saturated soil conditions (ground water), all the soil pores are filled with water and flow occurs in response to the gravitational and hydrostatic potentials, while under unsaturated soil conditions water movement occurs in response to the gravitational and matric potentials. The most important difference between saturated and unsaturated flow is the hydraulic conductivity. Since all the pores are filled under saturated conditions, flow is maximal. However, under unsaturated conditions, some of the pores are air-filled and the conductive cross-sectional area is reduced. The first pores to empty are the largest and, therefore, the most conductive. There can be a dramatic difference (several orders of magnitude) between the saturated and unsaturated hydraulic conductivity of the soil (Figure 3-1).

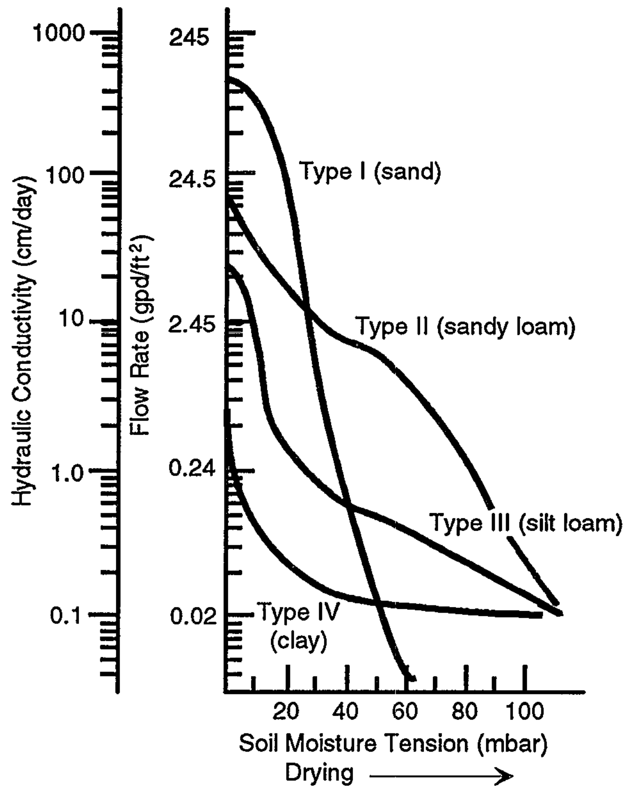


**Figure 3-1. Hydraulic Conductivity of Various Soils versus Soil Moisture Tension (EPA, 1980).**

In layered soils where the hydraulic conductivities of the layers differ, water that enters unsaturated soil will move through alternating unsaturated and saturated soil conditions as it percolates downward. Where a coarse soil with a higher saturated hydraulic conductivity overlies a finer textured soil, water will move through the coarse soil rapidly to the boundary between the two where it will be slowed to the rate the finer soil can accept. The coarse soil will saturate above the boundary with the finer soil, and the rate of water movement through the soil profile will be controlled by the saturated conductivity of the finer soil. Similarly, saturated conditions will occur above the boundary when the finer soil overlies the coarse soil. However, in this case, the soil suction or matric potential of the finer soil is greater than the force that is exerted by the larger pores of the coarse soil. The water must saturate the finer soil at the boundary before the matric potential is reduced sufficiently for the water to continue its downward path. Thus in layered profiles even a thin layer of soil with a higher or lower conductivity will impede vertical percolation of water and the rate of movement through the profile will be controlled by the soil layer with the lowest saturated hydraulic conductivity. Land-based systems have failed because seemingly minute texture changes in the soil profile were ignored or even overlooked.

### 3.2.3 Hydraulic Capacity of Soils to Accept Wastewater

Water transport from land application systems typically occurs through three zones in the soil: the infiltration zone, the vadose (unsaturated zone), and the saturated zone (Figure 3-2). Wastewater enters the soil at the surface of the infiltration zone, a biologically active zone, the thickness of which will vary with the type of land application system used. It acts as a physical, chemical, and biological filter to remove suspended solids and organics from the wastewater. The filtered solids accumulate on the infiltrative surface, within the soil pores, and on the soil matrix in this zone providing a source of food and nutrients for an active biomass. The biomass and metabolic by-products also accumulate in this zone. Hydraulically, the infiltration zone is a transitional zone where flow changes from saturated to unsaturated with a concomitant sharp decline in hydraulic conductivity because of the blockage and filling of the soil pores by the accumulated solids, biomass, and metabolic by-products.

Below the infiltration zone, the water enters the vadose zone. In this zone, the water is under a negative pressure potential or matric potential; consequently, flow occurs only in the smaller pores, while larger pores remain gas-filled. Water transport occurs primarily vertically over soil particle surfaces and through capillary pores due to

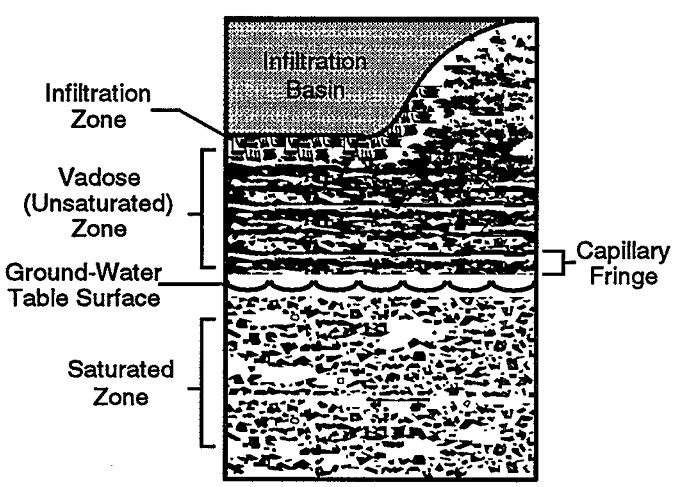


**Figure 3-2. Fluid Transport Zones through Soil below Land Application Systems.**

the gravitational potential, but the matric potential does cause some dispersive lateral flow.

Below the vadose zone, the water enters the saturated zone or ground-water table. All the soil pores are filled in this zone, and flow occurs vertically and/or horizontally in response to the gravitational and hydrostatic pressure potentials. It is in this zone that the applied wastewater ultimately leaves the site. The additional water from the wastewater treatment system causes a ground-water mound to form above the ground-water table. This mound must be taken into account in the hydraulic design to ensure that a sufficient vadose zone remains for treatment below the application area.

Soil factors that affect hydraulic performance of the land application systems include texture, structure, bulk density, horizonation (stratigraphy, layering), and soil moisture. Texture, structure, and bulk density relate directly to the size, shape, and continuity of the soil pores. Texture is defined by the relative proportion of the various sizes of the soil particulates or soil "separates" (i.e., sand, silt, and clay), while structure refers to the relative arrangement of the soil particles. Bulk density is the ratio of the soil mass to its bulk or volume. Finer textured soils such as silts and clays would be expected to have smaller, less-continuous pores than sands, but because the particles can agglomerate to form structural units or peds, large interpedal pores can exist to enhance permeability. Soils with higher bulk volumes tend to have lower permeabilities because the pore volume is less.

Horizonation (or layering) and soil moisture distribution in the soil can impact the rate of water movement through the profile. Soil layers of differing texture, structure, and/or bulk density can impede vertical percolation because of their differing hydraulic conductivities. Zones of higher soil moisture are indicative of reduced conductivity due to lower matric potentials. High moisture conditions also limit the movement of air through the soil. Soil aeration is necessary to support aerobic biochemical reactions for rapid degradation of applied wastes.

Factors in the design of land application systems also affect hydraulic performance. These include geometry and elevation relative to ground surface of the infiltrative surface, uniformity and periodicity of wastewater application, and the quality of the applied wastewater. Each factor can affect the aeration status of the soil at and below the infiltrative surface of the soil due to the oxygen demand exerted by constituents in the applied wastewater. The hydraulic capacity of the system decreases if anaerobic conditions are allowed to occur for extended periods.

### 3.2.4 Wastewater Treatment Capabilities of Soil

Physical, chemical, and biological processes occur in the soil providing effective wastewater treatment through retention, transformation, or destruction of pollutants. Physical processes include filtration, dispersion, and dilution. Volatilization, adsorption, complexation (with organic residues), precipitation, and photodecomposition are chemical processes that can occur. Several biological processes including biological oxidation (mineralization), nitrification, denitrification, immobilization, predation, and plant uptake can take place depending on the environmental conditions. These processes occur most effectively when the soil is unsaturated because the wastewater is forced to percolate over the soil particle surfaces where many of the treatment phenomena take place and air is able to diffuse through the soil. In combination, these processes can produce a treated water of acceptable quality for discharge to the ground-water table under the proper conditions.

Whether these processes occur and their effectiveness in treatment depends on the physical characteristics of the soils and the environmental conditions of the soil through which the wastewater percolates. Studies have shown that conventional wastewater parameters (e.g., biochemical oxygen demand ($BOD_5$), suspended solids (SS), and fecal indicator organisms) are nearly completely removed where an aerobic, unsaturated zone of medium-to-fine textured soil, 0.6 to 1.5 m (2 to 5 ft) in thickness, with neutral pH is maintained below the infiltrative surface during system operation. Soils with excessive permeability (coarse texture soil or soil with large continuous pores), low organic content, low pH, low cation exchange capacity and redox potential, high moisture content, and low temperature have been shown to have reduced treatment efficiency.

Other wastewater parameters, such as levels of nitrogen, phosphorus, heavy metals, toxic organics, and virus, are removed to varying degrees. Organic or ammonia nitrogen is readily and rapidly nitrified biochemically in aerobic soil. Because of its high solubility, nitrate leaches to the ground water. Some biochemical denitrification can occur in the soil, but without plant uptake, 60 to 90 percent of the nitrate enters the ground water. Under anaerobic soil conditions, nitrification will not occur, but the positively charged ammonium ion is retained in the soil by adsorption onto the soil particles. The ammonium may be held until aerobic soil conditions return allowing nitrification to occur. Phosphorus and metals can be removed through adsorption, ion exchange, and precipitation reactions, but the capacity of the soil to retain these ions is finite. The capacity varies with the soil mineralogy, organic content, pH, redox potential, and cation exchange capacity. The fate and transport of virus is not well documented. Limited data suggest some types of virus are able to leach to ground water; however, when this occurs their infectivity is soon lost. Fine-textured soil, aerobic subsoils, and high temperatures favor virus destruction. Toxic organics appear to be removed in aerobic soils, but further study of the fate and transport of these compounds is needed.

Land treatment system design and operation will also affect the treatment capability of soil. Systems that maintain unsaturated, aerobic subsoil conditions provide the most effective treatment of wastewater. Geometry of the infiltrative surface, elevation of the surface relative to final grade, wastewater mass loading rates, and uniformity and periodicity of wastewater application influence the degree of saturation and aeration status of the subsoil. The loading of oxygen-demanding materials per unit area must not exceed the rate at which oxygen can be supplied to the subsoil. Where the infiltrative surface is exposed to the atmosphere, mass flow of air will occur behind the wetting front as the wastewater infiltrates the soil. Thus the rate of application and the period between applications are important operating parameters. In subsurface systems, however, diffusion from the perimeter of the system is the primary pathway of oxygen to the subsoil. Therefore shallow, narrow infiltrative surfaces, controlled mass loadings, and uniform, periodic applications will enhance aeration.

The most significant impacts that land application systems are known to have on the quality of ground water are increased concentrations of nitrogen (primarily nitrate), chlorides, and total dissolved solids; temperature may also be impacted. Once the treated wastewater enters the ground water, it does not readily mix with the ground water but remains as a distinct plume for as much as several hundred feet. Solute concentrations within the plume can remain above ambient groundwater concentrations. Dilution of the plume is dependent on the quantity of natural recharge and travel distance from the source.

### 3.2.5 Hydraulic and Treatment Performance of Land Application Systems

There are significant differences in design and performance between the various types of land application systems because of the differences in application and operation. A comparison of the systems is presented in Table 3-1.

## 3.3 Approach to Site Evaluation

Successful performance of a land application system for wastewater treatment depends upon the characteristics of the site on which it is built and how its design and operation are adapted to those characteristics. The design engineer must understand how the soil, ground water, vegetation, and other site factors will impact system performance so that design features and operation procedures can be adopted to ensure acceptable performance. Therefore, careful site evaluation is critical.

The scale and detail of the site evaluation depend on the quantity and quality of the wastewater and the characteristics of the site to which the wastewater is to be ap-

**Table 3-1. Design and Treatment Performance Comparisons for Land Application Systems for Domestic Wastewater**

| Feature | Slow Rate | Rapid Infiltration | Subsurface Infiltration | Overland Flow |
|---|---|---|---|---|
| **Site Conditions** | | | | |
| Soil texture | Sandy loams to clay loams | Sands, sandy loams | Sands to clay loams[a] | Silt loams, clay loams |
| Depth to ground water[b] (m) | 1.0 | 1.0 | 1.0 | Not critical[c] |
| Vegetation | Required | Optional | Not applicable | Required |
| Climatic restrictions | Growing season[d] | None | None | Growing seasond |
| **Design Loadings** | | | | |
| Pretreatment[e] | Primary sedimentation[f] | Primary sedimentation[f] | Primary sedimentation | Primary sedimentation |
| Average daily loading (cm) | 1.2 – 1.5 | 1.5 – 10 | 0.2 – 4.0[g] | 1.0 – 6.0 |
| Application method | Sprinkler or flooding | Flooding | Flooding | Sprinkler or flooding |
| Disposition of wastewater | Evapotranspiration and percolation | Percolation | Percolation | Surface runoff and evapotranspiration |
| **Treatment Performance** | | | | |
| $BOD_5$ (mg/L) | 5 | 10 | 5 | 15 |
| SS (mg/L) | 5 | 5 | 5 | 20 |
| Total nitrogen as N (mg/L) | 3 – 8[h] | 10 – 20i | 25 – 35[i] | 5 – 10[i] |
| Total phosphorus as P (mg/L) | 0.1 – 0.4 | 1 – 2 | 0.1 – 0.5 | 4 – 5 |
| Toxic organics[j] | ? | ? | ? | ? |
| Fecal coliforms (per 100 mL) | < 10 | < 200 | < 10 | < 2000 |
| Virus, log removal average | ≈ 3+ | ≈ 2 | ≈ 3 | < 1 |
| Metals (%) | High | Medium | High | Low |

[a]Applies to single or small cluster household systems; larger systems limited to sands and sandy loams (where significant, depth to top of ground-water mound).
[b]Minimum separation distance from infiltration surface to highest ground-water mound elevation.
[c]Critical only if significant percolation occurs.
[d]Application during few weeks before and after growing season.
[e]Minimum pretreatment requirements.
[f]With restricted public access; crops not for direct human consumption.
[g]Loading based on trench bottom area, not total site area.
[h]Varies with applied concentration and crop.
[i]Varies with applied concentration.
[j]Data are limited, but good removals (>90%) appear to occur at low application rates in aerobic soils for biodegradable organics, adsorbed species are removed effectively until the underlying soil column becomes saturated, whereupon removals cease; volatiles are removed effectively in the unsaturated soil zone if rates are sufficiently low.

plied. Because of the many factors involved, site evaluation for land application systems requires the input of a multidisciplinary team of qualified engineers, soil scientists, hydrogeologists, and agronomists experienced in the siting, design, and management of such systems. It may involve extensive, and in some cases expensive, site investigations and testing.

To avoid unnecessary effort and expense, the site evaluation should focus on only the most promising sites. Therefore, the evaluation effort should be phased. A three-phase approach is suggested.

The first phase includes the identification and screening of potential sites. Efforts during this phase are limited to review of available resource materials and the development of site screening criteria based on knowledge of the wastewater source and its characteristics. This phase concludes with the elimination of sites that do not meet the established screening criteria.

During phase two, a reconnaissance survey of the potential sites identified in the first phase is made. A visual survey and preliminary soil borings are performed to confirm the potential of each site. This phase also includes a preliminary layout of an appropriate system design to determine if sufficient suitable area exists to construct the system. On the basis of the findings during this phase, the sites are ranked according to priority for conducting detailed site evaluations.

The final phase is the detailed investigation. It is during this phase that the critical site characteristics that affect site suitability and system design and operation are identified. The investigation typically includes soil profile descriptions, deep soil borings, ground-water assessments and testing, soil permeability measurements, and topographic mapping. Only the highest rated site need be investigated unless the investigation indicates that it should not be ranked above the others or that it should be eliminated from consideration. At that point, the next ranked site should be investigated. This procedure is followed until a suitable site is selected.

## 3.4 Site Identification

### 3.4.1 Wastewater Characteristics

The projected wastewater volume and characteristics impact both site suitability and cost. Site suitability is frequently determined by the estimated hydraulic capacity of the site to accept the projected wastewater flows. Where wastewater flow data are not available, per capita flows of 200 to 400 L/d (50 to 100 gpd) are typically used to estimate average daily flow. A factor of 1.8 to 2.0 is often used to estimate the peak daily flows. The parameters used to estimate flows for a particular application depend on the characteristics of the community, the climate, and the condition of the collection system. If significant commercial and/or industrial development exists in the community, daily wastewater flows should be estimated for each establishment and added to the residential estimate.

Conventional wastewater parameters such as BOD and SS seldom limit land application system feasibility or capacity. (Typical BOD loading rates for municipal wastewater systems are presented in Table 3-2.) However, wastewater constituents such as nitrogen, phosphorus, potassium, chlorides, total dissolved solids, pH, and trace elements may override hydraulic considerations in determining land area requirements or selecting a suitable land application design.

**Table 3-2. Typical BOD Loading Rates for Land Application Systems for Treatment of Municipal Wastewater[a]**

| Technology | (kg/ha/yr) | (lb/ac/yr) |
|---|---|---|
| Slow rate | 370 – 1,830 | 2,000 – 10,000 |
| Rapid infiltration | 8,000 – 40,000 | 44,000 –253,000 |
| Subsurface infiltration | 5,500 – 22,000 | 30,000 – 121,000 |
| Overland flow | 2,000 – 7,500 | 11,000 – 41,000 |

[a] EPA, 1980 and 1981; WPCF, 1990; Reed, 1988.

Frequently, nitrogen controls system sizing and, hence, land area requirements. Excessive nitrogen contamination of potable ground-water aquifers below land application systems often is a concern, but this threat can be controlled by limiting the nitrogen loading or promoting nitrogen removal through system design and operation. Nitrate nitrogen concentrations in the ground water at the project property boundary should not exceed the drinking water standard of 10 mg-N/L. To meet this limit, the total nitrogen loading rather than the hydraulic loading may determine system sizing, potentially increasing land area requirements. The nitrogen removal potential of the system depends on the crop grown, if any, and system management practices, which vary with the type of system selected.

In some cases, other wastewater constituents such as phosphorus or trace elements control system sizing and design. For example, if wastewater trace element concentrations exceed the maximum recommended concentrations for irrigation water (Table 3-3), slow-rate land application systems may not be feasible or may require special operating conditions. Most land application systems, however, will be controlled by hydraulic or nitrogen loadings.

**Table 3-3. Comparison of Trace Elements in Wastewaters to Recommended Limits for Irrigation Water[a]**

| Element | Untreated Wastewater[b] (ppm) | Recommended Maximum Concentrations for Irrigation Water[c] (ppm) |
|---|---|---|
| Arsenic | 0.003 | 0.1 |
| Boron | 0.3 – 1.8 | 0.5 – 2.0 |
| Cadmium | 0.004 – 0.14 | 0.01 |
| Chromium | 0.02 – 0.70 | 0.1 |
| Copper | 0.02 – 3.36 | 0.2 |
| Iron | 0.9 – 3.54 | 5.0 |
| Lead | 0.05 – 1.27 | 5.0 |
| Manganese | 0.11 – 0.14 | 0.2 |
| Mercury | 0.002 – 0.044 | No standard |
| Nickel | 0.002 – 0.105 | 0.2 |
| Zinc | 0.03 – 8.31 | 2.0 |

[a]EPA, 1981.
[b]Range of reported values. See EPA, 1981.
[c]Based on unlimited irrigation at 1 m/yr (3 ft/yr).

### 3.4.2 Climate

Local climate may affect system selection and land area requirements. Precipitation, evapotranspiration potential, temperature, length of the growing season, wastewater application cycles and storage requirements, and other factors will impact land area requirements and the type of system selected. These factors should be considered as candidate sites are selected because of the influence they have on the suitability of a site for land application of wastewater.

### 3.4.3 Review of Resource Materials

Many resource materials are readily available to assist in identifying suitable sites for a land application system. They should be used to select candidate sites before field investigations are initiated. The materials include soil surveys, U.S. Geological Survey (USGS) quadrangles, well logs, geologic maps, hydrologic and ground-water maps, land-use and zoning maps, and National Oceanographic and Atmospheric Administration (NOAA) climatic data. Interviews with local residents knowledgeable about the area should also be included in this review.

Soil surveys are available from the local Soil Conservation Service (SCS) office. They contain a collection of aerial photographs of the mapping area, usually a county, on which the distributions of the various soil series are shown. The scales of the maps usually range from 1:31,680 to 1:15,840. A typical profile description to a depth of 1.5 m (5 ft) for each soil series is provided. The descriptions include texture, structure, consistency, color, rooting, estimated permeability, and thickness of each horizon. The surveys also provide limited information on permeability, slopes, landscape position, drainage, erosion potential, flooding potential, chemical properties, engineering properties, general suitability for locally grown crops, and management information. Where published surveys are not available, field sheets with interpretive tables may be available through the local county agent. Typical soil textures and permeabilities suitable for land application systems that can be used in site identification are presented in Table 3-4.

USGS quadrangles provide information on topography, landscape position, rock outcrops, wetlands, regional drainage patterns, and surface water elevations. These maps, which are usually drawn to a scale of 1:24,000 (7.5 minute series) or 1:62,000 (15 minute series), can help identify potential sites based on landscape position, slope, drainage, and separation from the ground water. Landscape positions best suited for land application systems include hilltops, ridge lines, and sideslopes. Depressions and footslopes of hills should be avoided. Slopes less than 12 percent are generally preferred. Ground-water gradients and elevations can be inferred from the elevations shown for surface water bodies.

Well logs are generally available locally and are an excellent source of information on soil profile and potential impermeable layers.

Existing land use also is an important factor in selecting candidate sites. Land use of both the identified parcel and the surrounding parcels should be determined. Land use can be tentatively determined from the soil survey maps, USGS quadrangles, and local zoning maps.

The location of the service area relative to potential treatment sites is an important economic consideration. The distance and elevation of the treatment site relative to the service area directly impact wastewater transmission costs. As a general guideline, sites within 10 to 12 km (6 to 7 mi) and less than 60 m (200 ft) above the elevation of the pumping station may still be competitive with other treatment alternatives.

### 3.4.4 Site Screening

Site screening criteria should be established according to which potential sites can be identified and ranked. The key criteria may be broken down into environmental-physical characteristics, economic and institutional considerations, and other technical and engineering factors. In developing these criteria, federal, state, and local regulatory requirements must be considered for any imposed limitations such as siting and design requirements, water quality requirements, and monitoring requirements. A numerical approach using qualitative factors for the key selection criteria should be used to rank sites. This will focus the following site investigation activities on the sites with the most potential.

Criteria should include distance and elevation from source, available area, soils, landscape position, slope, ownership, and land use. The second phase of evaluation activities should be carried out only with the highest ranked sites.

## 3.5 Site Reconnaissance

### 3.5.1 Visual Inspection

Site reconnaissance begins with a visual survey of each of the sites identified from a review of the resource materials. The objective of the visual survey is to verify the site information collected from the resource materials and to note any general features that may affect site suit-

**Table 3-4. Typical Soil Textures Suitable for Land Application Systems[a]**

| | Technology | | | |
|---|---|---|---|---|
| **Soil Parameter** | **Slow Rate** | **Rapid Infiltration** | **Subsurface Infiltration** | **Overland Flow** |
| Textural class[b] | Clay loams to sandy loams | Sand to sandy loams | Sand and sandy loams | Clay and clay loams |
| Unified soil class[c] | GM, SM, ML OL, MH, PT | GW, GP, SW, SP | GW, GP, SW, SP | GM, GC,SM, SC, CL OL, CH, OH |

[a]EPA, 1981.
[b]USDA-SCS System (SCS, 1951).
[c]Group symbols are from ASTM Unified Classification System.

ability or system design. It is important to walk the site noting significant features including topography, landscape position, vegetation, land use surrounding the site, and others that may affect site suitability or system design and performance.

Site topography impacts system design and surface and subsurface drainage. Long, planar slopes or large, level areas allow greater flexibility in system layout than steeply sloping or hummocky sites. Sloping sites are usually better drained than large, level sites. Drainage is an important factor when considering the contribution of surface runoff to the hydraulic load of the system and the response of the ground water below the site to system loading.

Landscape position is an important consideration in site drainage. Although sites may have favorable topography, the position of the site relative to the surrounding land form may impact site suitability. For example, hilltops and sideslopes can be expected to have good surface and subsurface drainage, while depressions or footslopes are more likely to drain poorly. There are known exceptions to these generalities, such as alluvial soils in the valleys of arid regions; however, without information on the particular locality, these rules of thumb serve as effective initial planning tools.

The type and extent of vegetation on the site provide an indication of soil depth and subsurface drainage. It also may impact site suitability directly. For instance, vegetation associated with wet soils suggests that subsurface drainage is not good, and numerous large trees may make system construction difficult.

Surrounding land use is a necessary consideration. Concerns for the fate of aerosols, odors, and ground-water drainage from the site must be addressed. Adequate buffer areas will need to be provided from developed areas.

Other significant features to observe include areas of flooding, surface water, rock outcrops, wells, roads, buildings, and buried utilities. Each of these will help to determine site suitability or system design. Surface water bodies, for instance, would suggest the need to investigate the potential water-quality and hydrologic impacts of the system. Rock outcrops would raise concerns about soil depth. Water supply wells raise concerns about ground-water flows and potential ground-water-quality impacts.

#### 3.5.2 Preliminary Soil Borings

Preliminary soil borings should be performed on sites that appear to be suitable following the visual survey. The purpose of the borings is to provide information regarding suitability and variability of the surface soils and to identify the presence of any shallow, impermeable layers. The number and density of these borings should be made to provide sufficient information regarding the soil properties and variability necessary to determine whether more detailed site investigations are warranted. A boring density of one per 0.2 ha (0.5 ac) is usually adequate to make this determination. If the soil variability is such that a greater density of borings is necessary to characterize the site, consideration should be given to eliminating the site from further evaluation.

Soil characteristics from the surface to approximately 2 m (6 ft) below the anticipated infiltrative surface elevation should be observed. Important characteristics to note include soil texture, structure, horizon thickness, moisture content, color, bulk density, and spatial variability.

A hand-held soil probe or auger is sufficient for these borings unless it is anticipated that the infiltrative surface of the system will be placed deep within the soil profile. If a deep system is anticipated, a truck-mounted soil probe may be necessary. Excavated pits or deep soil borings typically are not performed during this phase because of the expense and damage of sites to which no commitment has been made.

#### 3.5.3 Conceptual System Design

While on the site, a conceptual layout of the proposed land application system should be made based on the knowledge of the site and projected wastewater flows. This is done to determine if the site is of sufficient size and to assess where more detailed site investigation efforts are needed if the site is selected for further consideration.

#### 3.5.4 Site Screening

The information gained from the reconnaissance survey should be used to revise the site ranking made during the first phase of the site evaluation. It is important that no potential sites be eliminated too early in the process; however, to limit the expense of the site selection process, sites must be carefully ranked so that further investigative efforts proceed systematically beginning with the highest ranked site.

### 3.6 Detailed Site Investigations

#### 3.6.1 Soil Profile Descriptions

The soil is a critical factor in determining site suitability and system design. It is the primary determinate of the hydraulic and treatment capacity of the site. The characteristics of the soil profile must be described in detail to observe and identify features that may affect the capacity of the site to accept and treat the applied wastewater. Any soil layer that may affect vertical percolation of water should be described even if less than 1 cm in thickness. The description should include texture, structure, particle sorting, coarse fragment content, relative density, macroporosity, rooting patterns, thickness, transition, and continuity for each identifiable soil horizon.

The characteristics of the surface soil materials to a depth of 2 to 5 m (5 to 15 ft) are the most critical for determining site suitability. Backhoe-excavated test pits and soil borings extending to a depth of 2 to 3 m (6 to 10 ft) below the proposed infiltrative surface elevation should be used. Backhoe pits are the most effective for exposing an undisturbed soil profile. However, unless appropriate safety precautions can be taken, hollow-stem-auger and split-spoon samples may be required.

Locations of test pits and borings should be carefully planned in advance based on information gained during the resource materials review and reconnaissance survey. The number and density of test pits and borings should be sufficient to enable accurate prediction of soil characteristics and variability, but not so excessive that unnecessary damage to the proposed infiltration area results. Where spatial variability is not great, a test pit density of one per 0.4 ha (1 ac) is usually adequate. Each test pit and boring should be carefully located on a plot plan and the surface elevation measured relative to a permanent bench mark.

A qualified soil scientist familiar with land application systems should describe the soil profile morphology. Particular emphasis should be placed on those features that affect water movement through the profile. U.S. Department of Agriculture Soil Conservation Service nomenclature, rather than the Unified System, is typically used to describe the soil characteristics in the zone of natural soil development, typically within 3 m (10 ft) of the ground surface. The USDA system, as described in the SCS Soil Survey Manual (1951), provides a more comprehensive description to aid in understanding morphological details relevant to water movement.

### 3.6.2 Characterization of the Unconsolidated Substratum

The deeper unconsolidated materials below the site must be examined also. Hydraulically restrictive layers may exist that would cause "perched" saturated zones to form under system operation. Because these zones could affect the hydraulic and treatment capacity of the site, they must be identified.

Hollow stem auger borings are used to examine the deep unconsolidated materials to depths of 3 to 8 m (10 to 25 ft) or to bedrock, whichever is shallower. The borings should be made near the center and at the perimeters of the proposed infiltration area. A minimum of three borings should be made across the site. More may be necessary for large sites or to assess any suspected ground-water boundary zones that may impact system operation, but the maximum density of borings usually need not be more than one per 0.4 ha (1 ac).

Split-spoon samples should be taken at maximum intervals of 1.5 m (5 ft) and described sufficiently to assess internal drainage. All pertinent details of morphology, lithology, and physical characteristics should be noted. Characteristic descriptions that are of particular importance are texture, consistence, blow counts, moisture content, estimated permeability, and the presence of free water. Borings are typically logged using the Unified System (ASTM) nomenclature.

Each boring should be located accurately on a plot plan of the site and the surface elevation established relative to a permanent bench mark.

### 3.6.3 Characterization of the Ground-Water Table

If ground water is present within 8 m (25 ft) of the ground surface, the hydraulic response of the water table to prolonged system loading should be evaluated. The primary concerns are the direction of ground-water flow from the site, potential development of a ground-water mound below the site that encroaches within the vadose zone needed for treatment, and water-quality impacts.

Characterization of the ground water to address these concerns may be accomplished by the installation of monitoring wells and/or piezometers. A minimum of three wells or piezometers that penetrate the water table by at least 1 to 1.5 m (3 to 5 ft) should be installed in the shallowest saturated zone in a triangular pattern across the site. After allowing the water surface to equilibrate in the well, water elevation measurements are taken and compared to establish the horizontal gradient of the ground water. On large sites, more piezometers are usually needed to allow a better estimate of the gradient. At least one nested group of three piezometers at different depths should be installed and monitored to determine the vertical ground-water gradient. All wells and piezometers should be fitted with a protective casing with a locking cover to prevent vandalism and foreign substances from entering the well.

To evaluate the mounding potential, the depth to ground water, ground-water gradient, hydraulic conductivity, effective porosity or specific yield, and thickness of the saturated zone are needed. The hydraulic conductivity can be determined by performing slug tests or pumping tests in one or more wells on the site (Bouwer, 1978; Bouwer and Rice, 1976; Freeze and Cherry, 1979). In some cases, it may be possible to estimate the saturated hydraulic conductivity from a particle size analysis of the aquifer materials (Bouwer, 1978; Freeze and Cherry, 1979). Pumping tests may also be used to determine the effective porosity or specific yield of the saturated zone.

One well should be installed upgradient of the proposed system to monitor ambient ground-water quality. The location of this well can only be determined after the ground-water gradient has been established. At a minimum, water quality monitoring should include field measurement of specific conductance, pH, and temperature, and laboratory analyses for total dissolved solids, total organic carbon, nitrogen species, phosphorus, chloride, and alkalinity. After system construction, this well can serve as the background monitoring well for assessing treatment performance.

If the ground water does not occur within 8 m (25 ft) of the ground surface, it may be appropriate to assume a "worst case" scenario to avoid the cost of deep well installation. This can be done by assuming that an impermeable barrier exists at that depth for the mounding analysis. A similar assumption can be made where ground water is encountered but the bottom of the aquifer cannot be determined. Again, by assuming an impermeable barrier at this depth, the worst case scenario is accepted in determining the thickness of the aquifer and the mounding analysis.

Once installed, the wells and piezometers should be carefully located on an accurate plot plan and the ground and top of pipe elevation measured to a permanent bench mark.

### 3.6.4 Measurement of Hydraulic Properties of the Soil

Knowledge of the capacity of the soil to accept and transmit water is critical to the design of land application systems. Infiltration rates and hydraulic conductivities should be measured in the field to provide the data necessary for determining design hydraulic loadings.

The infiltration rate of soil is defined as the rate at which water enters the soil from the surface, while the hydraulic conductivity is defined as the rate at which water moves through the soil. The initial infiltration rate of a clean, exposed soil surface can equal the saturated hydraulic conductivity of the soil below, but with time, the infiltration rate typically declines well below the capacity of the soil to transmit the water. Infiltration rate decline is caused by deposition of wastewater SS at the infiltrative surface, growth of active biomass stimulated by the carbon and nutrient rich wastewater, and changes in the characteristics of the soil. As a result of the decline in the infiltration rate, the subsoil cannot maintain saturation, and unsaturated conditions prevail. The unsaturated conductivity of soil is one to four orders of magnitude less than the saturated conductivity. Therefore the saturated hydraulic conductivity of the soil cannot be used directly to determine the long-term infiltration rate for design purposes.

There are both field and laboratory methods for estimating infiltration rates. Field methods provide better estimates and, therefore, are preferred. They include flooding basin, single- or double-ring infiltrometer, sprinkler infiltrometer, and air-entry permeameter tests. These and other test procedures are described elsewhere (Black, 1965a; EPA, 1980; 1981; 1984; ASTM).

Double-ring infiltrometer and basin tests are most commonly used for land application system design. Infiltrometer tests are small-scale tests that measure the hydraulic conductivity of the soil over an area approximately 50 cm (1.5 ft) in diameter. Such tests can be easily used to estimate the saturated hydraulic conductivity at several locations and soil horizons over the site. Basin tests are large-scale tests that are typically run over an area 3 m (10 ft) in diameter. Basin tests provide more accurate measurements of hydraulic conductivity, but require much larger volumes of water to run. Therefore they are usually run on only the most restrictive soil horizons in the design area.

Laboratory methods for estimating hydraulic conductivity are described elsewhere (Black, 1965a; ASTM). Because only small, disturbed soil samples can be tested, the results are much less accurate than field tests. Methods that minimize disturbance of the soil sample and best reproduce field operational conditions are preferred. The concentric-ring permeameter (Hill and King, 1982) and the cube method (Bouma and Dekker, 1981) are the most useful techniques.

### 3.6.5 Analysis of Soil Chemical Properties

Soil chemical properties can affect permeability and crop growth potential. Chemical properties of interest include pH, cation exchange capacity (CEC), exchangeable sodium percentage (ESP), and electrical conductivity (EC). Soil pH has a significant effect on the solubility of various compounds, microorganism activity, and ion exchange capacity. The CEC is a measure of the capacity of the soil to adsorb exchangeable cations. The sum of the exchangeable sodium, potassium, calcium, and magnesium expressed as a percentage of the CEC is called percent-base saturation. There are optimum ranges for percent-base saturation for various crop and soil type combinations. Also, high percentages of sodium and potassium will result in significant reductions in permeability due to soil swelling. ESP should be kept below 15 percent. EC is a diagnostic tool used to determine salinity of the soil water. Salinity will affect plant growth and the type of plants that will thrive. Analytical procedures for measuring soil chemical properties can be found elsewhere (Black, 1965b; Jackson, 1958; Walsh and Beaton, 1973).

### 3.6.6 Topographic Mapping

A topographic survey of the site should be completed in sufficient detail to produce accurately a topographic map with 25 to 50 cm (1 to 2 ft) contour intervals. All soil borings, soil pits, monitoring wells, and piezometers should be located on this map with horizontal and vertical positions established relative to a permanent bench mark. Significant site features such as surface water, rock outcrops, and large trees that may affect system layout and design should also be accurately located on this map.

### 3.6.7 Data Interpretation

Interpretation of the site data collected can be a complex and difficult task. The interpretation process must establish the elevation of the infiltrative surface and the acceptable ranges of wastewater loadings that can be applied. It must also establish the acceptable boundaries of the system and its geometry.

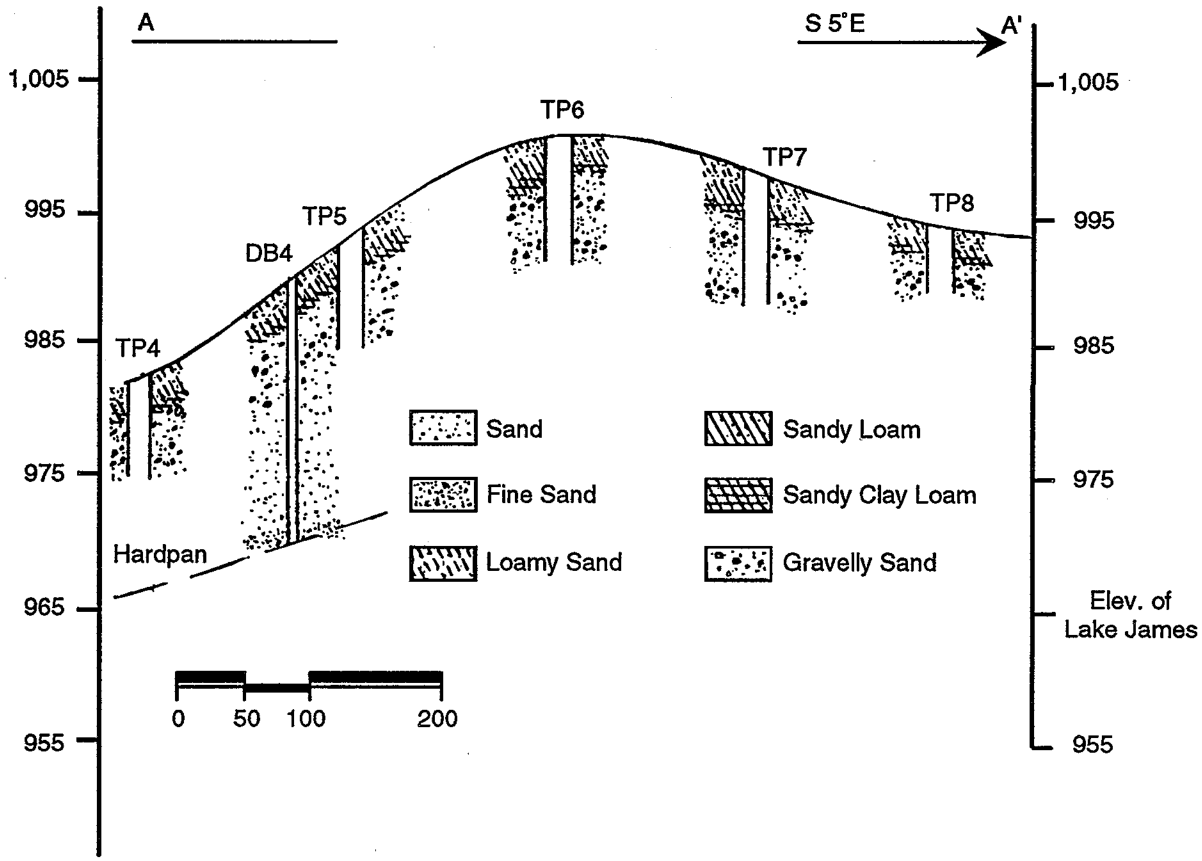


**Figure 3-3. Example of Stratigraphic Cross Section Constructed from Soil-Boring Log Data.**

Stratigraphic cross sections of the site are useful to determine the most appropriate elevation of the infiltrative surface. Using data from borings and pits, enough cross sections should be constructed along various transects of the site to depict the continuity and interrelationships of key strata affecting system design and operation (Figure 3.3).

The interpretation process must take into account the type of land application design that is proposed for the site because required site characteristics vary with system type. The designer is referred to Chapter 5 for description of the required site conditions for each system type.

Where ground water or a hydraulically restrictive horizon exists within 8 m (25 ft) of the proposed elevation of the infiltrative surface, the response of the ground water to system loading must be analyzed. Both analytical and numerical ground-water mounding models are available. Because of the large number of data points necessary for numerical modeling, analytical models are most commonly used. Analytical models have been developed for various hydrogeologic conditions (Brock, 1976; Finnemore and Hantzshe, 1983; Hantush, 1967; Kahn et al., 1976). The assumptions used for each model must be compared to the specific site conditions found to select the most appropriate model. (For examples of model selection and their computations, see EPA 1981; 1984.)

## 3.7 Construction Considerations

Methods used to construct land application systems are critical to the long-term performance of the installations. Earthmoving operations and general construction traffic will damage the soil properties that are necessary for favorable wastewater infiltration and percolation. Careful planning of system construction can minimize soil damage, maintaining the capacity of the system to accept and treat the applied wastewater.

Soils are not equally susceptible to construction damage. The degree of soil compaction, smearing, and puddling that occurs is a function of the soil texture and moisture content. Soil with greater than 25 percent by weight of clay (all soils excepting sands and loamy sands) is the most susceptible to damage.

To limit damage to the soil, a construction plan should be developed for each project. The plan should address type of equipment to be used, methods and sequence of construction operations, site traffic and materials stockpiling, site preparation, and soil conditions during construction operations.

Only low-load-bearing equipment should be used in the design area. High-flotation tires and track-mounted vehicles are preferred. Scrapers or blades should not be used near the infiltrative surface because the scraping action can smear the soil. In areas to be filled with sand, blades may be used to spread the fill material if a minimum of 30 cm (1 ft) of fill is maintained below the treads of the vehicle.

Construction methods and the sequencing of operations must be carefully planned in advance to prevent unnecessary damage to the soil. Unlike conventional earthwork practices, soil compaction is to be avoided. The number of passes required to be made over the design area to reach the desired grade and elevation should be kept to a minimum. Once the infiltrative surface is established, further traffic over the area should be avoided. This requires that the construction of large systems be sequenced to limit unnecessary traffic. Testing of the soils during construction may be included in the construction specifications as a measure of contractor performance.

Site traffic must be controlled at the outset. Site access and acceptable traffic lanes and stockpiling areas for construction machinery and materials delivery should be delineated on the construction drawings. The site should be staked accordingly before work commences. Approaches to the design area should be made from the upslope side wherever possible. If the approach must be made from the downslope side, traffic should be kept to minimum and routed as far from the area as possible.

Brush and tree removal may be required prior to other earthwork. Grubbing of the site following tree removal should not be done since it will damage the structure of the soil and aggravate soil compaction during subsequent operations. If the area is to be filled, the site should be mowed and raked before chisel plowing to a depth of 20 to 30 cm (8 to 12 in.) along the slope contour.

The soil conditions during construction should be carefully monitored. If the soil is near its plastic limit or if the soil is frozen within 30 cm (1 ft) of the infiltrative surface finish elevation, construction should not proceed.

If the soil is damaged to the point that the infiltration rate is significantly affected, the damaged soil must be removed to expose an undamaged surface. As much as 10 to 15 cm (4 to 6 in.) may have to be removed to restore the infiltrative capacity of the infiltrative surface. This will lower the finished elevation and may require adjustments in the design.

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When an NTIS number is cited in a reference, that reference is available from:

National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650


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# CHAPTER 4
## Wastewater Characteristics

## 4.1 Introduction

The effective management of any wastewater flow requires a reasonably accurate knowledge of its characteristics. This is particularly true for flows from rural residential dwellings, commercial establishments, and other facilities where individual water-using activities create an intermittent flow of wastewater that can vary widely in volume and degree of pollution. For small communities with collection systems, the possibility of a significant infiltration and inflow (I/I) contribution must be taken into account for any wastewater planning effort. Detailed characterization data regarding these flows are necessary not only to facilitate the effective design of wastewater treatment and disposal systems, but also to enable the development and application of water conservation and waste load reduction strategies.

For existing developments, characterization of the actual wastewaters to be encountered can often be carried out. However, for many existing developments, and for almost all new developments, wastewater characteristics must be predicted. The purpose of this chapter is to provide a basis for characterizing the wastewater from rural developments. A detailed discussion of the characteristics of residential wastewaters is presented first, followed by a limited discussion of the characteristics of the wastewaters generated by nonresidential establishments, including those of a commercial, institutional, and recreational nature. Finally, a general procedure for predicting wastewater characteristics for a given residential dwelling or nonresidential establishment is given.

## 4.2 Residential Wastewater Characteristics

Residential dwellings exist in a variety of forms, including single- and multi-family homes, condominiums, apartment houses, and cottages or resort residences. In all cases, occupancy can occur on a seasonal or year-round basis. The wastewater discharged from these dwellings is comprised of a number of individual wastewaters generated through water-using activities employing a variety of plumbing fixtures and appliances. The characteristics of the wastewater can be influenced by several factors. Primary influences are the characteristics of the plumbing fixtures and appliances present as well as their frequency of use. Additionally, the characteristics of the residing family in terms of the number of resident family members, age levels, and mobility are important as is the overall socioeconomic status of the family. The other characteristics, including seasonal or yearly occupancy, geographic location, and method of water supply and wastewater disposal, appear as additional, but lesser, influences.

### 4.2.1 Wastewater Flow

#### *4.2.1.1 Average Daily Flow*

The average daily wastewater flow from a typical residential dwelling is approximately 170 Lpcd (45 gpcd) (Table 4-1). While the average in-house daily flow experienced at one residence compared to that of another can vary considerably, it is typically no greater than 227 Lpcd (60 gpcd) and seldom exceeds 284 Lpcd (75 gpcd) (Figure 4-1). Residential water use in arid regions of the country can be significantly greater owing to on-lot demands, which nationally average nearly one-half of the residential (in-house) demand cited above. These figures are even higher in the Southwest.

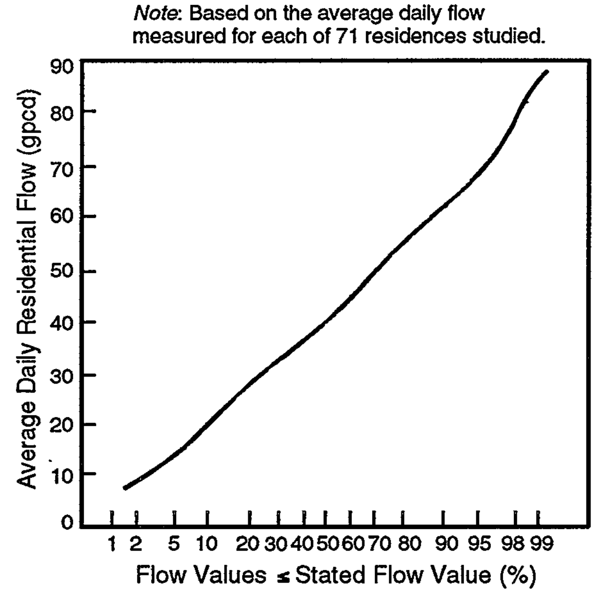


**Figure 4-1. Frequency Distribution for Average Daily Residential Water Use/Waste Flows.**

**Table 4-1. Summary of Average Daily Residential Wastewater Flows**

| Study[a] | Number of Residences | Duration of Study (months) | Wastewater Flow: Study Avg. (gpcd) | Wastewater Flow: Range of Individual Residence Averages (gpcd) |
|---|---|---|---|---|
| Linaweaver et al. | 22 | – | 49.0 | 6 – 66 |
| Anderson and Watson | 18 | 4 | 44.0 | 18 – 69 |
| Watson et al. | 3 | 2–12 | 53.0 | 25 – 65 |
| Cohen and Wallman | 8 | 6 | 52.0 | 27.8 – 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 | 36.0 | 8 – 71 |
| Duffy et al. | 16 | 12 | 42.3 | – |
| **Weighted Average** | | | **44.0** | |

[a]See References at end of chapter.

#### *4.2.1.2 Individual Activity Flows*

The individual wastewater generating activities within a residence are the building blocks that serve to produce the total residential wastewater discharge. The average characteristics of several major residential water-using activities are presented in Table 4-2. A water-using activity that falls under the category of miscellaneous in this table, but deserves additional comment, is water-softener backwash/regeneration flows. Water-softener regeneration typically occurs once or twice a week, discharging about 114 to 333 L (30 to 88 gal)/regeneration cycle (Welckart, 1976). On a daily per capita basis, water-softener flows have been shown to average about 19 Lpcd (5 gpcd), ranging from 8.7 to 59.4 Lpcd (2.3 to 15.7 gpcd) (Siegrist et al. 1976).

**Table 4-2. Typical Residential Water Use by Activity[a]**

| Activity | Gal/use | Uses/cap/d | gpcd[b] |
|---|---|---|---|
| Toilet flushing | 4.3<br>4.0 – 5.0 | 3.5<br>2.3 – 4.1 | 16.2<br>9.2 – 20.0 |
| Bathing | 24.5<br>21.4 – 27.2 | 0.43<br>0.32 – 0.50 | 9.2<br>6.3 – 12.5 |
| Clotheswashing | 37.4<br>33.5 – 40.0 | 0.29<br>0.25 – 0.31 | 10.0<br>7.4 – 11.6 |
| Dishwashing | 8.8<br>7.0 – 12.5 | 0.35<br>0.15 – 0.50 | 3.2<br>1.1 – 4.9 |
| Garbage grinding | 2.0<br>2.0 - 2.1 | 0.58<br>0.4 – 0.75 | 1.2<br>0.8 – 1.5 |
| Miscellaneous | NA | NA | 6.6<br>5.7 – 8.0 |
| **Total** | NA | NA | **45.6**<br>**41.4 - 52.0** |

[a]Mean and ranges of results reported in Cohen and Wallman, 1974; Laak, 1975; Bennett and Linstedt, 1975; Siegrist et al., 1976; and Ligman et al., 1974.
[b]gpcd may not equal gal/use multiplied by uses/cap/d due to difference in the number of study averages used to compute the mean and ranges shown.
NA = not applicable

#### *4.2.1.3 Wastewater Flow Variations*

The intermittent occurrence of individual wastewater-generating activities creates large variations in the wastewater flow rate from a residence.

*Minimum and Maximum Daily Flows.* The daily wastewater flow from a specific residential dwelling is typically 10 to 300 percent of the average daily flow at that dwelling, with the vast majority within 50 and 150 percent of the average day. At the extreme, however, minimum and maximum daily flows of 0 and 900 percent of the average daily flow may be encountered (Anderson and Watson, 1976; Watson et al., 1967; Witt, 1974).

*Minimum and Maximum Hourly Flows.* Minimum hourly flows of zero are typical. Maximum hourly flows are more difficult to quantify accurately. On the basis of typical fixture and appliance usage characteristics, as well as an analysis of residential water usage demands, maximum hourly flows of 380 L (100 hr)/gal can occur (Anderson and Watson, 1967; Jones, 1974). Hourly flows in excess of this can occur due to plumbing fixture and appliance misuse or malfunction (e.g., faucet left on or worn toilet ball cock).

*Instantaneous Peak Flows.* The peak flow rate from a residential dwelling is a function of the characteristics of the fixtures and appliances present and their position in the overall plumbing system layout. The peak discharge rate from a given fixture/appliance is typically around 0.3 L/s (5 gpm), with the exception of the tank-type water closet that discharges a short-duration peak flow of up to 1.6 L/s (25 gpm). The use of several fixtures/appliances simultaneously can increase the total flow rate over the rate for isolated fixtures/appliances. However, attenuation occurring in the residential drainage network tends to decrease the peak flow rates in the sewer exiting the residence.

Although field data are limited, peak discharge rates from a single-family dwelling of 0.3 to 0.6 L/s (5 to 10 gpm) can be expected. For multifamily units, peak rates in excess of these values commonly occur. A crude estimate of the peak flow in these cases can be obtained using the fixture-unit method described in Section 4.3.1.2.

### 4.2.2 Wastewater Quality

#### *4.2.2.1 Average Daily Flows*

The characteristics of typical residential wastewater are outlined in Table 4-3, including daily mass loadings and pollutant concentrations. The wastewater characterized is typical of residential dwellings equipped with standard water-using fixtures and appliances, excluding garbage disposal(s), that collectively generate approximately 170 Lpcd (45 gpcd).

#### *4.2.2.2 Individual Activity Contributions*

Residential water-using activities contribute varying amounts of pollutants to the total wastewater flow. The individual activities may be grouped into three major wastewater fractions: garbage disposal wastes, toilet wastes, and sink, basin, and appliance wastewaters. A summary of the average contribution of several key pollutants in each of these three fractions is presented in Table 4-4.

With regard to the microbiological characteristics of the individual waste categories, studies have demonstrated that the wastewater from sinks, basins, and appliances can contain significant concentrations of indicator organisms as total and fecal coliforms (Olsson et al., 1968; Hypes et al., 1974; Small Scale Waste Management Project, 1978; Brandes, 1978). Traditionally, high concentrations of these organisms have been used to assess the contamination of a water or wastewater by pathogenic organisms. One assumes, therefore, that these wastewaters possess some potential for harboring pathogens.

**Table 4-3. Characteristics of Typical Residential Wastewater[a]**

| Parameter | Mass Loading (gm/cap/d) | Concentration (mg/L) |
|---|---|---|
| Total solids | 115 – 170 | 680 – 1,000 |
| Volatile solids | 65 – 85 | 380 – 500 |
| SS | 35 – 50 | 200 – 290 |
| VSS | 25 – 40 | 150 – 240 |
| $BOD_5$ | 35 – 50 | 200 – 290 |
| OD | 115 – 125 | 680 – 730 |
| Total N | 6 – 17 | 35 – 100 |
| Ammonia | 1 – 3 | 6 – 18 |
| Nitrites and nitrates | <1 | <1 |
| Total P | 1 – 2 | 6 – 12 |
| Phosphate | 0.3 – 1.5 | 2 – 9 |
| Total coliforms[b] | 1,010 – 1,012 | |
| Fecal coliforms[b] | 108 – 1,010 | |

[a]For typical residential dwellings equipped with standard water-using fixtures and appliances (excluding garbage disposals) generating approximately 170 Lpcd (45 gpcd). Based on the results presented in Laak, 1975; Bennett and Linstedt, 1975; Siegrist et al., 1976; Ligman et al., 1974; and Jones, 1974.
[b]Concentrations presented in organisms/L.

#### *4.2.2.3 Wastewater Quality Variations*

Since individual water-using activities occur intermittently and contribute varying quantities of pollutants, the strength of the wastewater generated from a residence fluctuates with time. Accurate quantification of these fluctuations is impossible. An estimate of the type of fluctuations possible can be derived from the pollutant concentration information presented in Table 4-5 considering that the activities included occur intermittently.

## 4.3 Nonresidential Wastewater Characteristics

The rural population, as well as the transient population moving through the rural areas, is served by a wide variety of isolated commercial establishments and facilities. For many establishments, the wastewater-generating sources are sufficiently similar to those in a residential dwelling that residential wastewater characteristics can be applied. For other establishments, however, the wastewater characteristics can be considerably different from those of a typical residence.

Providing characteristic wastewater loadings for "typical" nonresidential establishments is a very complex task due to several factors. First, there is a relatively large number of diverse establishment categories (e.g., bars, restau-

**Table 4-4. Pollutant Contributions of Major Residential Wastewater Fractions[a] (gm/cap/d)**

| Parameter | Garbage Disposals | Toilets | Basins, Sinks, Appliances | Approximate Total |
|---|---|---|---|---|
| $BOD_5$ | 18.0 | 16.7 | 28.5 | 63.2 |
| | 10.9 – 30.9 | 6.9 – 23.6 | 24.5 – 38.8 | |
| SS | 26.5 | 27.0 | 17.2 | 70.7 |
| | 15.8 – 43.6 | 12.5 – 36.5 | 10.8 – 22.6 | |
| N | 0.6 | 8.7 | 1.9 | 11.2 |
| | 0.2 – 0.9 | 4.1 – 16.8 | 1.1 – 2.0 | |
| P | 0.1 | 1.2 | 2.8 | 4.0 |
| | 0.1 – 0.1 | 0.6 – 1.6 | 2.2 – 3.4 | |

[a]Means and ranges of results reported in Laak, 1975; Bennett and Linstedt, 1975; Siegrist et al., 1976; Ligman et al., 1974; Olsson et al., 1968

**Table 4-5. Pollutant Concentrations of Major Residential Wastewater Fractions[a] (mg/L)**

| Parameter | Garbage Disposals | Toilets | Basins, Sinks, Appliances | Approximate Total |
|---|---|---|---|---|
| $BOD_5$ | 2,380 | 280 | 260 | 360 |
| SS | 3,500 | 450 | 160 | 400 |
| N | 79 | 140 | 17 | 63 |
| P | 13 | 20 | 26 | 23 |

[a]Based on the average results presented in Table 4-4 and the following wastewater flows: Garbage disposals—8 Lpcd (2 gpcd); toilets—61 Lpcd (16 gpcd); basins, sinks, and appliances—110 Lpcd (29 gpcd); total—178 Lpcd (47 gpcd).

rants, drive-in theaters). The inclusion of diverse establishments within the same category produces a potential for large variations in waste-generating sources and the resultant wastewater characteristics. Further, many intangible influences, such as location and popularity, may produce substantial wastewater variations between otherwise similar establishments. Finally, there is considerable difficulty in presenting characterization data in units of measurement that are easy to apply yet predictively accurate (e.g., at a restaurant, wastewater flow in volume/seat is easy to apply to estimate total flow, but is less accurate than if volume/meal served were used).

In this section, limited characterization data for nonresidential establishments, including commercial establishments, institutional facilities, and recreational areas, are presented. These data are meant to serve only as a guide, and as such should be applied cautiously. Wherever possible, characterization data for the particular establishment in question, or a similar one in the vicinity, should be obtained.

### 4.3.1 Wastewater Flow

#### *4.3.1.1 Average Daily Flows*

Typical daily flows from a variety of commercial, institutional, and recreational establishments are presented in Tables 4-6 through 4-8.

#### *4.3.1.2 Wastewater Flow Variation*

The wastewater flows from nonresidential establishments are subject to wide fluctuations with time. While difficult to quantify accurately, an estimate of the magnitude of the fluctuations, including minimum and maximum flows on an hourly and daily basis, can be made if consideration is given to the characteristics of the water-using fixtures and appliances, and to the operational characteristics of the establishment (e.g., hours of operation, patronage fluctuations).

**Table 4-6. Typical Wastewater Flows from Commercial Sources[a]**

| | | (gpd/unit) | |
|---|---|---|---|
| Source | Unit | Range | Typical |
| 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 | 580.0 |
| | 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[b] | Meal | 9.0 – 12.0 | 10.0 |
| Rooming house | Resident | 23.8 – 50.1 | 39.6 |
| Shopping center | Parking space | 0.5 – 2.1 | 1.1 |
| Store, department | Toilet room | 423 – 634 | 528.0 |
| | Employee | 7.9 – 13.2 | 10.6 |

[a]Metcalf and Eddy, 1979.
[b]Does not include all of the facility's wastewater streams, only those related to customers.

**Table 4-7. Typical Wastewater Flows from Institutional Sources[a]**

| Source | Unit | (gpd/unit) Range | Typical |
|---|---|---|---|
| Hospital, medical | Bed | 132 – 251 | 172.0 |
| | Employee | 5.3 – 15.9 | 10.6 |
| Hospital, mental | Bed | 79.3 – 172 | 106.0 |
| | Employee | 5.3 – 15.9 | 10.6 |
| Prison | Inmate | 79.3 – 159 | 119.0 |
| | Employee | 5.3 – 15.9 | 10.6 |
| Rest home | Resident | 52.8 – 119 | 92.5 |
| | Employee | 5.3 – 15.9 | 10.6 |
| School, boarding | Student | 52.8 – 106 | 74.0 |
| 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 |

[a]Metcalf and Eddy, 1979.

**Table 4-8. Typical Wastewater Flows from Recreational Sources[a]**

| Source | Unit | (gpd/unit) Range | Typical |
|---|---|---|---|
| Apartment, resort | Person | 52.8 – 74 | 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 (developed) | 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 | 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 – 687 | 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 |

[a]Metcalf and Eddy, 1970.

Peak wastewater flows can be estimated utilizing the fixture-unit method (Water Pollution Control Fed., 1976; Uniform Plumbing Code, 1976). As originally developed, this method was based on the premise that under normal usage, a given type of fixture had an average flow rate and duration of use (Hunter, 1940; 1941). One fixture unit was arbitrarily set equal to a flow rate of 0.5 L/s (7.5 gpm), and various fixtures were assigned a certain number of fixture units based upon their particular characteristics (Table 4-9). On the basis of probability studies, relationships were developed between peak water use and the total number of fixture units present (Figure 4-2).

### 4.3.2 Wastewater Quality

The qualitative characteristics of the wastewaters generated by nonresidential establishments can vary significantly between different types of establishments due to the extreme variation that can exist in the waste-generating sources present. Consideration of the waste-generating sources present at a particular establishment can give a general idea of the character of the wastewater and serve to indicate whether the wastewater will contain any problem constituents, such as high grease levels from a restaurant or lint fibers from a laundromat.

**Table 4-9. Fixture Units per Fixture[a]**

| Fixture Type | Fixture Units |
|---|---|
| One bathroom group consisting of tank-operated water closet, lavatory, and bathtub or shower stall | 6 |
| Bathtub (with or without overhead shower) | 2 |
| Bidet | 3 |
| Combination sink-and-tray with food-disposal unit | 4 |
| Combination sink-and-tray | 3 |
| Dental lavatory | 1 |
| Dental unit or cuspidor | 1 |
| Dishwasher, domestic | 2 |
| Drinking fountain | 0.5 |
| Floor drains | 1 |
| Kitchen sink, domestic, with food-waste grinder | 3 |
| Kitchen sink, domestic | 2 |
| Laundry tray (1 or 2 compartments) | 2 |
| Lavatory | 2 |
| Shower stall, domestic | 2 |
| Showers (group) per head | 3 |
| Sinks | |
| Surgeon's | 3 |
| Flushing rim (with valve) | 8 |
| Service (trap standard) | 3 |
| Service (P trap) | 2 |
| Pot, scullery, etc. | 4 |
| Urinal, pedestal, siphon jet, blowout | 8 |
| Urinal, wall lip | 4 |
| Urinal stall, washout | 4 |
| Urinal trough (each 2-ft section) | 2 |
| Wash sink (circular or multiple) each set of faucets | 2 |
| Water closet, tank-operated | 4 |
| Water closet, valve-operated | 8 |

[a]Water Pollution Control Fed., 1976.

If the waste-generating sources present at a particular establishment are similar to those typical of a residential dwelling, an approximation of the pollutant mass loadings and concentrations of the wastewater produced may be derived using the residential wastewater quality data presented in Tables 4-3 to 4-5. For establishments where the waste-generating sources appear significantly different from those in a residential dwelling, or where more refined characterization data are desired, a detailed review of the pertinent literature as well as actual wastewater sampling at the particular or a similar establishment should be conducted.

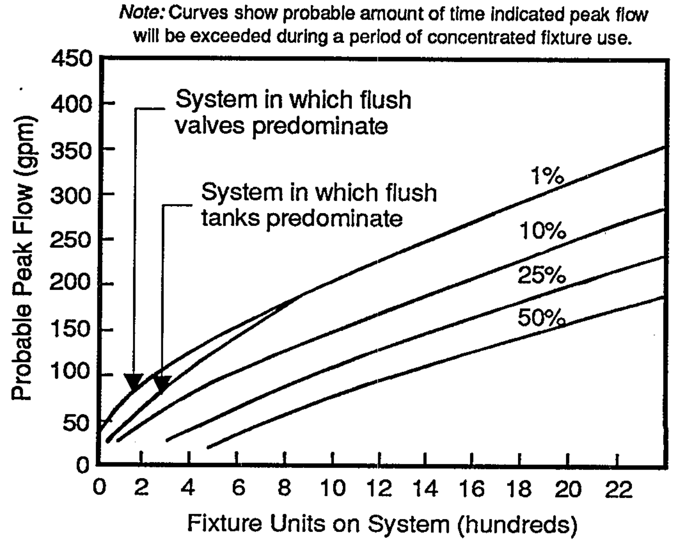


**Figure 4-2. Peak Discharge versus Fixture Units Present.**

## 4.4 Predicting Wastewater Characteristics

### 4.4.1 General Considerations

#### *4.4.1.1 Parameter Design Units*

In characterizing wastewaters, quantitative and qualitative characteristics are often expressed in terms of other parameters. These parameter design units, as they may be called, vary considerably depending on the type of establishment considered. For residential dwellings, daily flow values and pollutant contributions are expressed on a per capita basis. Applying per capita data to predict total residential wastewater characteristics requires that a second parameter be considered; namely, the number of persons residing in the residence. Residential occupancy is typically 1.0 to 1.5 persons/bedroom. Although it provides for a conservative estimate, the current practice is to assume that maximum occupancy is two persons per bedroom.

For nonresidential establishments, wastewater characteristics are expressed in terms of a variety of units. Although per capita units are employed, a physical characteristic of the establishment such as per seat, per car stall, or per square foot is more commonly used.

#### *4.4.1.2 Factors of Safety*

To account for the potential variability in the wastewater characteristics at a particular dwelling or establishment, versus that of the average, conservative predictions or factors of safety are typically utilized. These factors of safety can be applied indirectly through choice of the design criteria for wastewater characteristics and the occupancy patterns, as well as directly through an overall factor. For example, if an average daily flow of 284 Lpcd (75 gpcd) and an occupancy of two persons per bedroom were selected, the flow prediction for a three-bedroom home would include a factor of safety of approximately 3 when compared to average conditions (i.e., 170 Lpcd (45 gpcd) and 1 person/bedroom). If a direct factor of safety were also applied (e.g., 1.25), the total factor of safety would increase to approximately 3.75.

Great care must be exercised in predicting wastewater characteristics so as not to accumulate multiple factors of safety that would yield an extremely conservative estimate.

### 4.4.2 Strategy for Predicting Wastewater Characteristics

Predicting wastewater characteristics from rural developments can be a complex task. Following a logical step-by-step procedure can help simplify the characterization process and render the estimated wastewater characteristics more accurate. A flow chart detailing a procedure for predicting wastewater characteristics is presented in Figure 4-3.

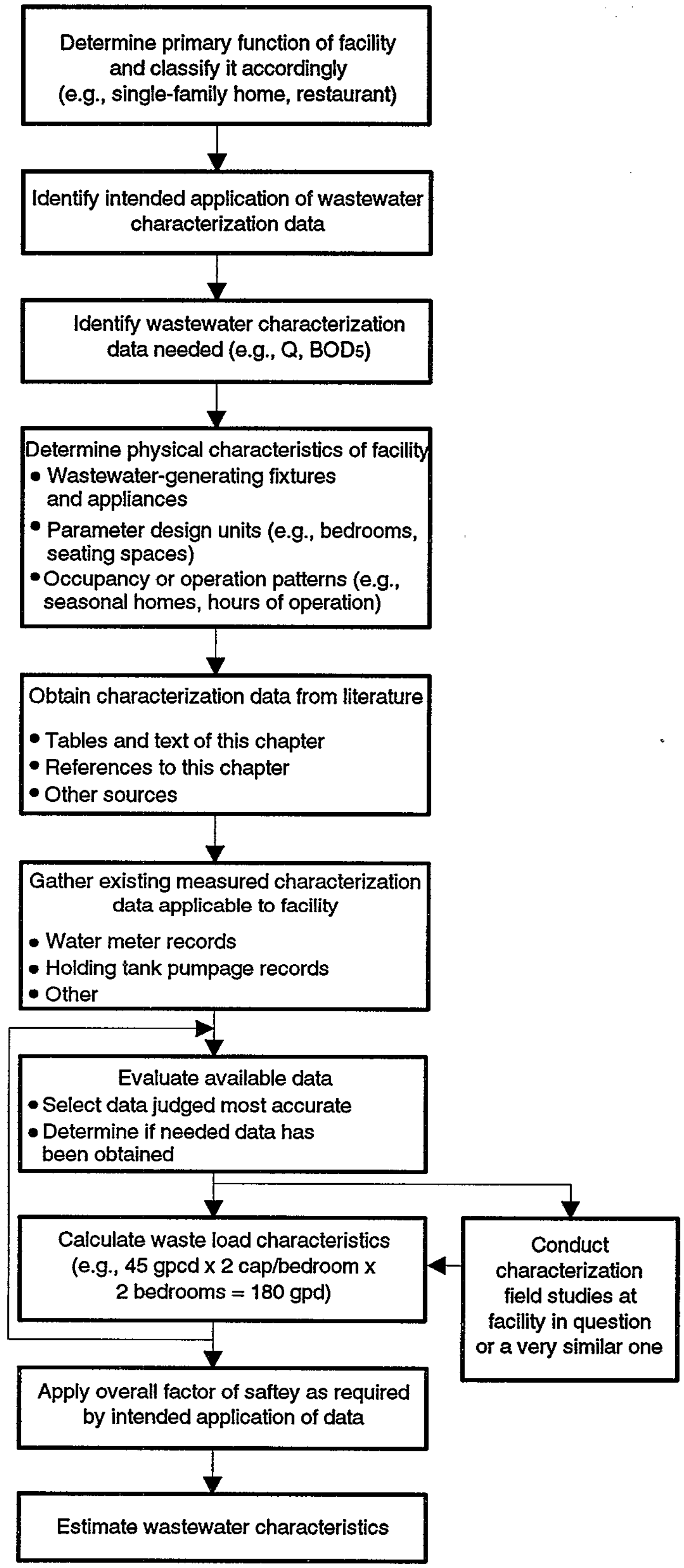


**Figure 4-3. Strategy for Predicting Wastewater Characteristics.**

## 4.5 Water Conservation and Wastewater Flow Reduction

An extensive array of techniques and devices are available to reduce the average water use and concomitant wastewater flows generated by individual water-using activities and, in turn, the total effluent from the residence or establishment. The diversity of present wastewater flow reduction methods is illustrated in Table 4-10. As shown, the methods may be divided into three major groups: elimination of nonfunctional water use; water-saving devices, fixtures, and appliances; and wastewater recycle/reuse systems.

### 4.5.1 Elimination of Nonfunctional Water Use

Wasteful water-use habits can occur with most water-using activities. A few illustrative examples include using a toilet flush to dispose of a cigarette butt, allowing the water to run while brushing teeth or shaving, or operating a clothes washer or dishwasher with only a partial load. Obviously, the potential for wastewater flow reductions through elimination of such wasteful use vary tremendously between homes, from minor to significant reductions, depending on habits.

#### *4.5.1.1 Improved Plumbing and Appliance Maintenance*

Unseen or apparently insignificant leaks from household fixtures and appliances can waste large volumes of water and generate similar quantities of wastewater. Most notable in this regard are leaking toilets and dripping faucets. For example, a steadily dripping faucet can waste up to several hundred gallons of water per day.

**Table 4-10. Selected Wastewater Flow Reduction Methods**

**Elimination of Nonfunctional Water Use**
- Improved water-use habits
- Improved plumbing and appliance maintenance and monitoring
- Reduced excessive water supply pressure

**Water-Saving Devices, Fixtures, and Appliances**
- Toilet
  1. Water-carriage toilets
     a. Toilet-tank inserts
     b. Water-saving toilets (3.5 gal.)
     c. Ultra-low flush toilets (ULF) (1.6 gal/flush or less)
        - Wash-down flush
        - Mechanically assisted
        - Pressurized tank
        - Compressed air
        - Vacuum
        - Grinder
  2. Nonwater carriage toilets
     a. Pit privies
     b. Biological toilets
     c. Incinerator toilets
     d. Oil-carriage toilets
- Bathing devices, fixtures, and appliances
  1. Shower flow controls
  2. Reduced-flow showerheads
  3. On/off showerhead valves
  4. Mixing valves
  5. Air-assisted, low-flow shower system
- Clotheswashing devices, fixtures, and appliances
  1. Front-loading washer
  2. Adjustable cycle settings
  3. Washwater recycle feature
- Miscellaneous
  1. Faucet inserts
  2. Faucet aerators
  3. Reduced-flow faucet fixtures
  4. Mixing valves
  5. Hot water pipe insulation
  6. Pressure-reducing valves
  7. Hot water recirculation

**Wastewater Recycle/Reuse Systems**
- Bath/laundry wastewater recycle for toilet flushing
- Toilet wastewater recycle for toilet flushing
- Combined wastewater recycle for toilet flushing
- Combined wastewater recycle for several uses

#### *4.5.1.2 Maintain Nonexcessive Water Supply Pressure*

The water flow rate through sink and basin faucets, showerheads, and similar fixtures is highly dependent on the water pressure in the water supply line. For most residential uses, a pressure of 280 kPa (40 psi) is adequate. Pressure in excess of this can result in unnecessary water use and wastewater generation. To illustrate, the flow rate through a typical faucet opened fully is about 40 percent higher at a supply pressure of 560 kPa (80 psi) versus that at 280 kPa (40 psi).

### 4.5.2 Water-Saving Devices, Fixtures, and Appliances

The quantity of water traditionally used by a given water-using fixture or appliance is often considerably greater than actually needed. Certain tasks may even be accomplished without the use of water. As presented in Table 4-2, over 70 percent of a typical residential dwelling's wastewater flow volume is collectively generated by toilet flushing, bathing, and clotheswashing. Thus efforts to accomplish major wastewater flow reductions should be directed toward these three activities.

#### *4.5.2.1 Toilet Devices and Systems*

Each flush of a conventional water-carriage toilet uses 15–18 L (4–5 gal) of water depending on the model and water supply pressure. On average, flushing typically generates approximately 16 L (4.3 gal) of wastewater. When coupled with 3.5 uses/cap/d, a daily wastewater flow of approximately 61 Lpcd (16 gpcd) results (Table 4-2). Since the mid-1980s 13L (3.5 gal)/flush (M) has been more common. Recently, several states have mandated that in new construction 6 L (1.6 gal)/flush toilets be installed. Nonetheless, the data above are reasonable estimates of demand per toilet flush in the United States.

A variety of devices have been developed for use with a conventional flush toilet to reduce the volume of water used in flushing. Additionally, alternatives to the conventional water-carriage toilet are available, certain of which use little or no water to transport human wastewater products. Tables 4-11 and 4-12 present a summary of various toilet devices and systems. Additional details regarding the nonwater-carriage toilets may be found elsewhere (Kreissl, 1985; Laak, 1975; Enferadi et al., 1986; Molland, 1984).

#### *4.5.2.2 Bathing Devices and Systems*

Although great variation exists in the quantity of wastewater generated by a bath or shower, typical values include approximately 95 L (25 gal)/occurrence coupled with a 0.4 use/cap/d frequency to yield a daily per capita flow of about 38 L (10 gal) (Table 4-2). The majority of devices available to reduce bathing-wastewater flow volumes are concentrated around the activity of showering, with their objective being to reduce the normal 0.25 to 0.63 L/s (4–10 gpm) showering flow rate. Several flow reduction devices and systems for showering are characterized in Table 4-13. The amount of total wastewater flow reduction accomplished with these devices is highly dependent on individual user habits. Reductions vary from a negative value to as much as 12 percent of the total wastewater volume.

**Table 4-11. Wastewater Flow Reduction—Water-Carriage Toilets and Systems**

| Generic Type | Description | Application Considerations | O&M | Water Use Per Event (gal) | Total Flow Reduction[a] (gpcd) (%) |
|---|---|---|---|---|---|
| Toilets with tank inserts | Displacement devices placed into storage tank of conventional toilets to reduce volume but not height of stored water<br><br>Varieties: Plastic bottles, flexible panels, drums or plastic bags | Device must be compatible with existing toilet and not interfere with flush mechanism<br><br>Installation by owner<br><br>Reliability low | Post-installation and periodic inspections to ensure proper positioning | 3.3–3.8 | 1.8–3.5 4–8 |
| Water-saving toilets | Variation of conventional flush toilet fixture; similar in appearance and operation. Redesigned flushing rim and priming jet to initiate siphon flush in smaller trapway with less water | Interchangeable with conventional fixture | Essentially the same as for a conventional unit | 1.0–3.5 | 3.2–13 6–20 |
| Washdown flush toilets | Flushing uses only water, but substantially less due to washdown flush<br><br>Varieties: Few | Rough-in for unit may be nonstandard<br><br>Drain-line-slope and lateral-run restrictions<br><br>Plumber installation advisable | Similar to conventional toilet<br><br>Cleaning possible | 0.8–1.6<br><br>(but more frequent) | 9.4–12.2 21–27 |
| Pressurized-tank toilets | Specially designed toilet tank to pressurize air contained in toilet tank. Upon flushing, the compressed air propels water into bowl at increased velocity<br><br>Varieties: Few | Compatible with most conventional toilet units<br><br>Increased noise level<br><br>Water supply pressure of 35–120 psi | Similar to conventional toilet fixture | 2.0–2.5 | 6.3–8.0 14–18 |
| Compressed air-assisted flush toilets | Similar in appearance and user operation to conventional toilet; specially designed to utilized compressed air to aid in flushing<br><br>Varieties: Few | Interchangeable with rough-in for conventional fixture<br><br>Requires source of compressed air; bottled or air compressor<br><br>If air compressor, need power source<br><br>Increased noise level | Periodic maintenance of compressed air source<br><br>0.002 kWh per use | 0.5–1.5 | 13.3 30 |

**Table 4-11. Wastewater Flow Reduction—Water-Carriage Toilets and Systems (continued)**

| Generic Type | Description | Application Considerations | O&M | Water Use Per Event (gal) | Total Flow Reduction[a] (gpcd) | (%) |
|---|---|---|---|---|---|---|
| Vacuum-assisted flush toilets | Similar in appearance and user operation to conventional toilet; specially designed fixture is connected to vacuum system that assists a small volume of water in flushing<br><br>Varieties: Few | Application largely for multi-unit installations, e.g. vacuum sewer system. Above floor, rear discharge. Drain pipe may be horizontal or inclined<br><br>Requires vacuum source, usually from vacuum sewer<br><br>Increased noise level | Periodic maintenance of vacuum source<br><br>0.002 kWh per use | 0.3 | 14 | 31 |

[a]Compared to conventional toilet usage (4.3 gal/flush, 3.5 uses/cap/d, and a total daily flow of 45 gpcd).

**Table 4-12. Wastewater Flow Reduction—Nonwater-Carriage Toilets**

| Generic Type[a] | Description | Application Considerations | O&M |
|---|---|---|---|
| Pit privy | Hand-dug hole in the ground covered with a seat in an enclosed structure<br><br>May be sealed vault rather than dug hole | Requires same site conditions as for wastewater disposal<br><br>Handles only toilet wastes<br><br>Outdoor installation<br><br>Odor potential | When full, cover with 2 ft of soil and construct new pit, unless sealed vault |
| Biological privy | Similar to pit privy except organic matter is added after each use. When pit is full it is allowed to compost for a period of about 12 months prior to removal | Can be constructed independent of site conditions if sealed vault<br><br>Handles only toilet waste and garbage<br><br>May be constructed by user<br><br>Outdoor installation<br><br>Residuals disposal | Addition of organic matter after each use<br><br>Removal and disposal/reuse of residuals may represent health hazard |
| Biological toilets | Large units with a separated decomposition chamber. Accept toilet wastes and other organic matter, and over a long time period partially stabilize excreta through biological activity and evaporation<br><br>Varieties: Several | Installation requires 6 to 12 in. diameter roof vent, space beneath floor for decomposition chamber, ventilation system, and heating<br><br>Handles toilet waste and some kitchen waste<br><br>Restricted usage capacity cannot be exceeded<br><br>Difficult to retrofit and expensive<br><br>Only units that meet Scandinavian and NSF testing standards should be used | Periodic addition of organic matter<br><br>Removal of product material at 6 to 24 month intervals should be performed by management authority<br><br>Power use: 0.3–1.2 kWh/d<br><br>Heat loss through vent |

**Table 4-12. Wastewater Flow Reduction—Nonwater-Carriage Toilets (continued)**

| Generic Type[a] | Description | Application Considerations | O&M |
|---|---|---|---|
| Incinerator | Small self-contained units that volatilize the organic components of human waste and evaporate the liquids<br><br>Varieties: Several | Installation requires 4-in. diameter roof vent<br><br>Handles only toilet waste<br><br>Power or fuel required<br><br>Increased noise level<br><br>Residuals disposal<br><br>Limited usage rate(frequency)<br><br>Only units that meet NSF testing standards should be used | Weekly removal of ash<br><br>Semiannual cleaning and adjustment of burning assembly and/or heating elements<br><br>Power: 1.2 kWh or 0.3 lb LP gas/use |

[a]None of these devices uses any water; therefore, the amount of flow reduction is equal to the amount of conventional toilet use: 16.2 gpcd or 36 percent of normal daily flow (45 gpcd). Significant quantities of pollutants (including N, $BOD_5$, SS, P, and pathogens) are therefore removed from wastewater stream.

**Table 4-13. Wastewater Flow Reduction—Showering Devices and Systems**

| Generic Type[a] | Description | Application Considerations | Water Use Rate (gpm) |
|---|---|---|---|
| Shower flow-control inserts and restrictors | Reduce flow rate by reducing the diameter of supply line ahead of shower head<br><br>Varieties: Many | Compatible with most existing showerheads<br><br>Installed by user | 1.5–3.0 |
| Reduced-flow showerheads | Fixtures similar to conventional, except restrict flow rate<br><br>Varieties: Many manufacturers, but units similar | Compatible with most conventional plumbing<br><br>Installed by user | 1.5–3.0 |
| On/off showerhead valve | Small valve device placed in the supply line ahead of showerhead allows shower flow to be turned on/off without readjustment of volume or temperature | Compatible with most conventional plumbing and fixtures<br><br>Usually installed by plumber | Unchanged, but duration (and waste) are reduced |
| Mixing valves | Specifically designed valves maintain constant temperature of total flow. Faucets may be operated (on/off) without temperature adjustment | Usually installed by plumber<br><br>Compatible with most conventional plumbing and fixtures | Unchanged, daily duration and use reduced |
| Air-assisted, low-flow shower system | Specifically designed system uses compressed air to atomize water flow and provide shower sensation | May be impossible to retrofit<br><br>Shower location <50 ft away from water heater<br><br>Requires compressed air source<br><br>Power source required<br><br>Maintenance of air compressor<br><br>Power: 0.01 kWh/use | 0.5 |

[a]No reduction in pollutant mass; slight increase in pollutant concentration.

Table 4-14. Wastewater Flow Reduction—Miscellaneous Devices and Systems

| Generic Type[a] | Description | Application Considerations |
|---|---|---|
| Faucet inserts | Device that inserts into faucet valve or supply line and restricts flow rate with a fixed or pressure-compensating orifice<br><br>Varieties: Many | Compatible with most plumbing<br><br>Installation simple |
| Faucet aerators | Devices attached to faucet outlet that entrain air into water flow<br><br>Varieties: Many | Compatible with most plumbing<br><br>Installation simple<br><br>Periodic cleaning of aerator screens |
| Reduced-flow faucet fixtures | Similar to conventional unit, but restrict flow rate with a fixed or pressure-compensating orifice<br><br>Varieties: Many | Compatible with most plumbing<br><br>Installation identical to conventional |
| Mixing valves | Specifically designed valve units that allow flow and temperature to be set with a single control<br><br>Varieties: Many | Compatible with most plumbing<br><br>Installation identical to conventional |
| Hot-water system insulation | Hot-water heater and piping is wrapped with insulation to reduce heat loss<br><br>Varieties: Many | May be difficult to wrap entire hot-water piping system |

[a]No reduction in pollutant mass; insignificant increase in pollutant concentration.

#### *4.5.2.3 Clotheswashing Devices and Systems*

The operation of conventional clotheswashers consumes varying quantities of water depending on the manufacturer and model of the washer and the cycle selected. For most, water usage is 87 to 201 L (23 to 53 gal)/use. On the basis of home water-use monitoring, an average water-use/wastewater flow volume of approximately 140 L (37 gal)/use has been identified, with the clotheswasher contributing about 38 Lpcd (10 gpcd) or 22 percent of the total daily water-use/wastewater flow (Table 4-2). Practical methods to reduce these quantities are somewhat limited. Eliminating wasteful water-use habits, such as washing with only a partial load, is one method. Front-loading automatic washers can reduce water used for a comparable load of clothes by up to 40 percent. In addition, wastewater flow reductions may be accomplished through use of a clotheswasher with either adjustable cycle settings for various load sizes or a wash-water recycle feature.

The wash-water recycle feature is included as an optional cycle setting on several commercially made washers. Selection of the recycle feature when washing provides for storage of the wash water from the wash cycle in a nearby laundry sink or a reservoir in the bottom of the machine for subsequent use as the wash water for the next load. The rinse cycles remain unchanged. Since the wash cycle accounts for about 45 percent of the total water use per operation, if the wash water is recycled once, about 64 L (17 gal) will be saved, if twice, about 129 L (34 gal), and so forth. Actual water savings and wastewater flow reductions are highly dependent on the user's cycle selection.

#### *4.5.2.4 Miscellaneous Devices and Systems*

There are a number of additional devices, fixtures, and appliances available to help reduce wastewater flow volumes. These are directed primarily toward reducing the water flow rate through sink and basin faucets. Table 4-14 presents a summary of several of these additional flow reduction devices. Experience with these devices indicates that wastewater volume can be reduced by 4 to 8 Lpcd (1 to 2 gpcd) when used for all sink and basin faucets.

### 4.5.3 Wastewater Recycle and Reuse Systems

Wastewater recycle and reuse systems collect and process the entire wastewater flow or the fractions produced by certain activities with storage for subsequent reuse. The performance requirements of any wastewater recycle system are established by the intended reuse activities. To simplify the performance requirements, most recycle systems process only the wastewaters discharged from bathing, laundry, and bathroom sink usage, and restrict the use of the recycled water to flushing water-carriage toilets and possibly to lawn irrigation. At the other extreme, systems are under development that process the entire wastewater flow and recycle it as a potable water source.

The flow sheets proposed for residential recycle systems are numerous and varied, and typically employ various combinations of the unit processes described elsewhere (Bennett and Linstedt, 1975; Witt, 1974; Hypes, et al.

**Table 4-15. Wastewater Flow Reduction—Wastewater Recycle and Reuse Systems**

| Flow-Sheet Description | Application Considerations | O&M | Total Flow Reduction (gpcd) | Comments |
|---|---|---|---|---|
| Recycle bath and laundry for toilet flushing | Requires separate toilet supply and drain line<br><br>May be difficult to retrofit to multistory building<br><br>Requires separate wastewater disposal system for toilet and kitchen sink wastes | Periodic replenishment of chemicals, cleaning of filters and storage tanks<br><br>Residuals disposal<br><br>Power use | 16 | Foaming is a common problem<br>Toilet water has a turbid apperance |
| Recycle portion of total wastewater stream for toilet flushing | Requires separate toilet supply line<br><br>May be difficult to retrofit to multistory building<br><br>Requires disposal system for unused recycle water | Cleaning/replacement of filters and other treatment and storage components<br><br>Residuals disposal<br><br>Periodic replenishment of chemicals | 16 | Foaming is a common problem<br>Toilet water has a turbid apperance<br>Colorant is usually used to disguise flush-water characteristics |
| Recycle toilet wastewaters for flushing water-carriage toilets | Requires separate toilet plumbing network<br><br>Utilizes low-flush toilets<br><br>Requires system for nontoilet wastewaters<br><br>May be difficult to retrofit<br><br>Application restricted to high use on multiunit installations | Cleaning/replacement of filters and other treatment components<br><br>Residuals disposal<br><br>Power use | 16 | Significant removal of pollutants<br>Limited to high-volume, commercial installations<br>Large capital investment required |

1974; Small Scale Waste Management Project, 1978). In Table 4-15, generic units are characterized according to their general recycle flow sheet.

## 4.6 Pollutant Mass Reduction

A second strategy for wastewater modification is directed toward decreasing the mass of potential pollutants at the source. This may involve the complete elimination of the pollutant mass contributed by a given activity or the isolation of the pollutant mass in a concentrated wastewater stream. Several methods for pollutant mass reduction are available. A few practical ones are described below.

### 4.6.1 Improved User Habits

Unnecessary quantities of many pollutants enter the wastewater stream when materials that could be readily disposed of in a solid waste form are added to the wastewater stream. A few examples include flushing disposable diapers or sanitary napkins down the toilet, or using hot water and detergents to remove quantities of solidified grease and food debris from pots and pans to enable their discharge down the sink drain.

### 4.6.2 Household Product Selection

The use of certain cleansing agents can contribute significant quantities of pollutants. In particular, cleaning activities, such as clotheswashing and dishwashing, can account for over 70 percent of the phosphorus in residential wastewater (Table 4-4). Modern detergents have reduced this percentage significantly due to lower phosphorus content, but very low P content products can reduce the total P content of wastewater to mg/L. Further reductions in nutrients, metals, and toxic organic chemicals are possible through product substitution. Additional details are available elsewhere (Atkins and Hawley, 1978; Hathaway, 1980).

### 4.6.3 Elimination of the Garbage-Disposal Appliance

The use of a garbage disposal contributes substantial quantities of $BOD_5$ and SS to the wastewater load (Table 4-4). As a result, it has been shown that the use of a garbage disposal may increase the rate of sludge and scum accumulation and produce a higher failure rate for conventional disposal systems under otherwise comparable conditions (Kreissl, 1985). For these reasons, and be-

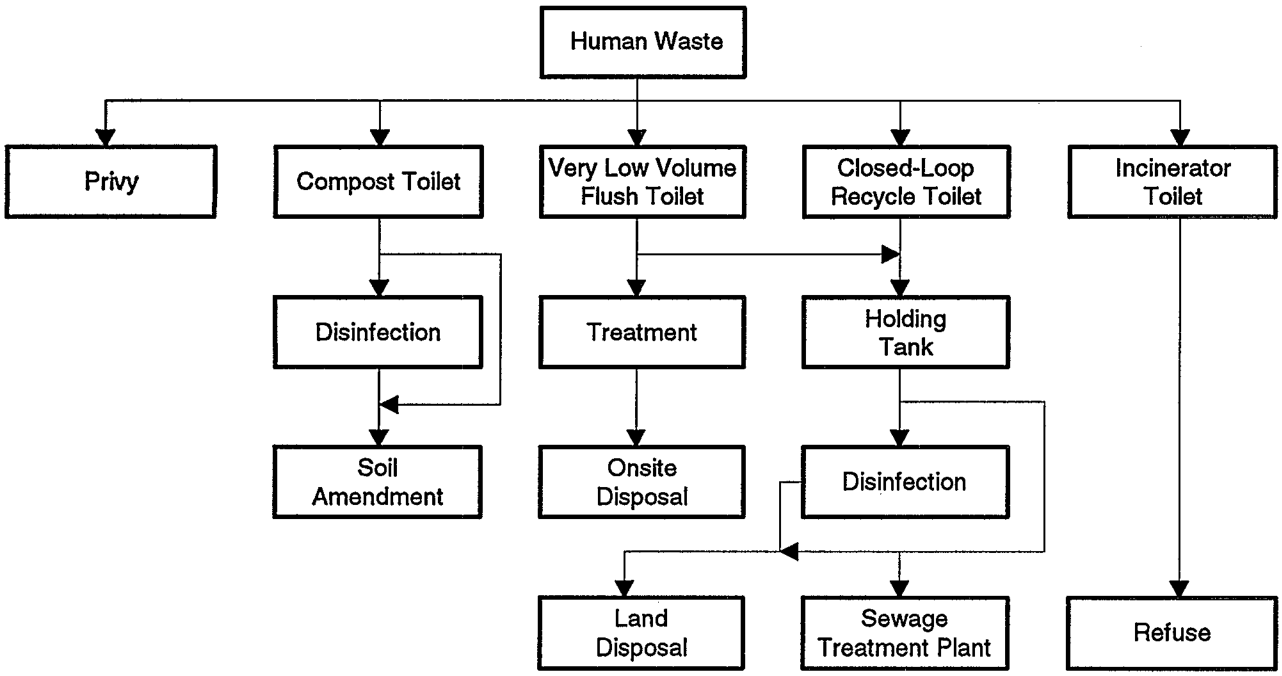


**Figure 4-4. Selected Strategies for Management of Segregated Human Wastes.**

cause most waste handled by a garbage disposal could be handled as solid wastes, the elimination of this appliance is advisable.

### 4.6.4 Segregated Plumbing Systems

Several toilet systems provide segregation and separate handling of human excreta (often referred to as *blackwater*) and, in some cases, garbage wastes. Removal of human excreta from the wastewater stream serves to eliminate significant quantities of pollutants, particularly SS, N, and pathogenic organisms (Table 4-4).

A number of potential strategies for management of segregated human excreta are presented in Figure 4-4. A discussion of the toilet systems themselves is presented in the wastewater flow reduction section of this chapter.

Wastewaters generated by fixtures other than toilets are often referred to collectively as *graywater*. Characterization studies have demonstrated that typical graywater contains appreciable quantities of organic matter, SS, P, and grease in a daily flow volume of 110 Lpcd (29 gpcd) (Siegrist et al., 1976; Hypes et al., 1974) (see Table 4-4). Its temperature as it leaves the residence is about 31°C (88°F), with a pH slightly on the alkaline side. The organic materials in graywater appear to degrade at a rate not significantly different from those in combined residential wastewater (Hypes et al., 1974). Microbiological studies have demonstrated that significant concentrations of indicator organisms as total and fecal coliforms are typically found in graywater (see References generally). One should assume, therefore, that graywater harbors pathogens.

Design allowances should be made only for the reductions in flow volume, as compared to typical residential wastewater. Although improved performance has been demonstrated for graywater-dosed soil absorption systems (SAS) when compared to combined wastewater SAS, (Malland, 1984) the variability of local codes dictates that SAS size reduction should not exceed the hydraulic reduction. For example:

SAS Area = $Q_{Graywater}/Q_{Total\ Wastewater}$.

## 4.7 Onsite Containment—Holding Tanks

Wastewaters may be contained onsite using holding tanks, and then transported offsite for subsequent treatment and disposal. However, this method is expensive and often aesthetically undesirable and is considered a "last resort" alternative. In many respects, the design, installation, and operation of a holding tank is similar to that for a septic tank. Several additional considerations do exist, however, as indicated in Table 4-16. A discussion regarding the disposal of pumpage from holding tanks is presented in Chapter 5.

## 4.8 Reliability

An important aspect of wastewater modification concerns the reliability of a given method to yield a projected modification at a specific dwelling or establishment over the long term. Reliability is of particular importance when designing an onsite wastewater disposal system based on modified wastewater characteristics.

**Table 4-16. Additional Considerations in the Design, Installation, and Operation of Holding Tanks**

| Item | Consideration |
|---|---|
| Sizing | Liquid holding capacity >7 days wastewater flow generation. Minimum capacity = 1,000 gal |
| Discharge | There should be no discharge/exfiltration |
| Alarm system | High-water alarm positioned to allow at least 3 days of use after activation |
| Accessibility | Frequent pumping is likely; therefore holding tank(s) must be readily accessible to pumping vehicle and employ proper fittings to minimize exposure |
| Flotation | Large tanks may be subject to severe flotation forces in high-groundwater areas when pumped |
| Cost | Frequent pumping and disposal of residuals result in very high operating costs |

Assessing the reliability of wastewater modification methods is a complex task that includes considerations of a technological, sociological, economic, and institutional nature. Major factors affecting reliability include:

- Actual wastewater characteristics prior to modification compared to the average
- User awareness and influence on method performance
- Installation
- Method performance
- User circumvention or removal

In most situations, projections of the impact of a wastewater modification method must be made, assuming the wastewater characteristics prior to modification are reasonably typical. If the actual wastewater characteristics deviate significantly from that of the average, a projected modification may be inaccurate.

The prospective user should be fully aware of the characteristics of a method considered for use prior to its application. Users who do not become aware of the characteristics of a method until after it has been put into use are more likely to be dissatisfied and attempt to circumvent or otherwise alter the method and negate the wastewater modification expected.

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-flush toilet is a passive device, while a flow-reducing shower head is an active one. Toilets that reduce flushing volume yet clean the bowl equally well are considered passive since additional flushing would not be required. Reduced shower flows can result in longer showers, thus negating potential savings, and are considered active devices.

Installation of any devices or systems should be made by qualified personnel. In many situations, a post-installation inspection is recommended to ensure proper functioning of the device or system.

Method performance is extremely important in assessing the reliability of the projected modification. Accurate performance data are necessary to estimate the magnitude of the reduction and to predict the likelihood that the method will receive long-term user acceptance. Accurate performance data can only be obtained through field tests and evaluations. Since many methods and system components are presently in various stages of development, only preliminary or projected operation and performance data may be available. These preliminary or projected data should be considered cautiously.

The continued employment of a wastewater modification method can be encouraged through several management actions. First, the user(s) should be made fully aware of the potential consequences should they discontinue employing the modification method (e.g., system failure, water pollution, rejuvenation costs). Also, the appropriate management authority can approve only those methods for which characteristics and merits indicate a potential for long-term user acceptance. Further, installation of a device or system can be made in such a manner as to discourage disconnection or replacement. Finally, periodic inspections by a local inspector within the framework of a sanitary district or the like may serve to identify plumbing alterations; corrective orders could then be issued.

In summary, to help ensure that a projected modification will actually be realized at a given site, efforts can be expended to accomplish the following tasks:

- Confirm that the actual wastewater characteristics prior to modification are typical.
- Make the prospective user(s) of the modification method fully aware of the characteristics of the method, including its operation, maintenance, and costs.
- Determine if the projected performance of a given method has been confirmed through actual field evaluations by organizations other than those with a proprietary interest.
- Ensure that any device or system is installed properly by competent personnel.
- Prevent user removal or circumvention of devices, systems, or methods.

## 4.9 Impacts on Soil-Based Treatment and Disposal Practices

### 4.9.1 Modified Wastewater Characteristics

Reducing the household wastewater flow volume without reducing the mass of pollutants contributed will increase the concentration of pollutants in the wastewater stream. The increase in concentrations will likely be insignificant for most flow reduction devices with the exception of those producing flow reductions of 20 percent or more. The effects of such changes on onsite treatment components are not considered sufficient to justify design modifications for single-family homes. The increase in pollutant concentrations in any case may be estimated utilizing Figure 4-5.

### 4.9.2 Wastewater Treatment and Disposal Practices

Table 4-17 presents a brief summary of several potential impacts that wastewater modification may have on soil-based disposal practices. It must be emphasized that the benefits derived from wastewater modification are potentially significant. Wastewater modification methods, particularly wastewater flow reduction, should be considered an integral part of any onsite wastewater disposal system.

*Note*: Assumes pollutant contributions are the same under the reduced flow volume.

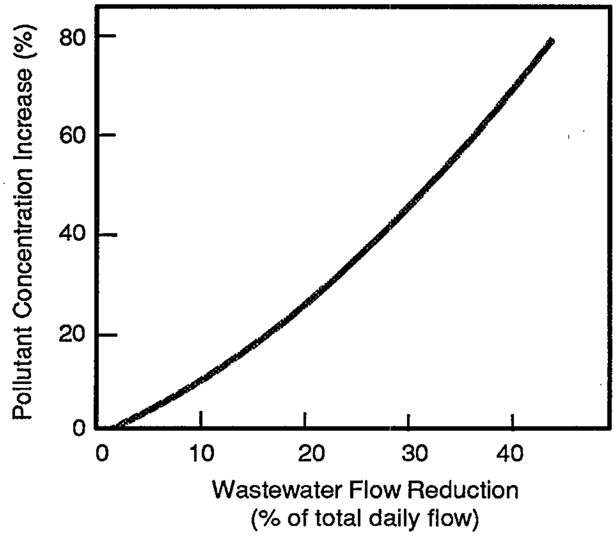


**Figure 4-5. Flow Reduction Effects on Pollutant Concentrations.**

**Table 4-17. Potential Impacts of Some Wastewater Modification on Disposal Practices**

| Disposal System Type | Potential Impact | Modification Practice: Flow Reduction | Modification Practice: Pollutant Reduction |
|---|---|---|---|
| Subsurface | May extend service life of functioning system | X | X |
| | Reduce contamination of ground water and surface water | | X |
| | Reduce frequency of septic tank pumping | X | |
| | Reduce sizing of infiltrative area | X | |
| | Partially relieve hydraulically overloaded system | X | |
| Surface discharge | Reduce O&M costs | X | X |
| | Reduce sizing and initial cost of components | X | X |
| | Eliminate need for certain components, (e.g., N removal) | | X |
| | Remedy hydraulically overloaded system | X | |
| Evapotranspiration | Remedy hydraulically overloaded system | X | |
| | Reduce sizing of ET area | X | |
| Containment (holding tank) | Reduce frequency of pumping | X | |
| | Reduce sizing of containment structure | X | |

## 4.10 References

When an NTIS number is cited in a reference, that reference is available from:

National Technical Information Service
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Springfield, VA 22161
(703) 487-4650


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# CHAPTER 5
## Technology Options

The summary of technologies in this chapter is provided to further aid the process of evaluation and selection of technologies by identifying both general factors and those specific to particular locations. Technologies discussed include soil-based approaches, mechanical systems, and collection systems. In an effort to be concise, not all the evaluation factors that are relevant to a technology or other considerations worthy of discussion have been included.

To facilitate the evaluation and selection process, the discussion of each technology is presented as criteria that should be considered to meet a certain need or application.

The criteria are:

- **Technology Description**—the function and operation of the technology, with some discussion of typical expected removal rates and modifications.
- **Applicability and Status**—the extent of use of the technology, along with typical flow ranges and compatibility with other options
- **Advantages/Disadvantages**—a relative comparison of the technology in regard to a variety of factors, including power consumption/energy cost, level of operator skill required, potential for odor nuisance, quality of effluent achievable, and sludge production
- **Design Criteria**—presentation of typical ranges of process loading rates, land requirements, and performance
- **Capital Cost Sensitivity**—consideration of the most obvious significant costs related to the particular technology as well as other costs that should be assessed such as pretreatment or transmission to the treatment site
- **O&M Requirements**—system complexity and operator skill requirements; reliability of mechanical equipment and spare parts needed to maintain operation in case of equipment failure
- **Monitoring**—frequency of sampling and level of analysis relative to permit requirements; process control monitoring relative to complexity and sensitivity of the system
- **Construction Issues**—desired level of operation and other considerations related to this phase of the project
- **Residuals**—characterization of residuals generated from the process, as well as volume relative to other processes; options for removal and disposal of sludges
- **Special Considerations**—factors not covered by other criteria such as special chemical requirements

## 5.1 Constructed Wetlands

### 5.1.1 Technology Description

Wetlands are lands where the water surface is near the ground surface for enough of the year to maintain saturated soil conditions and promote related vegetation. Constructed wetlands are similar systems specifically designed for wastewater treatment.

Constructed wetlands can be considered part of a wastewater treatment system, while most natural wetlands are considered receiving waters and are therefore subject to applicable laws and regulations regarding discharge. The influent to currently operating constructed wetland systems ranges from septic tanks to secondary effluents. One constructed wetland is included as a component in a system designed to treat septage.

The two different types of constructed wetlands are characterized by the flow path of the water through the system. The first, called a free-water surface (FWS) wetland, includes appropriate emergent aquatic vegetation in a relatively shallow bed or channel. The surface of the water in such a system is exposed to the atmosphere as it flows through the area. The second type, called a subsurface flow (SF) wetland, includes a foot or more of permeable media—rock, gravel, or course sand—that supports the root system of the emergent vegetation. The water in the bed or channel in such a system flows below the surface of the media.

Both types of constructed wetlands typically include a barrier to prevent ground-water contamination beneath the bed or channel; barrier materials range from compacted clay to membrane liners. A variety of wastewater application methods have been used with constructed wetlands, and a number of different outlet structures and methods have been included to control the depth of water in the system.

### 5.1.2 Applicability and Status

As of 1991 a total of 143 communities were known to be using or considering the feasibility of using a constructed wetland system for wastewater management (Brown and Reed, 1991). Systems in use include 31 FWS and 26 operating SF wetlands. The range in size is from 2.2 to 880 L/s (0.05 to 20 mgd) for FWS systems and from 0.04 to 132 L /s (0.001 to 3 mgd) for SF systems. (Because the information gathered to date is incomplete, the data that follow may not contain information for all of the wetlands.)

Wastewater treatment using constructed wetlands has come into wider use only since the late 1980s, although a few FWS systems were in operation earlier. Although currently there are slightly more FWS than SF systems in operation, SF systems are projected to outnumber FWS systems over time.

The majority of constructed wetland systems (70 percent of FWS and 90 percent of SF) treat less than 44 L/s (1 mgd). Smaller systems tend to be SF configurations (mean = 13 L/s (0.3 mgd), median = 2 L/s (0.04 mgd)). Larger systems tend to be FWS configurations that range in size from 2.3 to 880 L/s (0.05-20 mgd) (mean = 85 L/s (1.9 mgd), median = 20 L/s (0.5 mgd)).

### 5.1.3 Advantages/Disadvantages

Constructed wetlands offer the following advantages for wastewater management:

- Low construction cost
- Passive system easily managed by small community with O&M personnel
- Excellent removal of biochemical oxygen demand (esp. $BOD_5$) and suspended solids (SS) from primary or septic/Imhoff tank effluents
- Generally attractive systems with secondary ecological benefits in terms of wildlife habitat enhancement

The following disadvantages should be taken in account when considering a constructed wetland system:

- Lack of generally agreed-upon design factors, resulting in several unproven approaches to design of land-intensive systems, especially FWS types where up to 4 ha/L/s (450 ac/mgd) have been required
- SF systems remain unproven for other than $BOD_5$ and SS removal

### 5.1.4 Design Criteria

No generally accepted consensus has been established in the United States regarding design of constructed wetland systems. There are several schools of thought concerning design approaches, ranging from unsubstantiated empirical to semirational models based on very limited data. Also, no consensus has been reached on system configuration and other system details such as aspect ratio, depth of water or media, type of media, slope of surfaces and bottoms of beds, and inlet and outlet structures. Moreover, pretreatment varies widely, with facultative lagoons the most common form of pretreatment. These are used at 41 percent of the FWS and 44 percent of the SF systems. Septic tanks are used at 24 percent of the SF systems. Aerated lagoons, secondary treatment, and advanced treatment have also been used.

About one-third of the systems in operation use a mixture of plant species; the other two-thirds use bulrush, cattail, or reeds alone. One-third of the FWS systems use only cattails, and 40 percent of the SF systems use only bulrush. Canna lilies, arrowhead, duckweed, reed canary grass, and torpedo grass have also been used in constructed wetland systems.

The broad ranges in the following parameters for SF systems illustrate the lack of consensus regarding system design: hydraulic loading rates varying from 0.02 to 0.4 $m^3/m^2 \times d$ surface area (0.5 to 10 gpd/sq ft) and 1 to 110 $m^3/m^2 \times d$ vertical facial area perpendicular to flow (25 to 2,700 gpd/sq ft); organic loading rates of 2 to 160 kg BOD/ha x d (0.2 to 140 lb/ac x d); gravel sizes of 6 to 130 mm (0.25 to 5 in.); and depths of 30 to 76 cm (1.0 to 2.5 ft). For comparison, the data below presents design parameters for state-of-the-art systems used in Europe, where over 500 constructed wetland sites have been established.

Areal requirement = 5 $m^2$/PE

1 PE = 56 gpd (200 L/d)

Performance:

$BOD_{rem}$: 80–90 percent

total $N_{rem}$: <30 percent

total $P_{rem}$: <15 percent

Plants: Common reed (Phragmites australis)

Pretreatment: Primary clarification or septic/Imhoff tanks

Bed slopes: top = 0 percent
bottom = 1 percent toward outlet

Bed depth: average = 0.6 m (2 ft)
at inlet ≥ 0.3m (1 ft)

Containing walls: slope = as vertical as possible

Freeboard: ≥0.5m

Liner: yes

Media: 3–10 mm (rounded)

Inlet: maximize horizontal distribution

Outlet: variable water level control (20 cm (8 in.) above rock surface to bed bottom)

Width: determined by following Darcy's equation:

$A_c = Q_s / k_f (dh/ds)$

where:

$A_c$ = facial area perpendicular to flow ($m^2$)

$Q_s$ = design flow ($m^3/s$)

$k_f$ = hydraulic conductivity (with gravel sizes recommended $\approx 3 \times 10^{-3}$ m/s)

dh/ds = slope of bed bottom (m/m)

Design parameters are similarly broad for FWS systems: hydraulic loadings are 10 to 40 L/$m^2$ surface area (0.2 to 1.0 gpd/sq ft) for BOD and SS removal (as high as 120 L/$m^2$ x d (3 gpd/sq ft) for nitrification) and organic loadings are 10 to 20 kg BOD/ha x d (9 to 18 lb/ac x d) for BOD and SS removal (as low as 1 kg/ha x d (0.9 lb/ac/d) for nitrification). Hydraulic residence time of FWS systems should be at least 7 days, but as many as 365 days have been used.

### 5.1.5 Capital Costs

Capital costs per unit of flow for both types of constructed wetlands in the United States are shown in Figure 5-1. The most sensitive costs are for land, earthwork, liners, vegetation planting, and, for SF systems, fill media.

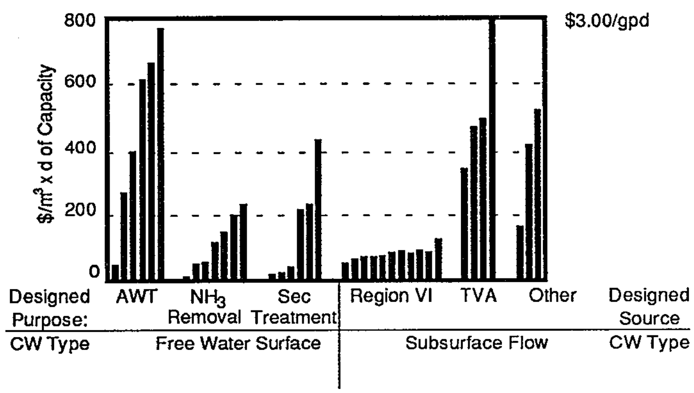



**Figure 5-1. Capital Costs for Wetland Systems.**

### 5.1.6 O&M Requirements/Costs

Constructed wetland systems should be inspected at least weekly. This O&M requirement includes inlet and outlet inspection and flow recording. Overall experience has not yet allowed quantification of these costs, but they are considered negligible in comparison to treatment alternatives. Sidewall maintenance is also required.

### 5.1.7 Monitoring

With constructed wetlands, vegetation growth should be monitored and standard sampling and analysis requirements of the permit should be fulfilled.

### 5.1.8 Construction Issues

Considerations concerning the construction of wetlands for wastewater treatment include:

- Media for SF systems should be rounded and washed prior to placement
- Vertical sidewalls for SF systems ensure proper vegetation at edge of bed
- Use of gabions and large slopes at the inlet simplifies construction and ensures good horizontal distribution of flow in SF systems

### 5.1.9 Residuals

Although not generally practiced, harvesting of an SF system's vegtetation would enhance removal of nutrients at the cost of increased O&M and residuals disposal problems. It is also possible that eventual dislodging may be required, but quantification of this measure is not yet available. SF systems should not be immediately preceded by facultative lagoons since algae will aggravate any clogging problems.

### 5.1.10 Special Considerations

The role of plants in a constructed wetland system is at present undefined. Moreover, the hybrid designs of constructed wetlands will evolve such that their present capabilities will be upgraded. At this time, however, the SF systems and small FWS systems should not be expected to remove more than BOD and SS in significant quantities; however, very large (very lightly loaded) systems may also accomplish nitrification.

## 5.2 Rapid Infiltration

### 5.2.1 Technology Description

Rapid infiltration is a soil-based wastewater treatment method that typically consists of a series of earthen basins with exposed soil surfaces designed for a repetitive cycle of flooding, infiltration/percolation, and drying (Figure 5-2). The method depends on a relatively high rate of wastewater infiltration into the soil and percolation through a vadose or unsaturated soil zone below the infiltrative surface before recharge to the ground-water table. System design is based on the capability of soils to provide acceptable treatment before the percolate reaches the ground water.

### 5.2.2 Applicability and Status

Rapid infiltration is a proven technology for year-round treatment of domestic, municipal, and other organic wastewaters. Its application is limited primarily by soil characteristics, ground-water impacts, and land costs. It is not well suited for inorganic industrial wastewaters.

### 5.2.3 Advantages

Rapid infiltration is a method of land application providing very favorable removal of the conventional wastewater parameters—including ammonia—that is simple to operate and requires only a minimum of operator intervention. Moreover, it requires less land area than other land application methods and may be operated year round. It is a

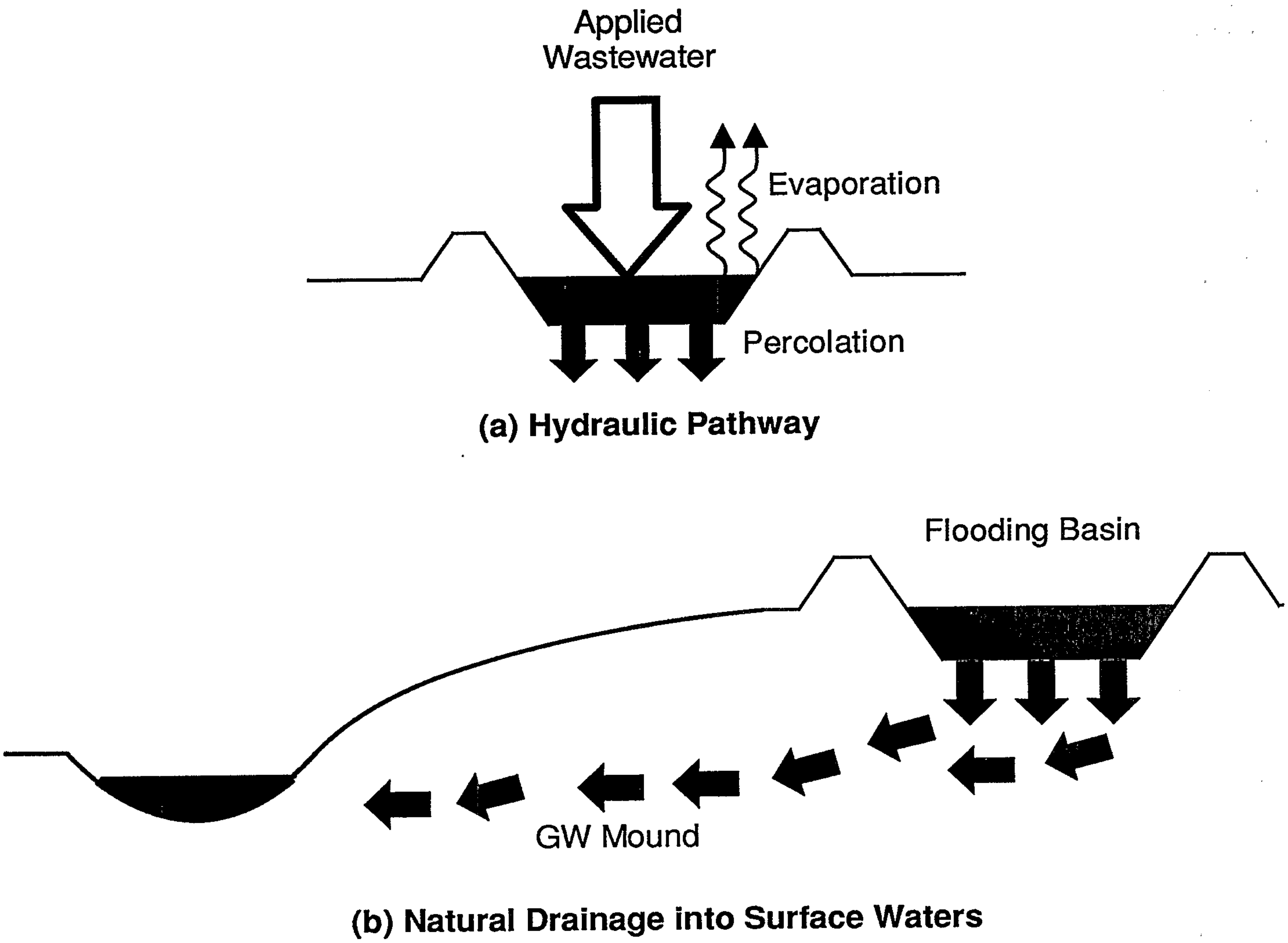



**Figure 5-2. Schematic of a Rapid Infiltration Facility (EPA, 1981).**

"zero-discharge" method that provides ground-water recharge rather than requiring an outfall for direct discharge to surface water. Frequently, renovated wastewater is recovered via wells for reuse in irrigation.

### 5.2.4 Disadvantages

Use of rapid infiltration is limited by site and soil characteristics that affect the capability of the soil to accept and treat the applied wastewater. Potential ground-water impacts from nitrate nitrogen are a serious concern and may also limit its application.

### 5.2.5 System Design

#### *5.2.5.1 Site Selection*

Site selection is based on the treatment capacity of the soils and the availability of sufficient land area of suitable topography. The treatment capacity of the soil is dependent primarily on texture, structure, and unsaturated thickness. No soil type provides optimum conditions for the removal of all wastewater constituents. Fine-texture soil, such as silt and clay loams, has a relatively low hydraulic conductivity and is therefore unsuitable for rapid infiltration. Coarse soil, such as sand, has a higher rate of hydraulic conductivity and reaeration to allow higher hydraulic and organic loading and shorter cycles for basin resting, which can permit rapid infiltration. However, coarse soil can be a less-effective physical filter, have a lower cation exchange capacity, and allow more rapid percolation of wastewater through the vadose zone where most treatment occurs. Typically, sites with sand to sandy loam soils are selected. A summary of typical treatment performance data is presented in Table 5-1.

Unsaturated depth of soil is another critical site-selection criterion. A minimum of 1.5 to 2.5 m (5 to 8 ft) of unsaturated soil with relatively uniform hydraulic conductivity is necessary below the infiltrative surface to provide necessary treatment. Greater unsaturated thicknesses are usually required in coarse, granular soils to provide longer residence times. Ground-water mound height analysis must be performed to determine if the separation distance can be maintained during system operation. If the desired separation distance cannot be maintained, the hydraulic loading can be reduced, basin geometry can be changed to have a high-aspect ratio in relation to the ground-water gradient, basin spacing can be increased, or ground-water drainage can be provided.

#### *5.2.5.2 Hydraulic Loading Rate*

The design hydraulic loading rate is determined by soil characteristics, ground-water mounding potential, treatment requirements, applied wastewater quality, and precipitation. The maximum hydraulic loading is established by the hydraulic conductivity of the least permeable soil horizon in the vadose zone. Typically, 4 percent of the measured saturated hydraulic conductivity is used as a preliminary estimate. This rate may be adjusted downward to limit the BOD loading to within 21 to 126 kg/ha/d (19 to 112 lb/ac/d) to control infiltrative surface clogging. Primary treatment is the minimum level of pretreatment suggested, but where the public has even limited access, secondary treatment is required. If there is concern about nitrogen contributions to the ground water below the system, nitrogen rather than BOD may control the hydraulic loading (pattern and total rate) and pretreatment. Potential for excessive ground-water mounding and other site-specific issues may require the hydraulic loading to be reduced further. Typical average hydraulic loading rates are 1.5 to 35.0 cm/d. To assist in selecting appropriate hydraulic loading rates, loadings used by systems operating under similar conditions should be compared (EPA, 1981, 1984; Reed & Crites, 1984; Overcash & Pal, 1979; WPCF, 1990).

#### *5.2.5.3 Application Patterns*

The selected application pattern for basin operation will determine the number and size of the basins and the land area required. The pattern is selected to control infiltrative surface clogging and/or to promote nitrogen removal (Lance et al., 1976; Lance, 1984). Typical ratios of application periods to drying periods vary from 0.2, where maximum hydraulic loading is the objective, to 1.0 for maximum nitrogen removal. Suggested loading cycles are presented in Table 5-2.

### 5.2.6 Design Modifications

Underdrains or extraction wells may be used to lower the water table to increase the unsaturated depth of soil below the infiltrative surface or to reclaim the renovated water. Information on the design of underdrain systems can be found elsewhere (USDA, 1971; U.S. Dept. of Interior, 1973; Luthin, 1973). Some states require an aquifer

**Table 5-2. Suggested Hydraulic Loading Cycles for Rapid Infiltration Systems[a]**

| Loading Cycle Objective | Applied Wastewater | Application Season | Application Period[b] (days) | Drying Period (days) |
|---|---|---|---|---|
| Maximize infiltration rates | Primary | Summer | 1–2 | 5–7 |
| | | Winter | 1–2 | 7–12 |
| | Secondary | Summer | 1–3 | 4–5 |
| | | Winter | 1–3 | 5–10 |
| Maximize nitrogen removal | Primary | Summer | 1–2 | 10–14 |
| | | Winter | 1–2 | 12–16 |
| | Secondary | Summer | 7–9 | 10–15 |
| | | Winter | 9–12 | 12–16 |
| Maximize nitrification | Primary | Summer | 1–2 | 5–7 |
| | | | 1–2 | 7–12 |
| | Secondary | Summer | 1–3 | 4–5 |
| | | | 1–3 | 5–10 |

[a]EPA, 1981.
[b]Application periods for primary effluent should be limited to 1–2 days to prevent excessive clogging. Periods may be affected by the rate of ground-water mounding (rise).

**Table 5-1. Typical Rapid Infiltration System Performance[a]**

| Parameter | Loading (kg/ha x d) | Removal | Comments |
|---|---|---|---|
| $BOD_5$ | 45–158 | 86–98% | Lower removals associated with high loading rates on coarse soils |
| Nitrogen | 3–37 | 10–80% | Removal depends on preapplication treatment, BOD:N, climate, hydraulic loading, and wet/dry cycle |
| Phosphorus | 1–12 | 29–99% | Removal correlates with soil texture, time of service, soil mineralogy, and travel distance |
| Toxic organics | NA | Varies with structure | Favorable removal of volatile and biodegradable organics appears to occur where the subsoil remains aerobic |
| Fecal coliforms | NA | 2-6 logs | Removals correlate with soil texture, travel distance, and resting time |
| Virus | NA | 2-4 logs | Limited data suggest favorable removals with low loadings, fine-texture soil, aerobic subsoil, and high temperatures |

[a]WPCF, 1990; Nilsson, 1991.
NA = not applicable

protection permit, which calls for demonstration of non-degradation and, in some cases, upgrading of pretreatment and monitoring requirements.

Continuously flooded infiltration basins are a modified operation method that has been used where high-rate infiltration is desired. Treatment performance of such systems is unfavorable because of the rapid percolation and the elimination of subsoil reaeration and aerobic conditions.

### 5.2.7 Capital Cost Sensitivity

Capital costs of rapid infiltration projects are controlled primarily by land costs and earthwork. Earthwork includes infiltration surface preparation and dike construction. If the topography is not favorable for basin construction, site grading may add a significant cost. Other associated costs that may be significant are installation of underdrains, if needed, transmission of wastewater to the treatment site, and pretreatment.

### 5.2.8 O&M Requirements

The primary O&M requirements of rapid infiltration systems include basin cycling, infiltration surface maintenance, winter operations, nitrogen management, and monitoring. Other O&M tasks include maintenance of pumps and other equipment and basin dikes. These tasks are easy to perform and require little specialized training. No special equipment is necessary except for a tractor and harrow for infiltration surface maintenance. A useful description of these tasks is provided by EPA (1984).

Costs of O&M are associated primarily with labor costs. Other costs include power costs for wastewater pumping and equipment depreciation. Total labor requirements, excluding maintenance of the pretreatment works, should not exceed 10 to 15 hr/wk. O&M costs are in the range of 5 to 10 cents/1,000 gal of wastewater treated.

### 5.2.9 Construction Issues

Construction activities include infiltration, surface preparation, dike construction, and installation of basin piping, inlet structures, and, where needed, basin underdrains. These construction activities are as important as site evaluation in achieving successful performance of a rapid infiltration system. Infiltration surface preparation is the most critical factor. The surface must be carefully leveled to within 5 cm (2 in.) of the specified finished elevation in each basin with a minimum of surface compaction. Once the surface is fine-graded, it should be ripped and cross-ripped to a depth of 60 cm (2 ft). Construction practices should minimize equipment travel in the direction of ground-water movement. The dikes between the basins must prevent leakage from the basins and provide vehicular access to each basin, and they must be graded such that erosion of the embankments does not occur. Proper construction procedures are discussed elsewhere (EPA, 1984; WPCF, 1990).

### 5.2.10 Monitoring

Monitoring requirements cover wastewater volumes applied to each basin, daily wastewater ponding levels in each basin, basin cycle times, applied wastewater quality, and ground-water elevations and quality.

### 5.2.11 Residuals

No residuals are usually produced with rapid infiltration systems. Occasional surface scrapings and residual algae, for instance, may be buried onsite given the minimal volume of such residuals.

## 5.3 Stabilization Ponds

### 5.3.1 Technology Description

Facultative lagoons, or ponds, are the most frequently used form of municipal wastewater treatment in the United States, with over 5,000 systems currently in operation. These lagoons are usually 1.2 to 1.8 m (4 to 6 ft) in operating depth and are not mechanically mixed or aerated. The layer of water near the surface is aerobic due to atmospheric reaeration and algal respiration. The layer at the bottom of the lagoon is anaerobic and includes sludge deposits. The intermediate layer, termed the *facultative zone*, ranges from aerobic near the top to anaerobic at the bottom. The layers may be indistinct or be clearly defined due to temperature-related water-density variations. Disruptions can occur in the spring and fall of each year when the surface layer of melted ice may have a higher density than lower layers. This higher density water induces vertical movement, mixing the pond contents and producing objectionable odors due to the release of anaerobically formed gases during these unstable periods.

The presence of algae in the near-surface water is essential to several aspects of treatment performance of facultative ponds. In the presence of sunlight the algal cells take up carbon dioxide from the water and release oxygen. On warm, sunny days the oxygen concentration in the near-surface water can exceed saturation levels. In addition, because of the temporary carbon dioxide removal, the pH of the near-surface water can exceed 10. This results in conditions favorable for ammonia removal via volatilization, and long-detention-time facultative ponds have been shown to be very effective for ammonia removal. This photosynthetic activity by the algae occurs on a diurnal basis so that both oxygen and pH levels shift from a maximum during daylight hours to their minimums at night.

The oxygen produced by the algae and by surface reaeration is used by aerobic bacteria to stabilize the organic material in the upper layer of water. Anaerobic fermentation is the dominant activity in the bottom layer in the lagoon. Both of these reaction rates are significantly reduced during the winter and early spring months in cold climates. Effluent quality may be reduced to the equivalent of long-detention-time settling when an ice cover persists on the water surface. For this reason, many states in the northern United States and in Canada prohibit discharge from facultative lagoons during the winter months.

Typical detention times range from 20 to 180 days for continuously discharging facultative lagoons. Detention times can approach 200 days in northern climates where discharge restrictions prevail. Effluent BOD ≤30/mg/L can usually be achieved; effluent TSS may range from ≤30 mg/L to over 100 mg/L depending on the algal concentrations. Many existing facultative lagoons are large, single-cell systems, with the inlet constructed near the center of the cell. That configuration can result in undermining the design volume of the system. A multiple-cell system, with at least three cells, is recommended with appropriate inlet and outlet structures to maximize the utilization of the design volume.

Although the concept is somewhat land-intensive, particularly in northern climates, stabilization ponds offer a reliable, easy-to-operate process that makes it attractive for small, rural communities where sufficient land is often available.

#### *5.3.1.1 Aerated Lagoons*

Aerated lagoons are smaller and deeper than facultative lagoons and are designed for biological treatment of wastewater on a continuous basis. In contrast to stabilization ponds, which obtain oxygen from photosynthesis and surface reaeration, aerated lagoons typically employ devices that supply supplemental oxygen to the system. Aerated lagoons evolved from stabilization ponds when aeration devices were added to counteract odors arising from anaerobic conditions. The aeration devices may be mechanical (i.e., surface aerator) or diffused air systems using submerged or overhead pipes. Surface aerators are divided into two types: caged aerators and the more common turbine and vertical shaft aerators. Diffused air systems utilized in lagoons typically consist of plastic pipes supported near the bottom of the lagoon cells with regularly spaced sparger holes drilled in the tops of the pipes, although overhead air headers and finer bubble systems have recenty been applied. Because aerated lagoons are normally designed to achieve partial mixing only, aerobic/anaerobic stratification may occur, and large fractions of incoming solids and the biological solids produced from waste conversion can settle to the bottom of the lagoon cells.

#### *5.3.1.2 Controlled Discharge*

A common operational modification is the controlled-discharge mode where pond discharge is prohibited during the winter months in cold climates and/or during the peak algal growth periods in all climates. In this approach, each cell in the system is isolated and then discharged sequentially. A common operational modification to aerated and facultative lagoons is the controlled-discharge pond system. Sufficient storage capacity is provided in the lagoon system to allow wastewater storage during winter months, peak algal growth periods, or receiving-stream low-flow periods. In this approach, each cell in the system is isolated and then discharged sequentially. In the Great Lakes states of Michigan, Minnesota, and Wisconsin, as well as in the province of Ontario, many pond systems are designed to discharge in the spring and/or fall when water quality effects are minimized. As a secondary benefit, operational costs are lower than for a continuous discharge lagoon because of reduction in laboratory monitoring requirements and the need for less operator control. A similar modification is called *a hydrograph controlled release lagoon* (HCRL) where water is retained in the pond until flow volume and conditions in the receiving stream are adequate for discharge, thus eliminating the need for costly additional treatment.

#### *5.3.1.3 Modifications*

A recently developed physical modification to upgrade the performance of existing one- or two-cell systems involves dividing existing lagoons into multiple cells to improve hydraulic conditions. Another recent patented development uses a floating plastic grid to prevent the transport of duckweed (*Lemna sp.*) plants on the surface of the final cell(s) in the lagoon system. This duckweed cover restricts the penetration of light to destroy algae and improve effluent SS quality. With sufficient detention time (≥30 days) and intensive harvesting, significant nutrient removal may be possible. Some systems also have the capability to draw effluent from several levels and thereby avoid the high algal concentrations near the water surface during discharge.

The rock filter, used as an alternative means of removing algae from lagoon effluents, consists of a submerged bed of rocks (5 to 20 cm (2 to 8 in.) diameter) through which the lagoon effluent is passed vertically or horizontally, allowing the algae to be removed in the rock filter.

The basic simplicity of O&M is the key advantage of this process. The effluent quality achievable and the dependability of long-term operation, however, have not been well documented yet. Rock filtration following treatment in single-cell lagoons has been problematic and therefore is not recommended.

#### *5.3.1.4 Sand Filter*

The intermittent sand filter is an outdoor, gravity-actuated, slow rate filtration system that capitalizes on the availability of land area. It is a biological and physical wastewater treatment mechanism consisting of an underdrained bed of granular material, usually sand. The filter surface is flooded intermittently with lagoon effluent at intervals that permit the surface to drain between applications. It is recommended that the flow be directed to one filter for 24 hours. That filter is then allowed to drain and dry for one to two days, and the flow goes to an adjacent filter. It is preferable to have three filter beds where good operation and treatment can be accomplished over a three-day cycle. (This technology is described in more detail in Section 5.3.4.6.) The system can remove suspended solids (algae) and BOD as well as convert ammonia to nitrate-nitrogen.

### 5.3.2 Applicability and Status

Lagoon (wastewater stabilization pond) technology, the most common means of wastewater treatment in the United States, is primarily used in smaller communities. The concept is well-suited for rural communities and industries where land is readily available but skilled maintenance is not. Lagoons that discharge to larger receiving streams have been designated as "equivalent to secondary treatment" by the EPA. However, many existing lagoons will require upgrading by methods such as those described above if they cannot meet stream effluent requirements or if they are leaking into the ground water.

### 5.3.3 Advantages/Disadvantages

The advantages of stabilization ponds include the following:

- Minimum operational skills required
- Low capital cost requirements
- Many means of upgrading available, minimizing capital outlays where lagoons already exist
- Sludge disposal required only at 10- to 20-year intervals

Disadvantages of this technology include:

- Large land area requirements
- Continous flow (traditional) facultative ponds cannot meet stringent effluent standards during warm seasons without upgrading
- Lagoons can negatively impact ground water if an inadequate liner is installed or if an existing liner is damaged
- Most lagoons discharge to smaller, water-quality-limited streams and may, therefore, require upgrading
- Several upgrading techniques are not fully characterized, making their adoption speculative

### 5.3.4 Design Criteria

#### *5.3.4.1 Facultative Lagoons*

Every system should have a series of at least three cells. Most states have design criteria that specify the areal organic loading (lb/ac/d) and hydraulic residence time. Typical organic loading for the system is 22 to 67 kg BOD/ha/d (20 to 60 lb/ac/d). Overall typical detention times range from 20 to 180 days or more, depending on location.

A number of empirical models exist for design of facultative lagoons. These include first-order plug flow, first-order complete mix, and models proposed by Gloyna, Marais, Oswald, and Thirumurthi. None of these have been shown to be clearly superior to the others. All will provide a reasonable design if the proper parameters are selected and if the hydraulic characteristics of the system are known.

Depth is normally 1.5 to 1.8 m (5 to 6 ft), although a range of 0.9 to 2.4 m (3 to 8 ft) is often suggested. Typical removals of $BOD_5$ are 75 to 95 percent, but SS may vary from negligible to 90 percent. A high degree of toxic organics and metals are generally removed. Ammonia conversion (20 to 80 percent) and phosphorus removal (10 to 50 percent) can be significant in warm, algae-controlled periods. Likewise, fecal coliform reductions are dependent on the detention time and are quite significant during these periods.

#### *5.3.4.2 Aerated Lagoons*

Every system should have a series of at least three cells that are lined (or similarly secured) to prevent adverse ground-water impacts. Many states have design criteria that specify the design loading, the hydraulic residence time, and the aeration requirements. Pond depths range from 1.8 to 6 m (6 to 20 ft), with 3 m (10 ft) being typical (the shallow depth systems are usually converted facultative lagoons). Detention times range from 3 to 20 days, with 10 days being typical (shorter detention times use higher intensity aeration). The design of these systems for BOD removal is based on first-order, complete-mix kinetics. Even though the system is not completely mixed, a conservative design will result. The model commonly used is:

$$\frac{C_e}{C_o} = \frac{1}{[1 + (K_T)(t/n)]^n}$$

Where:

$C_e$ = effluent BOD

$C_o$ = influent BOD

$K_T$ = temperature-dependent rate constant 12.5 $t^{-1}$ at 20°C)

t = total detention time in system

n = number of equal-size cells in system

Oxygen requirements are typically 30 to 40 hp/Mgal capacity; they may range as high as 100 hp/Mgal where complete mixing is desired.

BOD removal can range from 80 to 95 percent. Effluent total suspended solids (TSS) can range from 20 to 60 mg/L depending on the concentration of algae in the effluent. Removal of ammonia nitrogen is usually less effective than in facultative lagoons. Nitrification of ammonia can occur in underloaded aerated lagoons or if the system is specifically designed for that purpose. Phosphorus removal is also less effective than in facultative lagoons due to the more stable pH and alkalinity conditions. Phosphorus removals of about 10 to 20 percent might be expected. Removal of coliforms and fecal coliforms can be effective, depending on detention time and temperature. Disinfection will be necessary to meet effluent limits consistently.

#### *5.3.4.3 Rock Filters*

Key considerations for rock filters include:

- Hydraulic loading rates (HLR): 0.3 to 0.9 $m^3/m^3/d$ (7.5 to 22 gpd/sq ft)
- Submerged inlets
- 7.5 to 15 cm (3 to 6 in.) rock
- SS removal (percent) = 100 – 66H (H=HLR in $m^3/m^3/d$)
- At least three cells are required in the system that precede the rock filter

#### *5.3.4.4 Hydrograph Controlled Release Lagoon (HCRL) Systems*

An HCRL system has three principal components: a stream-flow monitoring system, a storage cell, and an effluent discharge system. The stream-flow monitoring system measures the flow rate in the stream and transmits this data to the effluent discharge system. The effluent discharge system consists of a controller and a discharge structure. The controller operates a discharge device (e.g., a motor-driven sluice gate) through which wastewater is discharged from the storage lagoon; however, these tasks can be manually performed.

Key considerations for HCRLs include:

- Need release model keyed to flow (as measured by depth) in receiving stream
- Outlets should be capable of drawing from different depths to ensure best quality
- Storage cells sized by use of water-balance equations

#### *5.3.4.5 Duckweed Systems*

The key considerations for duckweed systems include:

- A 30-day retention time with shallow depth and frequent harvesting required (≈ every 1 to 3 days at peak season) if nutrient removal sought
- Plan must be in place on fate of harvested plants (i.e., processing on site, transportation, and ultimate fate)
- Manageability by local authorities is yet to be established (need contract management)
- May need post-aeration to meet effluent dissolved oxygen (DO) requirements

#### *5.3.4.6 Intermittent Sand Filters*

The preferred design (EPA, 1980) for lagoon upgrading is as follows:

- 3 cells in series, all 76 to 91 cm (30 to 36 in.) deep
- Progressively finer sands (e.g., effective sizes of 0.72 mm, 0.40 mm, and 0.17 mm)
- Effluent quality ≈ 10 mg/L of $BOD_5$ and SS
- Complete nitrification expected, except under extremely cold conditions

This configuration is considered to provide the most effective approach (in terms of achieving the most favorable effluent quality attainable) for lagoon upgrading.

### 5.3.5 Capital Costs

#### *5.3.5.1 Facultative Lagoons*

*a. Assumptions*

1. Design basis:
   Areal loading method, 45 kg BOD/ha x d (40 lb/ac x d) in warm climates, 22 kg BOD/ha x d (20 lb/ac x d) in the primary cells only in cold climates
   Water depth 1.5 m (5 ft)
   Effluent BOD = 30 mg/L
   Effluent TSS = 60 mg/L
2. Costs based on November 1990 prices (ENR = 4780)
3. Construction costs include excavation, grading, berm construction, and inlet and outlet structures. Costs do not include a lagoon liner, purchase of the land, or delivery of wastewater to the site.
4. The construction cost equations below are valid for a preliminary estimate for design flows up to 440 L/s (10 mgd), where
   C = Costs (in millions of dollars)
   Q = Wastewater flow (mgd)

*b. Construction Costs*

Warm climates: $C = 0.750(Q)^{0.729}$
Cold climates: $C = 1.800(Q)^{0.711}$

#### *5.3.5.2 Aerated Lagoons*

*a. Assumptions*

1. Design basis:
   Influent BOD = 210 mg/L
   Effluent BOD = 30 mg/L
   Depth = 3 m (10 ft)
   Detention time = 10 days
   Floating mechanical aerators at 10 hp/Mgal
2. Costs based on November 1990 prices (ENR = 4780).
3. Construction costs include excavation, grading, berm construction, and inlet and outlet structures, and membrane liner. Costs do not include purchase of the land or delivery of wastewater to the site.
4. The construction cost equations below are valid for a preliminary estimate for design flows up to 440 L/s (10 mgd), where
   C = Costs (millions of dollars)
   Q = Wastewater flow (Mgal/a)

*a. Construction Costs*

0–1 mgd: $C = 0.310\ (Q)^{0.301}$

1–10 mgd: $C = 0.310\ (Q)^{0.412}$

#### *5.3.5.3 Rock Filters and Intermittent Sand Filters*

Information on costs is currently insufficient to provide firm estimates, but construction costs of $1.00/gpd of capacity should be considered quite conservative.

#### *5.3.5.4 Duckweed and HCRL*

Cost information is currently insufficient to serve as a basis for providing estimates.

### 5.3.6 O&M Costs

#### *5.3.6.1 Facultative Lagoons (All Climates)*

$C = 0.016\ (Q)^{0.496}$

#### *5.3.6.2 Aerated Lagoons*

0–1 mgd: $C = 0.025\ (Q)^{0.513}$

1–10 mgd: $C = 0.025(Q)^{0.732}$

#### *5.3.6.3 Upgrading Systems*

A range of 50 to 75 cents per year per gallons per day of capacity is a reasonable estimate of O&M costs for intermittent sand filters.

Controlled discharge, HCRL, and duckweed and rock filters: No valid data.

## 5.4 Overland Flow

### 5.4.1 Technology Description

Overland flow is a land application method of wastewater treatment with a point discharge to a surface water. The technology consists of a series of uniformly sloped, vegetated terraces with a wastewater distribution system located at the top of the terrace and a runoff collection channel at the bottom (Figure 5-3). Facilities for wastewater storage during wet or freezing weather are also provided. In overland flow, wastewater is applied intermittently across the top of the terraces and allowed to sheet flow over the vegetated surface to the runoff collection channel. The system is not designed for soil percolation, though some percolation may occur.

Treatment is achieved primarily through sedimentation, filtration, and biochemical activity as the wastewater flows through the vegetation on the terraced slope. SS settle or are filtered from the flow to be degraded or incorporated into the soil. Effluent TSS concentrations of less than 10 mg/L are possible. However, algae removal is not consistent because many algal cells are buoyant or motile and resist removal by sedimentation or filtration. Organics are degraded by biological films attached on the plant and soil surfaces. $BOD_5$ removals of 90 percent are often achieved. Total nitrogen removals of up to 80 percent can be achieved, but will vary with the mass loading, ambient temperature, and other environmental factors and are more commonly much lower. The principal removal mechanism is biological denitrification, but some plant uptake and ammonium volatilization may occur. Phosphorus may be partially removed by soil adsorption (in the early operational stage) and plant uptake. Removal efficiencies as high as 50 percent are possible, but 20 to 60 percent is common). Overland flow is not effective in pathogen removal, however. Little is known about treatment performance for metals and toxic organics removals. Because treatment is dependent on active biomass and vegetation, the terraces are operated on wet/dry cycles and applications are ceased during freezing periods. Further information on treatment performance is available elsewhere (WPCF, 1990).

### 5.4.2 Applicability and Status

Overland flow first gained widespread use in the early 1970s. It was developed for areas where soils are poorly suited for other land application methods dependent on ground-water recharge for ultimate disposal of the treated wastewater. Today, overland flow is successfully used by many small communities, primarily in the southern half of the United States. Its use is limited primarily by land suitability, climate, and land costs.

### 5.4.3 Advantages

Overland flow is well suited for wastewater treatment by rural communities and seasonal industries with organic wastes. It provides secondary or advanced secondary treatment, yet is relatively simple and inexpensive to operate. If the vegetative cover can be harvested and sold (e.g., as forage), overland flow can provide an economic return from the reuse of water and nutrients. Of the land application methods of wastewater treatment, overland flow is the approach least restricted by soil characteristics; however, this method does require a relatively impermeable soil for conventional operation.

### 5.4.4 Disadvantages

Overland flow is primarily limited by climate, crop water tolerances, and land slope. Application is restricted during wet weather and can be limited when temperatures remain below freezing. Application rates may be restricted by the type of crop grown. Steeply sloping or flat terrain is not well suited for this method of treatment. Disinfection is required in order to meet effluent permit requirements prior to discharge (unique to land application processes).

### 5.4.5 Design Criteria

#### *5.4.5.1 Site Selection*

Topography is the most important site-selection criterion because of its impact on the costs of earthwork. Terrace slopes should be between 2 and 8 percent and relatively uniform, with sufficient length to provide adequate travel time for treatment. Flatter slopes may result in wastewater ponding on the terraces, and erosion may be significant on steeper slopes. Terrace lengths are typically 30 to 60 m (100 to 200 ft). South-facing slopes are preferred

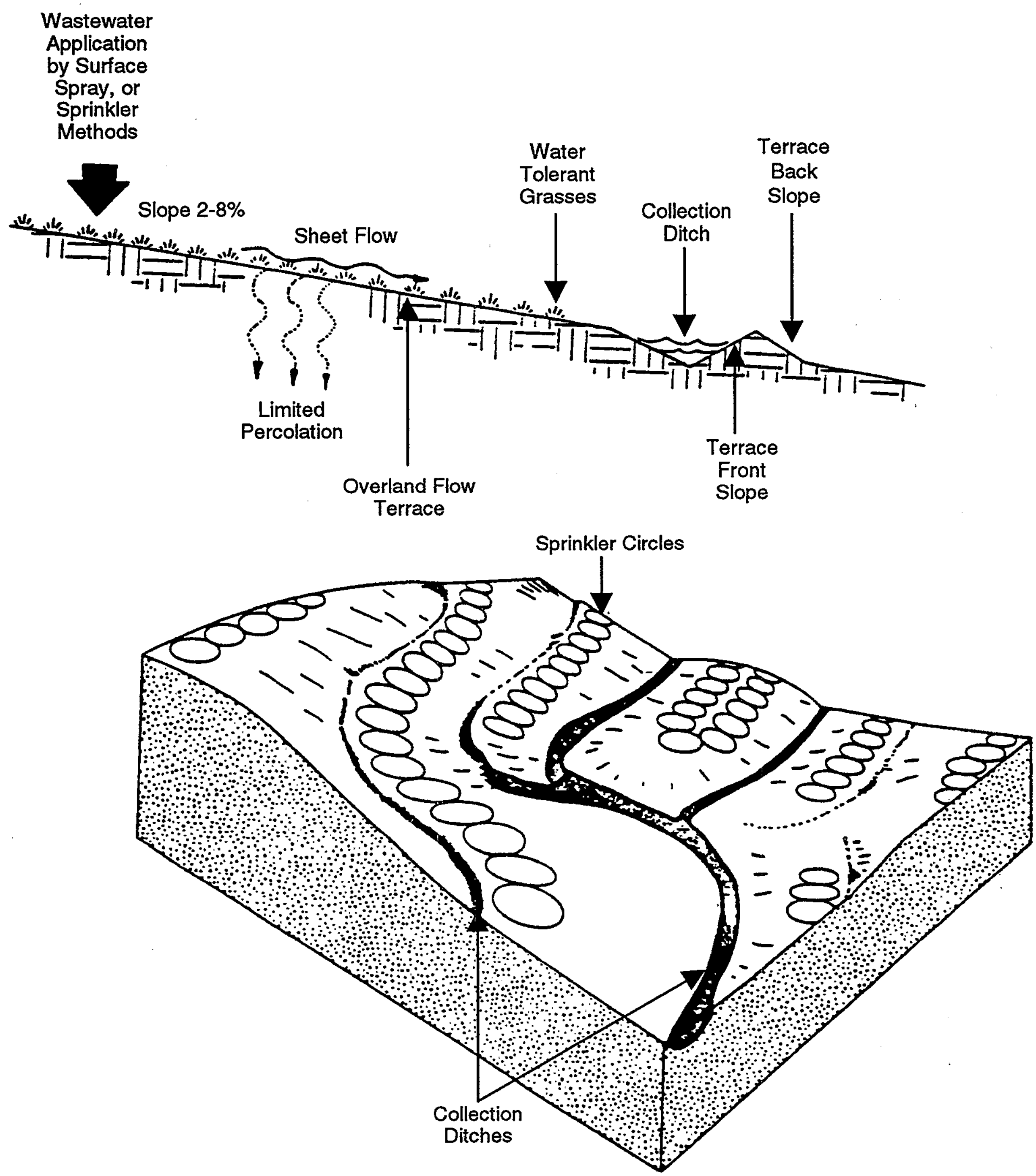



**Figure 5-3. Schematic of an Overland Flow System.**

in cold climates to extend the operating season (EPA, 1984; WPCF, 1990).

Soil type and permeability are of limited importance. Overland flow was developed for use on soils with low permeabilities. Therefore, detailed soil evaluation is only necessary if significant wastewater percolation into the soil is expected. Where the soil is permeable, it should meet the same criteria used for slow-rate land application if ground-water protection is to be provided.

#### *5.4.5.2 Preapplication Treatment*

Secondary treatment levels (30 mg/L $BOD_5$ and 30 mg/L TSS or greater) can easily be achieved by overland flow systems with primary pretreatment. Whether higher levels of pretreatment are required depends on local regulations and aesthetics. The most common method of pretreatment used in the United States is lagoons, which effectively remove grit and other solids that may damage or clog the pumps and distribution system. However, in lagoons with long retention times, such as facultative or aerated ponds, algae growth can be significant, adversely affecting effluent quality from overland flow systems. Therefore, primary treatment or lagoon stabilization with retention times limited to 1 to 2 days are suggested (WPCF, 1990). Where climatic restrictions obtain, storage facilities must also be provided that are sufficient to hold the flow during extented periods.

#### *5.4.5.3 Hydraulic Loading Rate*

The rate of wastewater application is dependent on the effluent discharge limitations, level of pretreatment, length and slope of the terrace, and climate. The degree of treatment provided is directly proportional to the length of the slope and the ambient temperature, and inversely proportional to the hydraulic loading rate and terrace slope. The degree of pretreatment primarily affects nitrogen removal. Low $BOD_5$ concentrations in the applied wastewater enhance nitrification, while higher $BOD_5$ concentrations enhance denitrification. Typical hydraulic loading rates for primary pretreated wastewater are 0.2 to 0.4 $m^3$/h x m of terrace width (16 to 32 gal/h x ft). For secondary pretreated wastewater, hydraulic loadings may reach 0.6 $m^3$/h x d (48 gal/h x t). The design rate selected for a particular site should take into account the influence of each of the factors above (EPA, 1981; 1984; WPCF, 1990).

#### *5.4.5.4 Operating Cycle*

Overland flow systems have performed best when applications of wastewater are alternated with drying periods. Common practice is to load a terrace for 12 hr followed by a 12-hr drying period. Lower ratios enhance nitrification.

#### *5.4.5.5 Distribution*

Uniform application of wastewater across the width of the terrace is critical to system performance. Wastewater is applied at the top of each terrace by sprinklers or weir overflow methods. Because weir overflow methods require precise leveling of the device used, they are suggested for small systems only. Sprinklers are the most commonly used method. They are located either at the top of the terrace or within the top one-third of the terrace. Where sprinklers are used, terrace lengths should be increased, with 45 m (150 ft) the minimum length. The slope length should be at least 20 m (65 ft) greater than the diameter of the sprinkler pattern. Distribution types and their design are described elsewhere (EPA, 1984; WPCF, 1990).

#### *5.4.5.6 Vegetation Selection*

Perennial, water-tolerant grasses are best suited for overland flow systems. Suitable cool-season grasses include reed canary grass, fescue, and rye grass. Suitable warm-season grasses include common and coastal Bermuda, Dallis, and Bahia. Local agricultural extension agents should be consulted for further guidance.

#### *5.4.5.7 Runoff Collection*

The runoff collection channel must be designed with sufficient capacity and grade to prevent water from ponding at the base of the terrace. At a minimum, it should be designed to handle the 10-year, 1-hour rainfall event for the area. The channel may be lined or unlined, but unlined ditches require more maintenance to control weed growth.

#### *5.4.5.8 Storage*

Sufficient storage is needed to hold wastewater during periods when applications cannot be made. In cold climates, the storage requirements are determined by the number of days the temperature is below freezing. In the United States, storage requirements can reach 160 days (EPA, 1984; WPCF, 1990). In climates where freezing is not a significant factor, sufficient storage should be provided to allow flexibility in operation and crop harvesting, typically 2 to 5 days.

### 5.4.6 Capital Cost Sensitivity

Land and earthwork costs are the most significant capital costs of overland flow systems. Distribution network construction may also be significant depending on the type used and the topography. Typical capital costs vary from $1.40 (0.1 mgd) to $14.00 (0.01 mgd) per gallon of daily capacity.

### 5.4.7 O&M Requirements

O&M requirements of overland flow systems include harvesting of the cover crop, maintenance of the distribution and collection systems, and pest control. Periodic mowing is necessary to maintain a healthy growth of grass. An annual minimum of 4 to 6 mowings per year is suggested. Removal of the grass is necessary for high-level nitrogen and phosphorus control. Mosquitoes and weed control through the use of pesticides or microbiological agents may be necessary.

Costs of O&M are associated primarily with labor costs. Almost no special equipment other than the appropriate agricultural equipment is required. Total labor requirements, excluding maintenance of the pretreatment works and effluent disinfection system, should not exceed 10 to 15 hr/wk. O&M costs typically range from $1.40 to $2.80/1,000 gal treated wastewater (at 10,000 gal/d) to $0.35 to $0.70/1,000 gal (at 100,000 gal/d).

### 5.4.8 Construction Issues

Terrace grading is critical to achieving sheet flow for successful operation. The slopes should be final graded with a land plane to within 1.5 cm (0.6 in.) of the prescribed slope. Soil compaction is not a critical factor except as it relates to establishment of the vegetative cover or to prevention of ground-water contamination. The terraces should be irrigated after seeding, but wastewater applications should be limited until the vegetation is well established; the acclimation period is typically 3 to 4 months in duration.

### 5.4.9 Monitoring

Monitoring requirements cover volumes, rates, frequencies, and patterns of wastewater applications and influent and effluent quality monitoring dictated by the permit. Where significant infiltration of wastewater into the soil occurs, ground-water monitoring may also be required.

### 5.4.10 Residuals

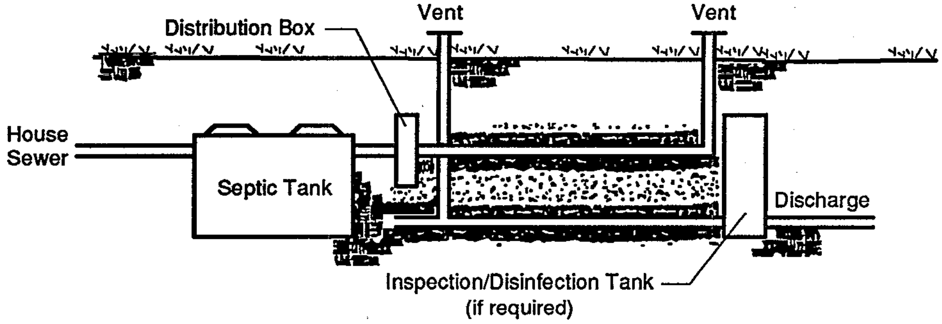
Residuals produced by overland flow systems are limited to harvested crops.

## 5.5 Slow Sand Filtration

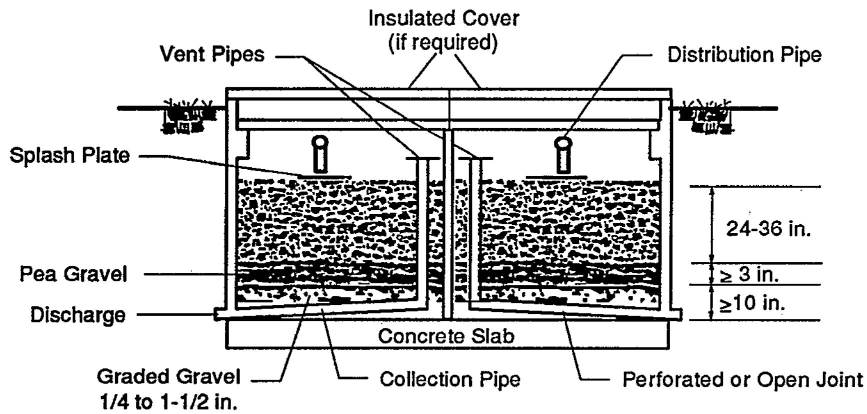
### 5.5.1 Technology Description

Slow sand filters are used for both small communities and individual homes. They consist of one or more beds of granular material, typically graded sand, 60 to 90 cm (2 to 3 ft) deep, underlain with collection drains imbedded in gravel. Pretreated wastewater is intermittently applied to the surface of the sand bed and allowed to percolate through the bed where it receives treatment. The filter media remains unsaturated and is vented to the atmosphere, such that an aerobic environment is maintained in the filter. The percolate is usually collected by the underdrains, which remove it from the filter for further treatment or disposal.

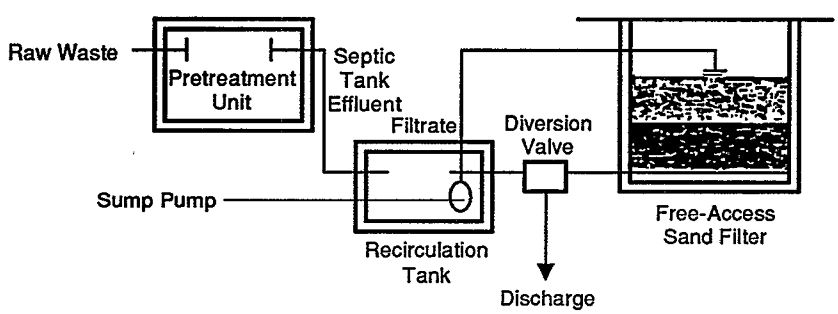
Three types of slow sand filters are commonly used. They include buried, open, and recirculating configurations (Figure 5-4). While all three are somewhat similar in design, they may differ in method of operation, performance, access, and filter media specifications.




(a) Buried (Single-Pass) Sand Filter




(b) Open (Intermittent) Sand Filter




(c) Basic Recirculating Sand Filter

**Figure 5-4. Schematics of Slow Sand Filters.**

#### *5.5.1.1 Buried Sand Filters*

Buried sand filters are single-pass filters constructed below grade in an excavation lined with an impermeable membrane and covered with backfill material. Distribution piping is imbedded in gravel or crushed rock placed on top of the filter media. A geotextile fabric over the top of the gravel prevents the backfill material from piping into the filter. To provide venting of the filter, the upstream ends of the underdrains are extended vertically above grade. The distribution piping may also be extended above grade for venting. These systems have been used primarily for individual home wastewater treatment.

#### *5.5.1.2 Open or Intermittent Sand Filters*

Open sand filters are single-pass (intermittent) filters exposed to the surface to allow inspection and periodic maintenance of the infiltrative surface. Wastewater is usually distributed over the surface by flooding. The filters are typically exposed, although removable covers may be used depending on climatic conditions.

#### *5.5.1.3 Recirculating Sand Filters*

Recirculating sand filters are open sand filters designed to recirculate the filtrate. The recirculation tank receives pretreated wastewater and a portion of the filtrate. The pump, which is engaged by a submersible pump operated by a timer, regularly doses the filter with the mixture of the pretreated wastewater and return filtrate. Modifications to recirculating filters that provide substantial nitrogen removal have been successfully applied (Lamb et al., 1990; Piluk and Hao, 1989; Sandy et al., 1988).

The mechanism of treatment is a combination of biochemical and physical filtration and chemical adsorption. Slow sand filters operate as aerobic, fixed-film biological reactors capable of producing a high-quality effluent consistently. The treatment process is very stable and reliable, but dependent on temperature. It is able to accept variations in hydraulic and organic loading with little effect on effluent quality.

Performance will vary with the media used, temperature, hydraulic and organic loadings, and method of operation (EPA, 1985; Oregon DEQ, 1982). Recirculating sand filters typically provide a high-quality effluent similar to single-pass (open) filters. Typically effluent $BOD_5$ and SS concentrations below 10 mg/L are achieved. The effluent is very low in turbidity. Nearly complete nitrification is also achieved by all filters except in cold-temperature conditions. Denitrification has been shown to occur in recirculating filters; varying with modifications in design and operation, removals of 50 percent or more of the applied nitrogen may be achieved. Phosphorus may be removed initially through chemical adsorption on the media grains, but removals decline with time as the adsorption capacity is reached. Fecal coliform removal accomplished with recirculating filters is less efficient than single-pass filters, but removal of two to three logs is commonly achieved.

### 5.5.2 Applicability and Status

Sand filters are a proven method for providing advanced secondary wastewater treatment. They have been used for treatment of municipal wastewater since the late 1800s. They are well suited to rural communities, small clusters of homes, individual residences, and business establishments. The life cycle costs are lower than for extended aeration package plants for small community treatment. Their use is limited by land availability and capital cost, but they are easy to operate, requiring personnel with a minimum of skills.

### 5.5.3 Advantages

Sand filters are moderately inexpensive to construct, have low energy requirements, and do not require highly skilled personnel to operate. They produce high-quality effluents, significantly better than those produced by extended-aeration package plants or stabilization lagoons. The treatment process is extremely stable, requiring limited intervention by operating personnel. Through modular design, treatment capacity can easily be expanded.

### 5.5.4 Disadvantages

Sand filters require somewhat more land area than package plants, but their land requirements are lower than for lagoons. The amount of head required by the filters typically exceeds 1 m (3.25 ft), possibly requiring pumping for effluent disposal where available land relief is insufficient. Odors from open, single-pass filters treating primary or septic/Imhoff tank effluent do occur and may require buffer zones from inhabited areas. Suitable filter media may not be available locally.

### 5.5.5 Design Criteria

#### *5.5.5.1 Preapplication Treatment*

The minimum of primary treatment of wastewater is required before application to sand filters. Either septic tanks or Imhoff tanks are commonly used. Higher levels of treatment can reduce filter size requirements or prolong filter life, but such cost savings must be weighed against the increased costs of pretreatment. For larger flows, stabilization ponds may be used (see Section 5.3).

#### *5.5.5.2 Media Characteristics and Depth*

Filter media are described by the effective size ($D_{10}$ or particle diameter with 10 percent of grains by weight or smaller) and uniformity coefficient ($D_{60}/D_{10}$) or some other measure of the grain size range. The smaller the effective size, the higher the level of treatment, the lower the hydraulic loading, and the more frequent the need for maintenence. The lower the uniformity coefficient, the longer the filter's life span. In practice, effective sizes are 0.10 to 1.5 mm, but the requirements differ for each type. Uniformity coefficients are typically less than 4.0.

Sand is the most commonly used media, but anthracite, garnet, mineral tailings, and bottom ash have been used. Alternative media must be durable and insoluble in water. The media used should have a total organic content of less than 1 percent, total acid soluble matter of less than 3 percent, a hardness greater than 3 on the Moh's scale, and be generally round in shape.

Media depths commonly used are 0.6 to 0.9 m (2 to 3 ft). Deeper beds tend to provide more complete treatment and more constant effluent quality. Greater depths are suggested where the media have smaller effective sizes, because the smaller media retains more moisture, which reduces aeration of the media necessary for effective biochemical activity.

#### *5.5.5.3 Loading Rates*

Hydraulic loading rates vary with the characteristics of the media, the type of filter design, and the wastewater strength. Rates are typically 3 to 40 cm/d (0.75 to 10 gpd/sq ft). Buried filters usually have the lowest hydraulic loadings, while recirculating filters have the highest (Table 5-3). Higher rates generally reduce the effluent quality and increase the frequency of maintenance for a given media and filter design. Organic loading rates have not been widely used in current designs.

**Table 5-3. Design Criteria—Slow Sand Filters[a]**

| Design Factor | Buried | Open | Recirculating |
|---|---|---|---|
| Pretreatment | ———Minimum of Sedimentation——— | | |
| Media specifications | | | |
| Effective size (mm) | 0.7–1.00 | 0.40–1.00 | 1.0–1.50 |
| Uniformity coefficient | <4.0 | <4.0 | <4.0 |
| Depth (m) | 0.60–0.90 | 0.60–0.90 | 0.60–0.90 |
| Hydraulic loading (cm/d) | 4–6 | 5–10 | 12–20 (forward flow) |
| Dosing frequency | 2–4/d | 1–4/d | 5–10 min/ 30 min |
| Recirculation ratio | NA | NA | 3:1–5:1 |

[a]EPA, 1985.
NA = not applicable

#### *5.5.5.4 Wastewater Application*

The manner in which wastewater is applied to the surface is critical to performance. Doses of wastewater must be applied uniformly to the filter surface at sufficient intervals for the wastewater to infiltrate completely. This allows ample aeration of the filter media to effect aerobic treatment. Distribution methods commonly used involve surface flooding, spray nozzles, and interlaced pipe laterals (gravity and pressure).

Dosing frequencies are related to the type of filter, media size, and wastewater strength. Smaller, more frequent doses are preferred for coarse media to increase residence time in the media. Typical dosing frequencies for intermittant filters range from 1 to 4 times daily. Recirculating filters are dosed for 5 to 10 minutes every half hour.

Multiple sand filters are suggested to provide standby capacity and allow filter loading and resting cycles. Typically, two or four cells are provided.

Dosing tanks are sized for the maximum size dose to be used. The dosing tank for recirculating filters must be sized for at least five times the forward-flow dose volume.

### 5.5.6 Capital Cost Sensitivity

Filter media availability will have the most significant impact on construction costs. Other associated costs that may be significant relate to land, earthwork, pretreatment, and transmission of wastewater to the treatment site.

### 5.5.7 O&M Requirements

Slow sand filters require relatively little operational control or maintenance. Primary O&M tasks include filter surface maintenance, dosing equipment servicing, and influent and effluent monitoring. With continued use, sand filter surfaces will become clogged with organic biomass and solids. Once operating infiltration rates fall below the hydraulic loading rate, permanent ponding of the filter surface will occur indicating that the filter should be taken off-line for "resting" and surface tilling or removal. Buried filters are designed to operate without maintenance for their design life. Filters exposed to sunlight may develop algae mats, which may require control by shading the surface. For community systems, disinfection will be required prior to discharge, but disinfectant quantity requirements are low due to their high quality.

### 5.5.8 Construction Issues

Filter media placement and underdrain construction are the most critical construction issues. The filter media is usually placed in lifts. Filter performance is best when the filter media is homogeneous. The surface of the filter must be graded carefully to ensure uniform treatment. The underdrains must also be sufficiently sloped to ensure adequate drainage.

### 5.5.9 Monitoring

Monitoring requirements cover wastewater volumes and quality applied to each filter, dosing frequencies, and surface infiltration rates. All discharge permit requirements must also be met.

### 5.5.10 Residuals

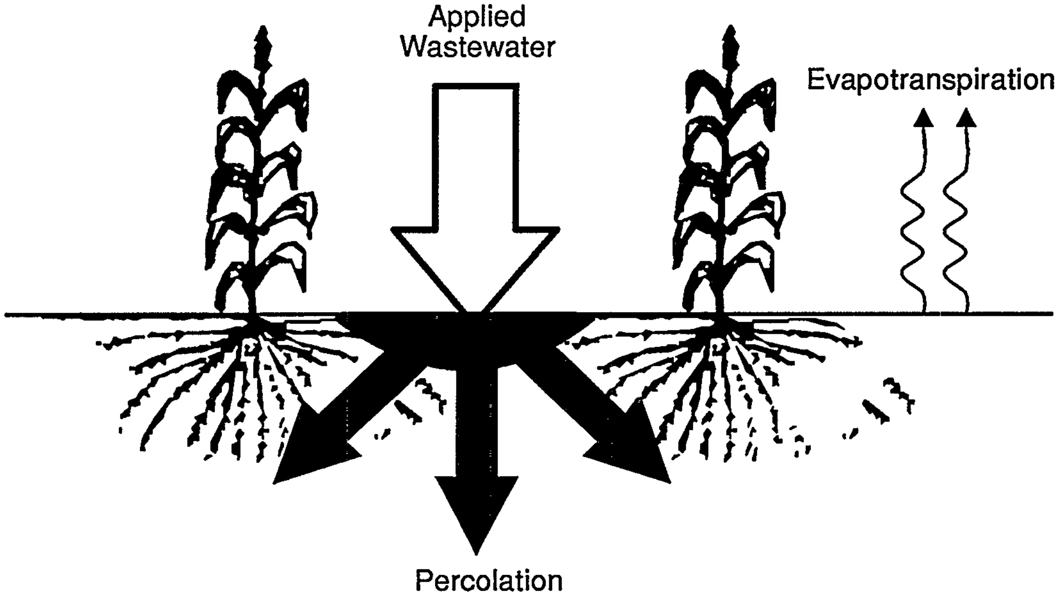
Small quantities of residuals are usually produced. In most intermittent sand filtration facilities, sand must be periodically removed from the surface as a method of filter rejuvenation. The sand may be stockpiled onsite and later washed and reused, or landfilled. Other filters generally do not generate solids in quantities sufficient to require utilization of a disposal method, except at the end of their service lives.

## 5.6 Slow Rate Land Application

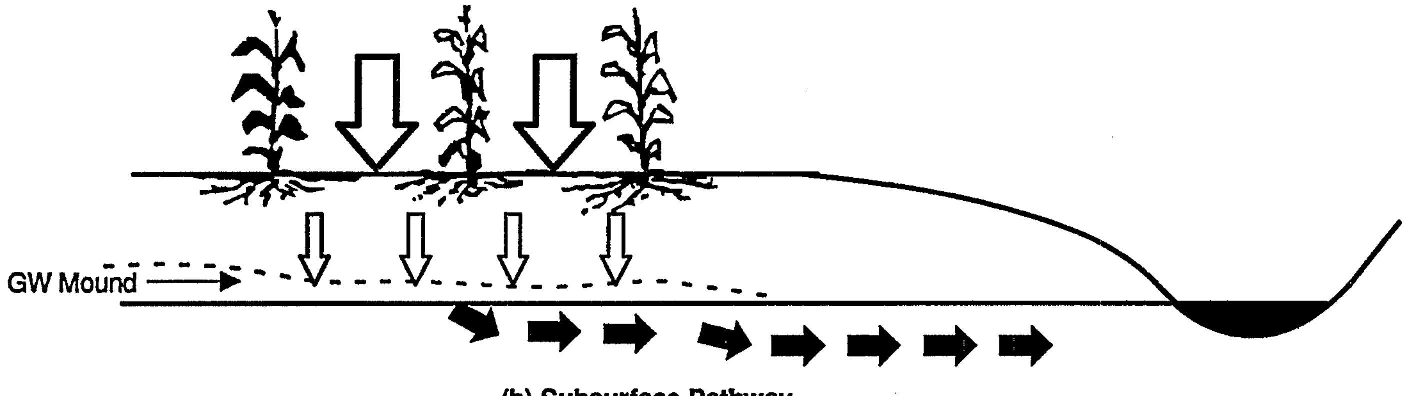
### 5.6.1 Technology Description

Slow rate land application is a soil-based wastewater treatment method designed to apply intermittently unchlorinated primary or secondary treatment effluent at a controlled rate to a vegetated soil surface of moderate to slow permeability (Figure 5-5). The wastewater is applied via sprinklers or flooding of furrows. Following application, the wastewater infiltrates the land surface and percolates through the soil profile to the ground-water table. A tailwater return system is usually provided to contain and recycle wastewater runoff from the site due to excessive application or precipitation. It consists of a collection pond, pump, and return pipeline. A storage reservoir must also be provided for adverse weather conditions, crop cultivation and harvesting, and emergencies.

The relatively low application rates on natural vegetated soil surfaces provide the potential for slow rate systems to produce the highest treatment levels of the land application methods. Wastewater constituents are removed in the soil matrix by filtration, adsorption, ion exchange, pre-




**(a) Application Pathway**




**(b) Subsurface Pathway**

**Figure 5-5. Schematic of a Slow Rate Land Application System.**

cipitation, microbial action, and plant uptake (WPCF, 1990). Part of the water is lost to evaporation and plant transpiration. Organics are removed by soil adsorption and biochemical oxidation. Nitrogen is removed primarily by crop uptake, but denitrification can also be significant. Chemical immobilization and plant uptake are mechanisms of phosphorus removal. Metals, certain toxic organics, and pathogens are also effectively removed. Typical treatment performance data are presented in Table 5-4.

**Table 5-4. Typical Slow Rate Land Application Treatment Performance[a]**

| Parameter | Loading (kg/ha x d) | Removal (%) | Comments |
|---|---|---|---|
| $BOD_5$ | 3–11 | 94–99 | Percolate typically <1 mg/L |
| Nitrogen | 0.3–2.4 | 65–95 | Removal efficiency depends on crop and crop management practices |
| Phosphorus | 0.1–3.0 | 75–99 | Plant uptake accounts for approximately 25 percent |
| Toxic organics | NA | Varies with structure | Limited data suggest effective removals are provided for volatile and biodegradable organics |
| Pathogens | NA | >99 | Nearly complete removal |

[a]EPA, 1981; WPCF, 1990.
NA = not applicable

Vegetation is an important component that serves to extract nutrients, control erosion, and maintain soil permeability. It should be selected and managed according to the site conditions and treatment objectives.

Either cultivated or forested sites are used. In cold climates, wastewater storage is required during most of the nongrowing seasons; however, year-round application is often employed on forested sites.

### 5.6.2 Applicability and Status

Slow rate land application for wastewater treatment is a proven technology for municipal and other organic wastewaters. Used for over one hundred years, it has evolved from a "disposal" method to one that can be used to recycle wastewater onto agricultural crops, forests, or park lands. Its use is limited primarily by land suitability, climate, and land costs.

### 5.6.3 Advantages

Slow rate land application is well suited for treatment of wastewater from rural communities and seasonal industries such as vegetable canning. It can provide an economic return from the reuse of water and nutrients for irrigation of landscaped areas or production of marketable, commercially processed crops. It also provides ground-water recharge. Of the various land treatment methods, slow rate land application is the least limited by surface slopes.

### 5.6.4 Disadvantages

Because of the vegetation component, slow rate land application is limited by climate and nutrient requirements of the vegetation. Climate affects the growing season and will dictate the period of wastewater application and storage requirements. Crop water tolerances, nutrient requirements, and nitrogen removal capacity of the vegetation-soil complex limit the hydraulic loading rate. Application must be suspended during wet periods or frozen soil conditions. Because of the limits on the hydraulic loading rate, the area of land necessary is significantly larger than for other land application methods.

### 5.6.5 Design Criteria

#### *5.6.5.1 Site Selection*

Important site-selection criteria include soil characteristics, ground-water conditions, and topography. Soil characteristics are important for wastewater treatment and crop management. Generally, loamy soils (loamy sands to clay loams) are best suited for slow rate land application systems. Finer texture soils (clays) do not drain well, retaining water for long periods, which makes crop management more difficult. Coarse texture soils (sands) can accept higher application rates and do not retain water, which may be important where crops with low moisture tolerance are used. In addition to texture, unsaturated depth of the soil is important. Adequate depth must be provided for root development and wastewater treatment. An unsaturated depth greater than 1.0 m (3.3 ft) to the top of any ground-water mound is usually necessary. Greater depths may be necessary for deep-rooted crops. In some cases, subsurface drainage can be provided where shallow ground water occurs. Ground-water mound height analysis is usually necessary to determine if the required unsaturated depth can be maintained during system operation.

Slope, relief, and susceptibility to flooding are important topographic features to consider. Where crop cultivation is to be practiced, slopes greater than 15 percent should be avoided. Uncultivated crops, such as pasture, can be irrigated on slopes of 15 to 20 percent. Woodlands have been irrigated successfully with sprinklers on slopes of up to 40 percent. Site relief, or the differences in surface elevations across the site, is an important consideration for wastewater distribution to the points of irrigation and wastewater ponding in depressions. Flood-prone areas should be avoided unless the land application system is designed to be an integral part of the flood management plan.

Land area requirements range from 25 to 220 ha (60 to 550 ac) for treatment of 3,780 $m^3$/d (1 mgd). Land for pretreatment facilities, storage, roads, and buffer zones is additional.

#### 5.6.5.2 Crop Selection

Important criteria for crop selection are climate, soil characteristics, wastewater characteristics and application rate, and available management skill, labor, and equipment. On sites where wastewater treatment rather than crop water requirements are the objective, the vegetation selected should have high nitrogen uptake capacity, high consumptive use or evapotranspiration potential, high soil-moisture tolerance, low sensitivity to wastewater constituents, and minimum management requirements. Crops meeting these requirements include various perennial forage grasses, turf grasses, some tree species, and some field crops. These include reed canary grass, tall fescue, Bermuda grass, perennial rye grass, Italian rye grass, and orchard grass. Mixed hardwoods and pines are the most common tree crops for systems with higher application rates. Where the wastewater is for crop irrigation, the crops listed above may be grown, as well as other crops such as legumes, cotton, soybeans, safflower, grains, and some fruit crops including citrus, apples, and grapes. Crop selection guidance can be obtained from local agricultural extension agents and available texts (CSWRCB, 1984; EPA, 1981; WPCF, 1990).

#### 5.6.5.3 Preapplication Treatment

The degree of pretreatment required for slow rate land application is dependent on public health considerations, nuisance control, soil and crop considerations, nitrogen concentration, and distribution method. In general, primary treatment is acceptable for isolated, restricted-access sites where the crops are not intended for human consumption. State requirements vary for controlled agriculture irrigation for various crops, for turf irrigation in parks, and on golf courses. Where nitrogen loading is critical or where other wastewater constituents can affect plant growth, additional pretreatment may be required.

#### 5.6.5.4 Hydraulic Loading Rate

The design hydraulic loading rate is controlled by either the hydraulic conductivity of the soil, nitrogen loading, or the water requirement of the crop. Local rates of precipitation and evapotranspiration potential of the crop must also be considered. The maximum daily application rate must not exceed the soil percolation rate. Typically, the maximum application rate used is 2 to 4 percent of the saturated hydraulic conductivity of the least permeable soil horizon within 2.4 m (8 ft) of the surface. This rate must be adjusted for precipitation and evapotranspiration during the application period. Where nitrogen contributions to the ground water must be controlled, the nitrogen mass loading will control the hydraulic loading rate. Crop requirements for water or tolerances to salinity or other wastewater constituents may also be a controlling factor. Methods for estimating the design hydraulic loading rate may be found elsewhere (EPA, 1981; WPCF, 1990).

The application period must also be determined. Applications cannot be made during very wet periods, during freezing weather, or during crop harvesting activities. These periods must be estimated to determine the total land area requirements.

#### 5.6.5.5 Wastewater Distribution

Common methods of wastewater distribution used in slow rate land application systems include sprinkling, flooding, and drip irrigation. Sprinkling is the most common method because is can be adapted to a wide range of soil and topographic conditions and is suited to a variety of crops. Either fixed, portable, center pivot, or traveling gun systems are used. Flooding of furrows or other bermed areas is used where it is compatible with conventional agricultural irrigation practices. Drip irrigation requires that the applied wastewater have a high level of pretreatment and be low in iron, hydrogen sulfide, and total bacteria to prevent emitter clogging. Therefore, this method is restricted to systems where the resource value of the wastewater is high. Guidance for distribution system design can be found elsewhere (EPA, 1981; WPCF, 1990; Booher, 1974; Hart, 1975; SCS, 1983; Irrigation Association, 1983).

#### 5.6.5.6 Storage Requirements

Storage of wastewater must be provided during periods when applications cannot be made. A water balance for each month of operation should be calculated to establish these requirements (EPA, 1981).

### 5.6.6 Capital Cost Sensitivity

Land, earthwork, distribution system, and storage facilities are the most significant capital costs. The land area required is dependent on the nature of the irrigated crop, soil characteristics, and acceptable application periods. Distribution costs can be affected by the type of system selected and site topography. In cold, wet climates, storage requirements can be significant. Other costs that should be considered are underdrain installation, if needed, transmission of wastewater to the treatment site, and pretreatment.

### 5.6.7 O&M Requirements

The primary O&M requirements of slow rate land application systems are crop management and servicing of the distribution tailwater return systems. Agricultural crops require the most intensive management, while forest application requires the least management. Management tasks may include soil tillage, planting and harvesting of crops, nutrient control, pH adjustment, tailwater return system maintenence, and sodium and salinity control (EPA, 1981). No special equipment other than the appropriate agricultural equipment is required. Costs of operation are associated primarily with labor costs. Other costs include power for wastewater pumping and equipment depreciation. Total labor requirements, excluding maintenance of the pretreatment works, are estimated to be approximately 15 to 20 cents/1,000 gal of applied water.

### 5.6.8 Construction Issues

Construction factors include site preparation and installation of runoff controls, irrigation piping, a tailwater system, return systems, and storage facilities. Since sustained wastewater infiltration is an important component of successful system operation, it is critical that construction activity be limited on the application site. Where storm-water runoff can be significant, measures must be taken to prevent excessive erosion including terracing of steep slopes, contour plowing, no-till farming, and the establishment of grass border strips and installation of sediment control basins.

### 5.6.9 Monitoring

Monitoring requirements may cover volumes, rates, frequencies, and patterns of wastewater applications, ground-water quality and elevations, soil fertility, and plant tissue. Ground-water monitoring may be required by the local regulatory agency to evaluate system performance. Soil fertility should be evaluated periodically to determine if soil amendments are necessary. Trace elements should also be analyzed to avoid toxic accumulations. Plant tissue analysis is used to determine whether there are deficient or toxic levels of elements.

### 5.6.10 Residuals

Residuals produced by slow rate land application systems are limited to harvested crops and crop residues.

## 5.7 Subsurface Infiltration

### 5.7.1 Technology Description

Subsurface wastewater infiltration systems (SWISs) are subgrade land application systems. The soil infiltration surfaces are exposed in buried excavations that are generally filled with porous media. The media maintains the structure of the excavation, allows the free flow of pretreated wastewater over the infiltrative surfaces, and provides storage of wastewater during peak flows. The wastewater enters the soil where treatment is provided by filtration, adsorption, and biochemical reactions. Ultimately, the treated wastewater enters and flows with the local ground water.

Various SWIS designs have been developed for use depending on the site and soil conditions encountered. The designs differ primarily in where the infiltrative surface is placed (Figure 5-6). The surface may be exposed within the natural soil profile (conventional) or at or above the surface of the natural soil (at-grade on mound systems). The elevation of the infiltrative surface is critical because of the need to provide an adequate depth of unsaturated soil between the infiltrative surface and a limiting condition (i.e., bedrock or ground water).

The geometry of the infiltrative surface also varies. Long, narrow infiltrative surfaces (trenches) are preferred. Wide infiltrative surfaces (beds) and deep infiltrative surfaces (pits and deep trenches) do not perform as well, even though they require less area.

Subsurface infiltration systems are capable of high levels of treatment for most domestic wastewater pollutants of concern (Table 5-5). Under suitable site conditions, removal of biodegradable organics, SS, phosphorus, heavy metals, and virus and fecal indicators is nearly complete.

The fate of toxic organics and metals has not been as well documented, but limited studies suggest that many of these constituents do not travel far from the system. Nitrogen is the most significant wastewater parameter that is not readily removed by the soil. Nitrate concentrations above the drinking water standard of 10 mg-N/L are commonly found in ground water below SWISs.

**Table 5-5. Typical Subsurface Wastewater Infiltration System Treatment Performance**

| Parameter | Applied Concentration (mg/L) | Removal (%) | References |
|---|---|---|---|
| $BOD_5$ | 130–150 | 90–98 | Siegrist et al., 1986<br>Univ. Wis., 1978 |
| Nitrogen | 45–55 | 10–40 | Reneau, 1977<br>Sikora & Corey, 1976 |
| Phosphorus | 8–12 | 85–95 | Sikora & Corey, 1976<br>Tofflemire & Chen, 1977 |
| Fecal coliforms | NA | 99–99.99+ | Gerba et al., 1975<br>Univ. Wis., 1978 |

NA = not applicable

### 5.7.2 Applicability and Status

Subsurface infiltration systems are well suited for treatment of small wastewater flows. Small SWISs, commonly called *septic tank systems*, are traditionally used in unsewered areas by individual residences, commercial establishments, mobile home parks, and campgrounds. Since the late 1970s, larger SWISs have been increasingly used by clusters of homes and small communities where wastewater flows are less than 1.1 L/s (25,000 gpd). They are a proven technology, but require specific-site conditions to be successfully implemented. SWISs are often preferred over mechanical treatment facilities because of their consistent performance with few O&M requirements, lower life cycle costs, and less aesthetic impact on the community.

### 5.7.3 Advantages

Because subsurface infiltration systems are buried, they are ideally suited for decentralized treatment of wastewater. For rural homes and business establishments, they are often the only method of wastewater treatment available. Some communities choose subsurface infiltration systems to avoid costly sewer construction. Where individual lots are not suited for their construction, suitable remote sites may be used to cluster homes onto a single SWIS, thereby limiting the extent of sewers. Alterna-

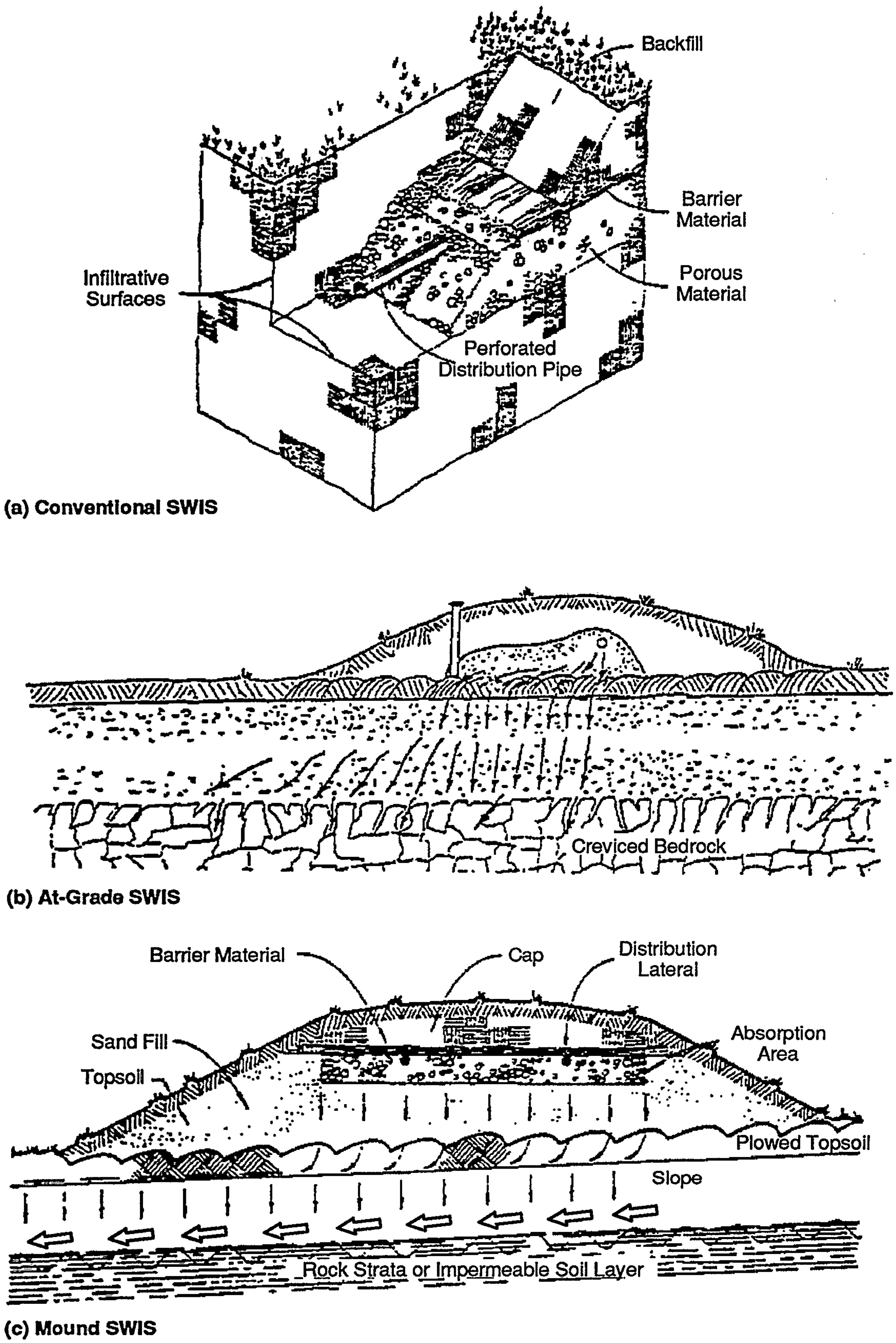



**Figure 5-6. Schematics of Subsurface Wastewater Infiltration Systems (SWISs).**

tively, wastewater from entire communities may be treated by an SWIS. Because the system is buried, the land area can be used as green space or park land. In addition, SWISs provide ground-water recharge.

### 5.7.4 Disadvantages

Use of SWISs are limited by site and soil conditions. Because the infiltrative surface is buried, it can be managed only by taking it out of service every 6 to 12 months to "rest." This requires that standby cells be constructed with alternating loading cycles. Therefore larger SWISs are usually restricted to well-drained sandy soils to reduce land area requirements. Because nitrogen is not removed effectively by SWIS, pretreatment to remove nitrogen may be necessary to prevent nitrate contamination above the drinking water standards in the underlying ground water.

### 5.7.5 Design Criteria

#### *5.7.5.1 Site Selection*

Site selection is based on the treatment capacity of the soils and the availability of sufficient land area of suitable topography. The treatment capacity of the soil depends primarily on texture, structure, and unsaturated thickness. No soil type provides optimum conditions for the removal of all wastewater constituents. Fine-texture soil, such as silt loams and clay loams, provides particularly favorable removal of conventional constituents. However, fine-texture soil has a relatively low hydraulic conductivity and reaeration rate. Therefore the hydraulic loading and infiltration basin rest cycle will control system sizing, and a large land area may be required. Coarse soil, such as sand, has a higher hydraulic conductivity and reaeration rate to allow higher hydraulic and organic loading and shorter cycles for basin "resting," which can reduce the required land area. However, coarse soil can be a less effective physical filter, have a lower cation exchange capacity, and allow more rapid percolation of wastewater through the vadose zone where most treatment occurs. Thus the treatment objectives may control system design. For large SWISs serving clusters of homes or communities (Table 5-6), sites with sand to sandy loam soils are typically selected. Site selection for individual home SWISs are regulated by local onsite system codes, which should be consulted before site investigations are made.

Unsaturated depth of soil is another critical site-selection criterion. A minimum of 1.5 to 2.5 m (5 to 8 ft) of unsaturated soil with relatively uniform hydraulic conductivity is necessary below the ground surface to provide necessary treatment. For large SWISs, ground-water mound height analysis must be performed to determine if the separation distance can be maintained during system operation. If the desired separation distance cannot be maintained, the hydraulic loading can be reduced. For large SWISs, the system should be located along a contour perpendicular to the ground-water gradient. Basin spacing can be increased or ground-water drainage provided in areas where limiting conditions place restrictions on conventional designs.

Topography should also be considered in site selection. Depressions, footslopes, concave slopes, flood plains, and other areas that exhibit poor surface and subsurface drainage should be avoided. Mild planar slopes, ridge lines, and shoulder slopes are best suited for an SWIS.

**Table 5-6. Typical Site Criteria for a Large SWIS[a]**

| Characteristic | Typical Application | Applications to Avoid |
|---|---|---|
| Site | | |
| Landscape position | Ridge lines, hill tops, shoulder slopes | Depressions, footslopes, concave slopes, flood plains |
| Topography | Planar, mildly undulating, slopes < 12 percent | Complex slopes, slopes >18 percent |
| Soil Characteristics | | |
| Texture | Sands, sandy loams | Clay loams, clays |
| Structure | Granular, blocky | Platy, prismatic, columnar |
| Mineralogy | <20 percent expansive (2:1) clays | >20 percent (2:1) clays |
| Permeability | Moderate, rapid | Very rapid, moderately slow to very slow, hydraulically restrictive horizons |
| Bulk density | Slight to moderate resistance to penetrometer | Moderate to strong resistance to penetrometer |
| Drainage | Moderately well, well, somewhat poorly drained | Extremely well, very poorly, excessively well drained |
| Hydrogeology | | |
| Depth to phreatic surface/bedrock | >1.5 m (4.5 ft) | <1.5 m (4.5 ft), sole source aquifers |
| Transmissivity | High | Low |
| Flow boundaries | Near discharge point | Near no-flow boundary |

[a]SWISs serving individual homes are adaptable to a broader range of soil and site conditions. Local codes should be consulted.

#### *5.7.5.2 Hydraulic Loading Rate*

The design hydraulic loading rate is determined by soil characteristics, ground-water mounding potential, and applied wastewater quality. Clogging of the infiltrative surface will occur in response to prolonged wastewater loading, which will reduce the capacity of the soil to accept the wastewater. However, if the loading is controlled, biological activity at the infiltrative surface will maintain waste accumulations in relative equilibrium so that reasonable infiltration rates can be sustained.

Selection of the design hydraulic loading rate must consider both soil and system design factors. Typically, design rates for larger SWISs are based on detailed soil analyses and experience, rather than measured hydraulic conductivities. Commonly used loading rates are presented in Table 5-7; however, these rates should be adjusted up or down depending on the specific site conditions, design concept, and applied wastewater quality (WPCF, 1990).

#### *5.7.5.3 Wastewater Pretreatment*

The minimum of primary treatment is required of wastewater before application to an SWIS. Septic tanks are commonly used for smaller sources and Imhoff tanks for larger ones. Higher levels of treatment can reduce SWIS size or prolong system life, but this must be weighed against the increased costs of pretreatment and potential damage from poor maintenance of the system. Where pretreatment is desirable, aerobic biological treatment processes are generally used.

#### *5.7.5.4 Design Concept*

SWIS designs should be adapted to the specific site and soil conditions and wastewater characteristics. The initial design concept should strive to include the following features:

- Narrow trenches, 0.2 to 1.0 m (0.5 to 3.0 ft) wide, excavated parallel to surface or ground-water piezometric surface contours (based on analysis or ground-water mounding potential) with level bottom surfaces
- Shallow placement of the infiltrative surfaces, less than or equal to 0.6 m (2 ft) below final grade
- Pretreatment capability to remove organics, suspended solids, grease, oils, etc. to concentrations less than or equal to typical domestic septic tank effluent
- Uniform dosing of infiltrative surfaces one to four times daily
- Multiple cells (3 to 4 minimum) to allow annual or semiannual resting and standby capacity for operational flexibility
- Devices for monitoring daily wastewater flows, infiltrative surface ponding, and ground-water elevations

Modifications to this design concept are usually required if site limitations cannot be removed. Guidelines for adapting SWISs to common site limitations encountered are presented elsewhere (Converse & Tyler, 1990; Converse et al., 1989; EPA, 1980; Siegrist et al., 1986; WPCF, 1990).

**Table 5-7. Typical Hydraulic Loading Rates on Horizontal Soil Infiltrative Surfaces Treating Domestic Septic Tank Effluent[a]**

| Soil Texture | Hydraulic Initiative Surface Hydraulic Loading Rate (cm/d[b]) |
|---|---|
| Gravel, very coarse sand | Not recommended |
| Coarse to medium sand | 4.0 |
| Fine sand, loamy sand | 3.2 |
| Sandy loam, porous loam | 2.1 |
| Loam, silt loam (moderate to strong blocky structure) | 2.5 |
| Clay loam (moderate to strong blocky structure) | 1.0 |
| Clay loams and clays with appreciable shrink-swell potential or weak, columnar, or prismatic structure | Not recommended |

[a]Adjustments to these rates may be necessary for specific applications to account for other soil factors and design concepts. See WPCF, 1990.
[b]Conversion: 1 cm/d = 0.24 gal/ft$^2$ d

### 5.7.6 Capital Cost Sensitivity

Land and earthwork are the most significant capital costs. Where select fill must be used to bed the primary infiltrative surface, the cost of transporting the material also becomes significant. Other costs that should be considered are pretreatment costs and transmission of the wastewater to the treatment site.

### 5.7.7 O&M Requirements

A well-designed SWIS requires limited operator attention. Management functions primarily involve tracking system status, testing for solids accumulation, evaluating pump performance, and monitoring system controls. Monitoring performance of pretreatment units, mechanical components, and wastewater ponding levels above the filtration surface are essential. If a change in status is noted, operator intervention may be required. Routine servicing of SWIS may be limited to annual or semiannual alternating of the infiltration cells.

### 5.7.8 Construction Issues

A frequent cause of early SWIS failure is poor construction. If the soil structure is damaged during construction, the system may not be able to accept the design hydraulic loading rates. To avoid damage, a construction plan should be developed that addresses the type of construction equipment, construction procedures, site access, site preparation, and existing soil conditions (Univ. Wis., 1978; WPCF, 1990). Compaction of the infiltrative surface should be avoided by using proper procedures and low load-bearing construction equipment. After excavating to grade, the infiltrative surface should be carefully scarified. Construction should not proceed if the soil moisture is near the plastic limit or frozen conditions exist.

### 5.7.9 Monitoring

Monitoring requirements cover applied wastewater flows and quality, ponding levels above the infiltrative surfaces, ground-water elevations below the system, and, if appropriate, ground-water quality. Flows should be recorded daily. Ponding levels should be measured biweekly or monthly. Water quality measurements should be performed at least quarterly.

### 5.7.10 Residuals

No residuals are produced from subsurface infiltration systems. In an individual home SWIS (septic tank systems), where the septic tank is considered an integral part of the system, septage is generated within the tank. Septage should be removed every three to five years. The septage is typically treated at a municipal treatment plant or is land spread (EPA, 1984).

## 5.8 Pressure Sewers

### 5.8.1 Technology Description

Pressure sewer systems generally use smaller pipe diameters than conventional sewers and are operated with pumping instead of gravity. In less populated areas they usually result in lower construction costs relative to conventional sewer systems. Pressure sewers are considerably independent of slope, and systems have been developed and applied to reduce the high capital cost of sewer systems that have been designed in accordance with accepted design parameters, namely slope and velocity. Pressure sewer systems involve a number of pressurizing inlet points and an outlet to a treatment facility or to a downstream gravity sewer, depending on the application.

The two major types of pressure sewer systems are the grinder pump (GP) system and the septic tank effluent pump (STEP) system. The major difference between the two systems is in the onsite equipment and layout. Neither pressure sewer system alternative requires any modification of household plumbing.

In both designs household wastes are collected in the sanitary sewer and conveyed by gravity to the pressurization facility. The onlot discharge piping arrangement includes at least one check valve and one gate valve to permit isolation of each pressurization system from the main sewer. GPs can be installed in the basement of a home to provide easier access for maintenance and greater protection from vandalism.

A GP pressure sewer has a pump at each service connection. The pumps are 0.75 kW (1 hp) or more, require 110V or 220V, and are equipped with a grinding mechanism that macerates the solids. The head provided by the pumps are typically about 15 to 30 m (50 to 100 ft) and flow rates vary widely. The pumps discharge into a pressurized pipe system that terminates at a treatment plant or a gravity collector. Because the mains are pressurized there is no infiltration into them; however, infiltration and inflow in the house sewers and the pump wells can occur. In areas where the GP sewer system has replaced septic tank and leaching field systems, these may be retained for emergency overflow, but they should be separated from the pump well by a gate valve that is opened only when necessary to accommodate emergency overflow from the GP unit; otherwise, the septic tank and leaching field can become sources of large volumes of infiltration.

Electrical service is required at each service connection. The pipe network typically has no closed loops. The sewer profile often parallels the ground surface profile. Horizontal alignment can be curvilinear. Plastic pipe is typically used. Service connection diameters are typically 1 1/4 in. (30 mm). Cleanouts are used to provide access for flushing. Automatic air release valves are required at and slightly downstream of summits in the sewer profile. Because of the small diameters and curvilinear horizontal and vertical alignment, excavation depths and volumes are typically much smaller for a GP pressure sewer than for conventional sewers, sometimes requiring only a chain trencher.

Centrifugal and positive displacement pumps have been used in GP systems. Positive displacement pumps have a discharge nearly independent of head. Although this may simplify some design problems, it results in some additional operational ones. The choice is typically up to the design engineer's preference.

Several dwelling units or other service locations can be clustered to a single pump well, which should have an increased working volume depending on the total population equivalent it serves. Clustered service connections have often led to disputes over billing and responsibility for nuisance conditions and service calls. Duplex pump wells are often used on clustered, commercial, institutional, or other larger services.

A STEP pressure sewer typically has a septice tank and a pump at each service connection. The pumps discharge septic tank effluent into a pressurized pipe system that terminates at a treatment plant or a gravity sewer. Because the mains are pressurized, there will be no infiltration into them, but infiltration and inflow into the house sewers and the septic/interceptor tanks should be minimized during the construction of onlot facilities. The volume of these tanks is usually about 3,800 L (1,000 gal). They remove grit, settleable solids, and grease. The pumps typically are 0.25 to 0.37 kW (1/3 to 1/2 hp) and require 110 to 120 V. The head provided by the pumps is typically about 9 to 15 m (30 to 50 ft) and 1 L/s (15 gpm), but flow rates vary widely. The working volume of the pump well is typically 150 to 230 L (40 to 60 gal). The discharge line from the pump is equipped with at least one check valve and one gate valve. Electrical service is required at each service connection. The pipe network can contain closed loops but typically does not. The sewer profile typically parallels the ground surface profile, and the horizontal alignment can be curvilinear. Plastic pipe is typically used. The service lateral diameter is typically 1 1/4 in. (30 mm). Cleanouts are used to provide access for flushing. Automatic air release valves are required at and slightly downstream of summits in all pressure sewer profiles. Because of the small diameters, curvilinear horizontal and vertical alignments, excavation depths and volumes are typically much smaller for pressure sewers than for conventional sewers, sometimes requiring only a chain trencher.

A service connection at an elevation higher than the hydraulic guide line may be served by gravity, avoiding the need for a pump. The use of a gravity connection in this situation is advantageous because a pump would be subject to siphoning and air-binding. Hybrid designs are common in current practice. Septic tanks with integral pump vaults are available; they reduce onlot excavation. Existing septic tanks are not used owing to their propensity to leak and be a source of infiltration and inflow.

### 5.8.2 Applicability and Status

Pressure sewer systems are most cost-effective where housing density is low, where the terrain has undulations with relatively high relief, and where the system outfall must be at a higher elevation than most or all of the service area. They can also be effective where flat terrain is combined with high ground water or bedrock, making deep cuts and/or multiple lift stations excessively expensive. They can be cost-effective even in densely populated areas where the terrain will not accommodate gravity sewers.

Since pressure systems do not have the large excess capacity typical of conventional gravity sewers, they must be designed with a balanced approach keeping future growth and internal hydraulic performance in mind.

Where pressure sewers are indicated, the choice between GP and STEP systems depends on two main factors: First, the costs of onlot facilities will be typically over 75 percent of the total system cost. Thus there will be a strong incentive to use a system with less expensive onlot facilities for a particular project. STEP systems may allow some gravity service connections, thus lowering onlot costs. GP systems must have a pump at each service connection to grind the solids.

Second, GP systems require a higher velocity because they carry heavy solids and grease. STEP systems will better tolerate the low flow conditions that occur in locations with a highly fluctuating seasonal occupancy and in locations with slow buildout from a relatively small initial population to the ultimate design population.

GP units are preferable for use at individual homes that discharge into a conventional gravity sewer at a higher elevation, while STEP systems are most compatible with small-diameter gravity sewers.

### 5.8.3 Advantages/Disadvantages

Key advantages of pressure sewer systems include:

- Pressure sewer systems are less expensive than conventional gravity sewerage, due to the fact that the cost of on-property facilities represents a major portion of the capital cost of the entire system. This can become an economic advantage since on-property components are not required until the house is constructed and then borne directly by the homeowner. Low front-end investment makes the present-value cost of the entire system lower than that of conventional gravity sewerage, especially in new development divisions where it may take many years before homes are built on all lots.
- Due to the fact that the wastewater is pumped, gravity flow is not necessary, and the strict alignment and slope restrictions for conventional gravity sewers can be discarded. Network layout does not depend on ground contours: pipes can be laid in any location, and extensions can be made in the street right-of-way at a relatively small cost without damage to existing structures or to the natural environment.
- Because the pipe size and depth requirements are reduced, material and trenching costs are significantly lower.
- Manholes are eliminated; low-cost clean outs and valve assemblies are used instead and are spaced further apart than manholes in a conventional system.
- Infiltration is greatly reduced or may even be eliminated, resulting in reductions in pipe size.
- The user pays for the electricity used to operate the pump unit. The resulting increase in the user's electricity bills is small, but this small increase often replaces large bills for central pumping, which is eliminated by the pressure system. In other words, the responsibilities for installation and paying for pumping are transferred from the authorities to the users; the authorities are involved in pump and collection maintenance only.
- Final treatment may be substantially reduced, both in hydraulic and organic loading in the case of STEP systems. For GP systems hydraulic loadings are reduced.
- More flexibility is allowed in siting final treatment facilities, and this may help reduce long outfall lines. The location can be chosen to facilitate site work.

Key disadvantages of pressure sewer systems include:

- A high level of institutional involvement is required, since the pressure system has many mechanical components located all over the area served.
- The O&M cost for a pressure system is often higher than that of a conventional gravity system due to the high number of pumps in use. However, the existence of lift stations in a conventional gravity sewer can quickly reverse this situation.
- Yearly preventive maintenance calls are usually scheduled for onlot components at pressure sewers, and STEP systems also require pumpout of interceptor tanks at prescribed (3 to 5 yr) intervals.
- Public education is necessary so that the user knows how to deal with emergencies and how to avoid blockages or other emergency maintenance initiators.
- Malodors and corrosion are potential problems since the wastewater in the collection sewers is usually anaerobic. Proper ventilation and odor control must be provided for in the design, and noncorrosive components should be used. Air release valves are often vented to soil beds to minimize odor problems, and special discharge and treatment designs are required to avoid terminal discharge problems.

### 5.8.4 Design Criteria

A wide variety of design flows has been used. When positive displacement GP units are used, the design flow is obtained by multiplying the pump discharge by the maximum number of pumps expected to be operating simultaneously. When centrifugal pumps are used, the equation used is: $Q = 20 + 0.5D$, where Q is the flow in gpm and D is the number of equivalent dwelling units served. The operation of the system under various assumed conditions should be simulated by computer as a check on the adequacy of the design. No allowances for infiltration and inflow should be required. No minimum velocity is generally used in design, but GP systems must attain 3–5 fps at least once per day. A Hazen-Williams coefficient (C) = 130 to 140 is suggested for hydraulic analysis. Pressure mains generally use 5 cm (2 in.) or larger PVC pipe (SDR 21), although at least 7.6 cm (3 in.) pipe is preferred owing to the availability of standard tapping equipment. Rubber-ring joints are preferred over solvent welding due to the high coefficient of expansion for PVC pipe. High-density polyethylene (HDPE) pipe with fused joints is widely used in Canada. Electrical requirements, especially for GP systems, may necessitate rewiring and electrical service upgrading of dwellings served. Pipes are generally buried to at least the winter frost penetration depth; in far northern sites insulated and heat-traced pipes are generally buried at a minimal depth. GP and STEP pumps are sized to accommodate the hydraulic grade requirements of the system. Discharge points must employ drop inlets to minimize odors and corrosion. Air release valves are placed at high points in the sewer and often are vented to soil beds. GP effluent is generally about twice the strength of conventional sewer wastewater (e.g., BOD and TSS of 350 mg/L). STEP effluent is, of course, pretreated and has a $BOD_5$ of 100 to 150 mg/L and SS of 50 to 70 mg/L. Both can be assumed to be anaerobic and potentially odorous if subjected to turbulence (stripping of gases such as $H_2S$).

### 5.8.5 Capital Costs

Pressure sewers are generally more cost-effective than conventional gravity sewers in rural areas. However, even though capital cost savings of 90 percent have been achieved, no universal statement of savings is possible owing to the site specificity of each system. Recent evaluations of the actual costs of pressure sewer mains and appurtenances (essentially the same for GP and STEP) and items specific to each type of pressure sewer have yielded the following data: Average installed unit costs for pressure sewer mains and appurtenances are presented in Table 5-8. Average unit costs for GP services and appurtenances are presented in Table 5-9. Average unit costs for STEP services and appurtenances are presented in Table 5-10.

Within Table 5-8 the linear cost of mains can vary by a factor of 2 to 3, depending on the type of trenching equipment and local costs of high-quality backfill and pipe.

### 5.8.6 O&M Requirements/Costs

Energy costs are borne by the homeowner. For GP systems this cost may vary from about $1.00 to $2.50/month depending on the horsepower of the unit. For STEP units the costs are almost always less than $1.00/month.

Preventive system maintenance is normally carried out annually for each unit with monthy maintenance of other mechanical components. STEP systems also require periodic pumping of tanks. Total O&M costs include troubleshooting, inspection of new installations, and

**Table 5-8. Average Installed Unit Costs for Pressure Sewer Mains and Appurtenances (Mid-1991)[a]**

| Item | Unit Cost ($) |
|---|---|
| 2 in. mains | 7.50/LF |
| 3 in. mains | 8.00/LF |
| 4 in. mains | 9.00/LF |
| 6 in. mains | 11.00/LF |
| 8 in. mains | 14.00/LF |
| Extra for mains in asphalt concrete pavement | 5.00/LF |
| 2 in. isolation valves | 250/each |
| 3 in. isolation valves | 275/each |
| 4 in. isolation valves | 350/each |
| 6 in. isolation valves | 400/each |
| 8 in. isolation valves | 575/each |
| Automatic air release stations | 1,500/each |

[a]EPA, 1991
LF = linear feet

**Table 5-9. Average Unit Costs for Grinder Pump Services and Appurtenances (Mid-1991)**

| Item | Unit Cost ($) |
|---|---|
| 2 hp centrifugal GP | |
| List price | 1,200/each |
| Quantity price | 600/each |
| Simplex GP package | |
| List price with 30 in. vault | 4,100/each |
| Quantity price with 30 in. vault | 1,800/each |
| Installation | 500–1,500/each |
| 4 in. building sewer | 16/LF |
| 1.25 in. service line | 6/LF |
| Abandon septic tank | 400/each |

**Table 5-10. Average Unit Costs for STEP Services and Appurtenances (Mid-1991)**

| Item | Unit Cost ($) |
|---|---|
| Effluent pump list price | 300–800/each |
| Effluent pump quantity price | 200–500/each |
| Simplex factory package list price | 2,500–3,000/each |
| Quantity package price w/external vault | 700–1,500/each |
| Quantity package price w/internal vault | 600–1,200/each |
| New septic tank | 600–1,000/each |
| Installation (retrofit of existing tank) | 600–1,200/each |
| Installation (with new septic tank) | 1,000–1,500/each |
| 4 in. building sewer | 14–18/LF |
| 1.25 in. service line | 4–8 /LF |
| Abandon septic tank | 300–500/each |

LF = linear feet

responses to problems are estimated at $100–200/yr/unit. The breakdown on emergency service calls on selected GP and STEP sewer projects are presented in Table 5-11.

Mean time between service calls (MTBSC) data vary greatly, but values of 4 to 10 yr for both GP and STEP units are reasonable estimates for quality installations.

### 5.8.7 Construction Issues

It is very important for the engineer to ensure that the building sewer and all onlot piping and structures are free of infiltration and inflow (I/I) and sump pump/foundation drain connections, via proper testing prior to hookup. Proper bedding of PVC pipe is also particularly important, and accurately prepared "as built" plans must be provided to the system owner, including lot facility plans showing the onlot component locations. Each section of pipe and all tanks should be tested prior to covering with backfill to ensure water-tight conditions. Videotaping before and after onlot construction will resolve most restoration claims.

**Table 5-11. Distribution of Causes for Call-out Maintenance on Selected GP and STEP Pressure Sewer Projects**

| | (% of occurrences) | |
|---|---|---|
| Category | GP Projects | STEP Projects |
| Electrically related | 25–40 | 40–60 |
| Pump related | 20–25 | 10–30 |
| Miscellaneous | 20–30 | 20–40 |
| Pump vault or tank related | 5–15 | 1–5 |
| Piping related | 5–15 | 1–10 |

### 5.8.8 Monitoring

Detailed daily records of maintenance and annual summaries should be provided. Also specific records for each unit should be kept with the lot facility plan in order to permit maintenance staff to evaluate potential problems prior to arrival at the site of the emergency call. On larger flow sources, cycle counters may be useful to track any trends, just as periodic line-pressure checks can alert the O&M staff to impending needs.

### 5.8.9 Residuals

STEP systems incorporate septic (interceptor) tanks, and therefore need periodic pumping to remove trapped grease and solids. Usually this is required every 3 to 5 yr, but some systems attempt to lengthen this pumping frequency by requiring biannual inspections to avoid unnecessary pumping. Heavy grease generators, such as commercial sources, may need to be pumped out annually or semiannually.

Owing to their tendency to accumulate grease in their tankage, GP units are often pumped as part of the annual preventive maintenance check.

## 5.9 Small Diameter Gravity Sewers

### 5.9.1 Technology Description

A small diameter gravity (SDG) sewer collects effluent from septic tanks at each service connection and transports it by gravity to a treatment plant or a gravity sewer. Such systems are also known as diameter effluent sewers, effluent drains, and small bore sewers. The volume of the septic tanks in these systems is often 1,000 gal but varies widely. The tanks remove grit, settleable solids, and grease, and they attenuate peak flows. Both the horizontal and vertical alignments of the pipes can be curvilinear. The pipe network includes no closed loops. Uphill sections can be used provided there is enough elevation head upstream to maintain flow in the desired direction and there is no backflow into any service connection. Minimum diameters are usually 10 cm (4 in.), but smaller sizes have been used successfully. Plastic pipe is typically used since it is economical in small sizes and resists corrosion by the septic wastewater. Cleanouts are used to provide access for flushing. Manholes are used infrequently, usually only at the major junctions of main lines. Air release risers may be required at or slightly downstream of extreme summits in the sewer profile. Because of the small diameters and flexible slope and alignment, excavation depths and volumes are typically much smaller than with conventional sewers, sometimes requiring only a chain trencher.

Two varieties of SDG systems have been used in the United States: variable grade and minimum grade. Variable grade systems follow surface contours rather strictly, taking advantage of the flexibility of horizontal and vertical alignment when there is enough elevation head to maintain flow in the desired direction and there is no backflow into any service connection at design flow. For minimum grade systems, minimum downward slopes are imposed. Recent designs blend both of these approaches, allowing variable grade but minimizing the number of flooded sections.

Individual service connections can be equipped with a septic tank effluent pump (STEP) unit, creating a hybrid of the STEP pressure sewer. The use of STEP connections is advantageous when excavation costs can be reduced enough to offset increased onlot costs. Hybrid designs are common in current practice. Inline lift stations can also be used if required by the terrain, but their use may signal a need to reevaluate the cost-effectiveness of an SDG system vs. other alternative sewer systems. Use of hybrid systems should be considered before evaluating high-cost lift stations.

While two-compartment septic tanks may be more efficient at retaining solids, single-compartment tanks have performed well. Also, several dwelling units or other service locations can be clustered to a single septic tank, which should have an increased volume depending on the total population equivalent it serves.

Other variations of SDG sewers—known as "simplified sewer systems," "steep slope sewers," and "flat-grade sewers"—are in use in Nebraska as well as in South America, Africa, and Asia. These systems employ smaller diameter pipes, simplified manhole requirements, shallower depths, and other cost-saving features in comparison to conventional gravity sewers.

### 5.9.2 Applicability and Status

Approximately 250 SDG sewers have been financed in the United States by the EPA Construction Grants Program. Many more have been financed with private or local funding in North America.

SDG sewer systems are likely to be most cost-effective where the housing density is low, the terrain has undulations of low relief, and the elevation of the system terminus is lower than all or nearly all of the service area. They can also be effective where the terrain is too flat for conventional gravity sewers without deep excavation.

SDG sewer systems do not have the large excess capacity typical of conventional gravity sewers. Therefore they must be designed with an adequate allowance for future growth if that is desired. These systems were introduced in the United States in the mid-1970s, but have been used in Australia since the 1960s.

### 5.9.3 Advantages/Disadvantages

Key advantages of SDG systems include:

- Construction is faster than for conventional sewerage, requiring less time to provide service.
- O&M can be carried out by unskilled personnel.
- Any required lift and pumping station can be reduced in size and simplified owing to the nature of the wastewater.
- Elimination of manholes helps to further reduce inflow (further reducing the sizes of pipes), lift/pumping stations, and final treatment.
- Reduced excavation costs: Trenches for SDG sewer pipelines are typically narrower and not as deep as in conventional sewers.
- Reduced material costs: As the name indicates, SDG sewer pipelines are smaller than conventional sewers, reducing pipe and filling costs.
- Power requirements are negligible or low.
- Final treatment requirements are not only reduced hydraulically but are also scaled down in terms of organic loading since partial treatment is performed at the source.
- Reduced depth of mains minimizes additional construction costs due to high ground water or rocky conditions.

While SDG systems have no major disadvantages specific to temperate climates, some restrictions exist generally that limit their application:

- SDG sewers are not suitable to serve as combined sewers, even if pipe size could be increased. Due to the nature of variable grades and relatively flat slopes, solids drawn to SDG sewers will block them.
- For the same reason, SDG sewers cannot handle commercial wastewater having high grit or settleable solids levels. Restaurants may be hooked up if they are equipped with effective grease traps. Laundromats could be a constraining factor for SDG in small communities. There has been no report on the use of SDG as a commercial wastewater collection option.
- Corrosion has been a problem in some SDG systems in the United States. Noncorrosive materials must be incorporated in the design.
- Desludging interceptor tanks and disposing of collected septage are probably the most complex aspects of the SDG system. This should be carried out by the local authorities. Contracting the private sector

to carry out these tasks is an option, provided the authorities have enforceable power for hygiene control.

- The most common problem has been related to odors. Many early systems utilized an onlot balancing tank that promoted stripping of $H_2S$ from the interceptor (septic) tank effluent. Other odor problems have been due to inadequate house ventilation systems and mainline manholes or venting structures. In all cases, appropriate engineering can control these problems.

### 5.9.4 Design Criteria

Peak flows are based on the formula $Q = 20 + 0.5D$, where Q is gpm and D is the number of dwelling units served by the system. A determination of peak flows is used for design instead of actual flow data. Each segment of the sewer is analyzed by the Hazen–Williams or Manning equations. Roughness coefficients of 120 to 140 (H–W) and 0.013 (M) are common. No minimum velocity is required and PVC pipe (SDR 35) is commonly employed for all gravity segments. But stronger pipe (e.g., SDR 21) may be dictated where several STEP units feed the system. Also, check valves may be used in flooded or other sections on service laterals where backup from the main is possible (surcharging).

All components must be corrosion-resistant and all discharges (e.g., to a conventional gravity interception or treatment facility) must be made through drop inlets below the liquid level to minimize odors. The system is ventilated through service-connection house vent stacks. Other atmospheric openings should be directed to soil beds for odor control, unless they are located away from the populace.

Interception tanks are generally sized in accordance with local septic tank codes. STEP units employed for below-grade services are covered under pressure sewers. It is incumbent on the engineer to ensure that onlot infiltration and inflow (I/I) be eliminated through proper testing of building sewers and pre-installation testing of interception tanks.

Mainline cleanouts are generally spaced at 120 to 300 m (400 to 1,000 ft) apart. Septic (interceptor) tank effluent is generally assumed to contain 100 to 150 mg/L $BOD_5$ and 50 to 75 mg/L SS. Treatment is normally by stabilization pond or by subsurface infiltration.

### 5.9.5 Capital Costs

The installed costs of the collector mains and laterals and the interceptor tanks constitute more than 50 percent of total construction cost. Average unit costs for 12 projects (adjusted to January 1991) were: 10 cm (4 in.) mainline, \$12.19/ft (\$3.71/m); cleanouts, \$290 each; service connections, \$9.08/ft (\$2.76/m); and 440 L (1,000 gal) interceptor tanks, \$1,315. The average cost per connection was \$5,353 (adjusted to January 1991).

### 5.9.6 O&M Requirements/Costs

The major O&M requirement for SDG systems is the pumping of interceptor tanks, usually at 3 to 5 yr intervals. Other O&M activities include line repairs from excavation damage, supervision of new connections, and inspection and repair of mechanical components. Most SDG system users pay \$10 to 20/month for management, including O&M and administrative costs.

### 5.9.7 Construction Issues

As with all alternative sewer systems, onlot construction is a major part of the project. All sites should be videotaped prior to and after construction to minimize claims from homeowners. Onlot facility plans should be developed at this time and used throughout the project by maintenance personnel. Testing of all piping installed should be performed, and as-built drawings of the installation should be created and submitted to the local authority. Where possible, trenching machines should be considered for substantial (≈50 percent) cost savings. Granular backfill is preferred, as are rubber-ring joints, given the nature of PVC pipe. Magnetic tape above the installed pipe is considered extremely cost-effective to prevent future damage. In some cases, the authority has included the building sewer in the project to ensure I/I control. Easements are required for O&M tasks and usually are in the form of a wide strip centered on the service lateral and interceptor tank.

### 5.9.8 Monitoring

Some management schemes involve biannual tank inspection and pumping only when needed. Most have merely dictated a pumping schedule (e.g., 3 to 5 yr for residential users and every year for commercial users). Otherwise, no monitoring plan is typically established.

### 5.9.9 Residuals

SDG systems, like STEP systems, require interceptor tank pumping. Septage treatment and disposal are described elsewhere.

## 5.10 Vacuum Sewers

### 5.10.1 Technology Description

A vacuum sewer system has three major subsystems: the central collection station, the collection network, and the onsite facilities. Vacuum pressure is generated at the central collection station and is transmitted by the collection network throughout the area to be served. Wastewater from conventional plumbing fixtures flows by gravity to an onsite holding tank. When about 38 to 57 L (10 to 15 gal) of wastewater has been collected, the vacuum interface valve opens for a few (3 to 30) seconds allowing the wastewater and a volume of air to be sucked through the service pipe and into the main. The difference between the atmospheric pressure behind the wastewater and the vacuum ahead provides the primary propulsive force. The fact that both air and wastewater flow simultaneously produces high velocities that prevent blockages under normal operating conditions. Following the valve closure, the system returns to equilibrium and the wastewater reaches the central collection tank, which is under vacuum. When the wastewater in that tank reaches a certain level, a conventional nonclog wastewater pump discharges it through a force main to a treatment plant or gravity interceptor.

The vacuum interface valve is the unique component of a vacuum sewer system. It operates automatically using pneumatic controls; thus, the onsite facilities do not use any electricity. The valve is placed in a valve pit that is usually buried above the holding tank. Plastic pipe is used throughout a vacuum sewer system. The gravity flow house sewer is usually 10 cm (4 in.) pipe. It normally incorporates an external vent to admit air when the valve cycles, thus preventing the house plumbing traps from being sucked dry. Typical service laterals are 7.6 cm (3 in.) pipe, and mains range from 10 to 25 cm (4 to 10 in.) depending on the flow and layout. Joints are either solvent-welded or (preferably) vacuum-certified rubber-ring type.

The profile of the collection network makes use of the limited ability of vacuum propulsion to flow upward in order to avoid excessive excavation. Where the ground slopes in the flow direction more than 0.2 percent, the pipe parallels the ground. Otherwise the pipe is laid with a downward slope of 0.2 percent until the depth becomes excessive. When this occurs, a lift formed by two 45 degree elbows and a short length of pipe is inserted to gain elevation. The typical lift raises the pipe by 0.6 m (2 ft) or less. Division valves are usually placed at main junctions and at 450 m (1,500 ft) intervals to facilitate troubleshooting and repairs. Service lines or tributary mains always join the continuing main from above through a wye connection.

Several mains may be served by a single collection station. Each main is connected directly to the collection tank through a division valve. Air usually flows from the collection tank through a vacuum reserve tank to the vacuum pumps, which discharge to the atmosphere. Dual vacuum pumps are provided to improve reliability. Both liquid ring and sliding vane vacuum pumps have been used. Automatic controls cycle the vacuum pumps alternately to maintain the vacuum in the desired range, usually 5.5 to 7.0 m (18 to 23 ft) of water. A backup diesel-generator set is used to maintain service during electrical outages. An autodialing telephone alarm is provided to summon the operator in case of malfunctions.

Detailed design recommendations differ among manufacturers and engineers. A single interface valve may serve several houses, a school, or a small business area. A few systems use vacuum toilets, which require about 1 to 2 L (0.25 to 0.5 gal)/flush, contain their own interface valves, and have their own vacuum service lines. Cleanouts are provided as access to mains.

The vacuum reserve volume may be provided in the collection tank rather than in a separate vacuum reserve tank. One manufacturer offers an ejector-type vacuum pump in which wastewater from the collection tank is recirculated by a centrifugal pump as the primary fluid in a multiphase jet pump. Another manufacturer provides a factory-assembled, skid-mounted central collection station that can handle design sewage flows of up to 10 L/s (150 gpm). In addition to residential applications, vacuum plumbing is used in office buildings, hospitals, factories, and marinas.

### 5.10.2 Applicability and Status

On January 1, 1990, there were 42 residential vacuum sewer systems operating in 12 states, including Alaska and Florida. These systems served more than 50,000 persons using 100 central collection stations, more than 10,000 vacuum interface valves, and about 160 vacuum toilets. The first of these systems has operated since 1970. All but three systems use the same brand of interface valves.

Vacuum sewers are most likely to be cost-effective when excavation costs are high, population densities are moderate to high (but isolated), and the topography is flat to moderately rolling. They are well suited for combined sewer separation projects in finite urban areas where prohibitive costs of construction must be minimized. Other factors favoring vacuum sewers are the need for water conservation and the need to minimize the risk of sewage spills.

### 5.10.3 Advantages/Disadvantages

Key advantages of vacuum sewers include:

- Low cost of mains
- No electricity required at service locations
- Shallow depth of mains
- Single valve can serve several homes (sources)
- With backup generator, complete independence from local power outages
- Well suited for dense but isolated housing development

Key disadvantages of this type of system include:

- Requires large (>75 generally) numbers of homes for cost-effectiveness
- Somewhat more mechanically sophisticated than other alternative systems
- Requires fast response to malfunctions and greater O&M skill level than other alternative systems
- Slightly more difficult to install main lines than other alternatives

### 5.10.4 Design Criteria

Although there are no universally accepted criteria, the following are widely used:

The maximum capacity of a 7.6 cm (3 in.) interface valve is 2 L/s (30 gpm). The minimum vacuum head needed to operate an interface valve is 1.5 m (5 ft) of water.

Vacuum sewer design rules have been developed largely by studying operating systems. Important design parameters are presented in Tables 5-12 and 5-13.

Table 5-14 shows at what length the 0.2 percent slope will govern vs. the percentage of pipe diameter for the slopes between lifts.

The AIRVAC company has developed a table recommending maximum design flows for each pipe size (Table 5-15).

The maximum number of homes served for various pipe sizes is presented in Table 5-16.

The sum of frictional and lift losses should not exceed about 4 m (13 ft) of water. Frictional losses may be estimated using a modified Hazen-Williams formula. The recommended height of a lift is 30 cm (1 ft) in 10 cm (4 in.) pipe and 46 to 70 cm (1.5 to 2.3 ft) in larger pipes. The loss due to a lift is taken as the invert to invert rise less the internal pipe diameter.

**Table 5-12. Main Line Design Parameters**

| | |
|---|---|
| Minimum distance between lifts | 20 ft |
| Minimum distance of 0.2 percent slope prior to a series of lifts | 50 ft |
| Minimum distance between top of lift and any service lateral | 6 ft |
| Minimum slope | 0.2% |

**Table 5-13. Guidelines for Determining Line Slopes[a]**

| Line Size | Use Largest of: |
|---|---|
| 4 in. Mains | – 0.2%<br>– Ground slope<br>– 80% of pipe dia. (between lifts only) |
| 6 in. Mains | – 0.2%<br>– Ground slope<br>– 40% of pipe dia. (between lifts only) |

[a]Assuming minimum cover at top of slope.

**Table 5-14. Governing Distances for Slopes Between Lifts**

| Pipe Diameter (in.) | Distance (ft) | Governing Factor |
|---|---|---|
| 4 | <135 | 80% of pipe diameter |
| 4 | >135 | 0.2% slope |
| >6 | <100 | 40% of pipe diameter |
| >6 | >100 | 0.2% slope |

**Table 5-15. Maximum Flow for Various Pipe Sizes[a]**

| Pipe Diameter (in.) | Maximum Flow (gpm) |
|---|---|
| 4 | 55 |
| 6 | 150 |
| 8 | 305 |
| 10 | 545 |

[a]BMCI, 1989.

**Table 5-16. Maximum Number of Homes Served for Various Pipe Sizes[a]**

| Pipe Diameter (in.) | Homes Served |
|---|---|
| 4 | 70[b] |
| 6 | 260 |
| 8 | 570 |
| 10 | 1,050 |

[a]BMCI, 1989.
[b]The recommended maximum length of any 4-in run is 2,000 ft, which may limit the amount of homes served to a value less than 70.

Use dual vacuum pumps; size each to handle airflow at design conditions. Use dual sewage pumps; size each to handle design flow. The collection tank volume is at least three times the working volume. Choose the working volume that allows a sewage pump to start every 15 minutes at design flow. A 1,500 L (400 gal) vacuum reserve tank is normally used. The vacuum pump run time should be 1 to 3 minutes.

### 5.10.5 Environmental Impact

Construction impacts may be much less than conventional sewers because of reduced excavation. Risk of sewage spills is minimal since pipes are under vacuum. Aeration of sewage in mains nearly eliminates odor problems.

### 5.10.6 Capital Cost Sensitivity

Costs are highly site-specific. The following data are generalized estimates based on a 1989 telephone survey concerning 32 out of 42 U.S. vacuum systems, on bid tabulations and information from manufacturers and design engineers. All costs are in December 1989 dollars (ENR Construction Cost Index = 4679). On the basis of data from 17 systems, the total construction cost of a vacuum sewer system may range from $7,000 to $18,000/valve. Note that one valve may serve more than one house. A more detailed estimate can be based on the following typical installed unit costs, but wide variations from these values are to be expected.

| | |
|---|---|
| 3 in. interface valve, pit, cover | $2,000-2,300 each |
| 4 in. auxiliary vent | 50–60 each |
| 4 in. gravity flow house sewer | 5/ft |
| 3 in. vacuum service pipe | 7/ft |
| 4 in. vacuum main | 8–11/ft |

| | |
|---|---|
| 6 in. vacuum main | 11–14/ft |
| 8 in. vacuum main | 14–17/ft |
| 10 in. vacuum main | 19/ft |
| 4 in. division valve | 350 each |
| 6 in. division valve | 400–500 each |
| 8 in. division valve | 550–700 each |
| 10 in. division valve | 1,000 each |
| 4 in. cleanout | 150 each |
| 6 in. cleanout | 180 each |
| Gauge taps | 50 each |
| Lifts | 50 each |
| Cycle counter | 125 each |

Table 5-17 gives average installed prices for both custom-designed stations as well as for package stations. The prices include the equipment (including the generator for all stations), station piping, electrical, excavation, site restoration, and labor.

### 5.10.7 O&M Requirements/Cost Range

Power consumption varies with design flow, length of mains, lift, and quality of maintenance. On the basis of records from six systems, annual power consumption ranges from 150 to 600 kWh/valve. A value of 200 kWh/valve/yr is recommended for preliminary estimates. A study of six systems yielded a MTBSC from 1 to 22 years. A planning value of 6 to 8 years is reasonable for new vacuum valve designs.

Preliminary O&M cost estimates can be derived with the following formula:

$$C = (2{,}430 \times NS) + (205 \times LR \times NS) + (0.5 \times LR \times NDV) + (5.1 \times NIV) + (1.2 \times LR \times NIV) + (500 \times NIV \times ER)$$

Where:

C = annual O&M cost in December 1989 dollars

NS = number of central collection stations

LR = labor rate including fringe benefits and overhead in December 1989$/hr

NDV = number of division valves

NIV = number of vacuum interface valves

ER = electric power rate in December 1989$/kWh

Major components of a vacuum sewer system are presented in Figure 5-7.

### 5.10.8 Construction Issues

Vacuum system manufacturers generally include assistance to engineers and communities during the construction phase of the project. One key difficulty arises due to the common practice of using two separate phases of in-

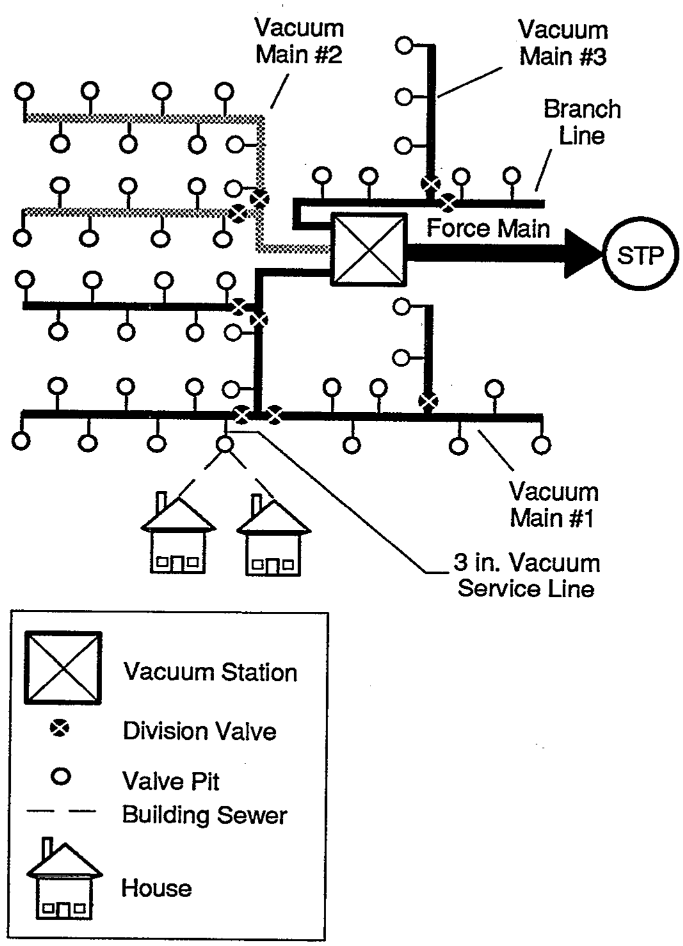



**Figure 5-7. Major Components of a Vacuum Sewer System.**

**Table 5-17. Average Installed Cost for Vacuum Station (Mid-1990)**

| Item | Number of Customers ($) | Equipment Cost[a] ($) | Building Cost ($) | Installed Cost ($) | Total Cost ($) |
|---|---|---|---|---|---|
| Package station | 10–25 | 50,000 | 25,000 | 20,000 | 95,000 |
| Package station | 25–50 | 75,000 | 30,000 | 25,000 | 130,000 |
| Package station | 50–150 | 90,000 | 40,000 | 30,000 | 160,000 |
| Custom station | 100–300 | 120,000 | 50,000 | 40,000 | 210,000 |
| Custom station | 300–500 | 140,000 | 60,000 | 50,000 | 250,000 |
| Custom station | >500 | 170,000 | 75,000 | 75,000 | 320,000 |

[a]Includes generator.

stalling onsite services, with subsequent main-line installation and hookup. Careful coordination of these activities is necessary. Vacuum testing of all sewer segments must be performed daily during construction. During construction of a vacuum sewer system there is potentially less noise impact, fewer problems with fugitive dust, and less erosion than with conventional sewers because of smaller equipment requirements.

### 5.10.9 Monitoring

In addition to operating tasks discussed above, cycle counter readings and spot checks of vacuum pressure at various locations in the piping network should be included in any monitoring program to anticipate potential problems.

## 5.11 Mechanical Systems for Wastewater Treatment

Mechanical systems utilize a combination of biological and physical processes for the treatment of wastewater, employing tanks, pumps, blowers, rotating mechanisms, and/or other mechanical components as part of the overall wastewater treatment system. Mechanical systems are frequently used for medium to large municipal wastewater treatment plants, but also have been widely applied for the treatment of wastewater associated with small, sewered communities or clusters of residential housing and commercial establishments. For very low flows (<2 L/s (<50,000 gpd)), preengineered "package plants" are the mechanical systems normally used. Effluent from mechanical systems can either be discharged to surface water or applied to the land.

Virtually all mechanical systems employ suspended-growth or attached-growth (fixed-film) biological processes or a combination of the two. In suspended-growth systems, microorganisms responsible for the breakdown of the organic matter are suspended in liquid by mixing. In attached-growth systems, microorganisms become attached to an inert medium, such as rock or plastic. Oxygen is provided mechanically in suspended-growth systems and naturally in attached-growth systems. The biological process is followed by a clarification step to allow separation of the biological solids from the treated wastewater. Suspended-growth treatment approaches discussed in this section include the sequencing batch reactor, oxidation ditch, and extended-aeration system. The trickling filter process is discussed as an example of an attached-growth system.

Mechanical systems have potential advantages over some "natural" alternatives in that they can provide a high-quality effluent, and they are generally very land-efficient. Disadvantages include the need for close skilled-operator supervision, high maintenance requirements, and high power consumption compared to natural wastewater treatment systems.

## 5.12 Extended-Aeration Activated Sludge

### 5.12.1 Technology Description

The extended-aeration process is a widely used modification of the conventional suspended-growth, activated-sludge process characterized by low loading rates and long hydraulic and solids retention times. Hydraulic retention times are typically 24 hours, with solids retention times of 20 to 40 days. Because of the low BOD loading, the process operates in the endogenous respiration phase of the microbial growth cycle, resulting in partial oxidation of biological solids. Some extended-aeration processes operate in a completely mixed regime, with the contents of the aeration basin being nearly homogeneous. Some technologies are designed to operate in a plug-flow mode. The high solids retention times associated with the process promote nitrification. In a well-operated facility, BOD and SS removals can be expected to range from 85 to 95 percent. A properly designed and operated extended-aeration facility can be expected to produce an effluent with BOD and SS levels less than 30 mg/L 90 percent of the time and less than 20 mg/L 50 percent of the time. Because of the long aeration times, biodegradable toxic compounds are likely to be removed. A flow diagram of the process is shown in Figure 5-8.

An extended-aeration process typically consists of coarse screening or comminution, activated-sludge aeration using course air diffusers or mechanical aerators, secondary clarification using surface skimming and return sludge pumping, disinfection often with chlorine storage and feed facilities, and transport to a contact basin. Sludge is typically wasted to an aerobic holding or aerobic stabilization compartment. Sludge disposal varies widely. Primary clarification is rarely used.

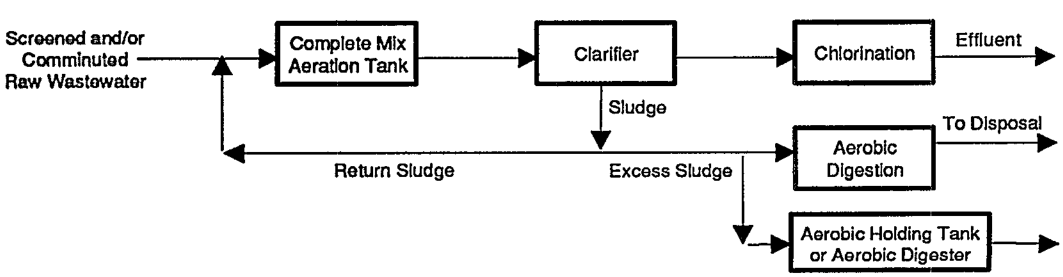



**Figure 5-8. Schematic of an Extended-Aeration Process.**

### 5.12.2 Applicability and Status

Extended-aeration processes are used widely throughout the United States for municipal wastewater flows less than 2 L/s (50,000 gpd). Most of these are preengineered package plants that serve residential subdivisions, small clusters of homes, commercial or institutional establishments, or individual homes. Such package plants are largely preassembled, allowing rapid installation with a minimum of site preparation. For this reason, package plants are sometimes used for temporary or emergency applications. Package plants generally utilize steel tankage, but precast concrete is also used. Preengineered extended-aeration systems are available for flows up to about 40 L/s (1 mgd). However, the extended-aeration process has been used to treat wastewater flows of 220 L/s (5 mgd) and greater.

Most package plants discharge to surface waters, although they have been utilized for treatment prior to land application or even subsurface disposal. However, the mechanical complexity of such plants is generally not compatible with low-maintenance, land-based disposal options.

Extended-aeration processes and package plants are considered a fully developed technology, having been in wide use since the 1950s.

### 5.12.3 Advantages/Disadvantages

Key advantages of the extended-aeration process include:

- Lowest sludge production of any activated-sludge process
- High-quality effluent achievable
- Preengineered package plants quickly installed with minimal site preparation
- Favorable reliability with sufficient operator attention
- Nitrification likely at wastewater temperatures >15°C
- Relatively minimal land requirements
- Relatively low initial cost
- Can handle moderate-shock hydraulic loadings with minimal problems

Key disadvantages of this type of system include:

- High power consumption and energy cost compared to land-based or natural systems
- High O&M requirements compared to land-based or natural systems; skilled operator necessary
- Susceptible to excursions in effluent SS and associated BOD due to high flow variations and operator inattention
- Potential freezing problems in cold climates
- Possibility of poor settleability of mixed liquor suspended solids (MLSS) due to formation of "pinpoint" floc
- Potential for rising sludge due to dentrification in final clarifier in warmer months
- Blower noise and sludge handling odor potential
- Preengineered plants may require additional components or modification to meet specified effluent limitations

### 5.12.4 Design Criteria

Table 5-18 summarizes the design criteria for the extended-aeration process. For package plants, components such as aeration basins and clarifiers are preengineered, and system selection for domestic wastewater applications typically is based solely on flow. The size of the system selected should be conservative to account for peak flow conditions. Table 5-19 provides typical sizing of unit processes at average design flows of 0.4 to 4.0 L/s (0.01 to 0.1 mgd).

For engineered plants using the extended-aeration process, final clarifier design should be conservative to account for high MLSS and poor settleability of the biological solids. Since the process lacks primary clarifi-

**Table 5-18. Summary of Design Criteria for Extended-Aeration Process**

| Parameter | Range |
|---|---|
| Volumetric loading (lb $BOD_5$/d/1,000 cu ft) | 8–15 |
| MLSS (mg/L) | 2,500–6,000 |
| F/M (lb $BOD_5$/d/lb MLVSS) | 0.05–0.15 |
| Aeration detention time (hr) (based on daily flow) | 18–36 |
| Air requirements (scf/lb $BOD_5$ applied lb $O_2$/lb $BOD_5$ applied) | 3,000–4,000<br>2.0–2.5[a] |
| Solids retention time (days) | 20–40 |
| Recycle ratio (R) | 0.75–1.5 |
| Volatile fraction of MLSS | 0.6–0.7 |

[a]Based on 1.5 lb $O_2$/lb $BOD_5$ removed + 4.6 lb $O_2$/lb $NH_4$-N removed.

**Table 5-19. Typical Component Sizing for Extended-Aeration Plants**

| Unit Process | At Average Design Flow (mgd) | | |
|---|---|---|---|
| | **0.01** | 0.05 | **0.10** |
| Raw sewage pumping (mgd) | 0.04 | 0.20 | 0.40 |
| Aeration basin volume (cu ft) | 1,330 | 6,690 | 13,400 |
| Secondary clarifier area (sq ft) | 40 | 200 | 400 |
| Chlorinator capacity (lb/d) | 10 | 25 | 50 |
| Chlorine contact chamber volume (cu ft) | 120 | 560 | 1,200 |
| Drying bed area (sq ft) | 400 | 1,000 | 2,000 |
| Site area (ac) | 0.5 | 0.7 | 1.0 |

cation and due to the potential for rising sludge, secondary clarifiers must be equipped with surface skimming devices to remove grease and floating solids. Surface aerators are not recommended for extended-aeration processes in cold climates because of potential for freezing problems.

Design life of a wastewater treatment plant using the extended-aeration process can be expected to vary depending on whether it is a preengineered system or one employing cast-in-place concrete tanks. Steel package plants may have an effective service life as short as 10 years if the maintenance program is inadequate, but as long as 20 years with regular and effective maintenance. Larger, custom-engineered systems may have a design life of 20 years or more. Mechanical equipment used in either type is likely to have a service life of 5 to 15 years, depending on the type and quality of the equipment.

Because of concerns for blower noise and odors associated with handling of residuals, adequate buffer zones are necessary between the facility and residential areas. This distance should be at least 200 ft. In addition, adequate fencing is necessary to restrict access to the site.

Small mechanical plants typically are unattended at night. For this reason, plants should be designed with an alarm system to alert emergency personnel, such as a police or fire department dispatcher.

### 5.12.5 Capital Cost Sensitivity

Package plants are relatively economical since site preparation and engineering costs are minimized. Some units are installed above ground on a concrete slab. In general, package plant costs are sensitive to flow, with larger units required to adequately handle peak flow conditions.

Costs of engineered systems or package plants may be affected by site conditions such as geology, climate, and surrounding aesthetics. Larger treatment plants may require buildings to house operations and control centers, laboratories, and sludge dewatering facilities. In general, preengineered package systems are less costly than custom-designed systems employing cast-in-place concrete tankage.

### 5.12.6 O&M Requirements

O&M requirements for an extended-aeration facility are high because of the need for skilled and regular operator supervision to ensure performance, and the general mechanical complexity of the components. O&M requirements are summarized in Table 5-20. An inventory of spare parts must be maintained in case of equipment failure, which eventually will occur. Inventories should include pump packing and seals, bearings, motors, diffusers, chlorinator components, and flow metering equipment. In many cases, lack of spare parts and the long lead times to procure parts and equipment have resulted in long periods of poor performance.

**Table 5-20. O&M Requirements of Extended-Aeration Facilities**

**Operation**

- Effluent quality monitoring required by NPDES permit
- Operation assessment analyses (e.g., MLSS, DO, sludge blanket, settleability)
- Regular cleaning of screens, weirs, skimmer mechanisms, diffusers, and other components
- Regular adjustment of sludge return rate and air injection rate
- Regular wasting of solids
- Sludge dewatering and disposal
- Control of disinfectant dosage
- Administration and recordkeeping

**Maintenance**

- Blowers or mechanical aerators
- Influent and return sludge pumps
- Electrical equipment and instrumentation
- Mechanical dewatering equipment
- Laboratory equipment such as pH, DO meters, chemicals
- Disinfection system

Many small communities lack the skilled personnel necessary to operate properly and maintain a mechanical facility of this type. Thus an extended-aeration plant may not be the system of choice for flows less than 2 L/s (50,000 gpd). If such a system already exists, the municipality may wish to procure, under contract, the services of an individual or firm qualified to operate and maintain the facility, relieving the municipality of this burden.

Energy consumption for an extended-aeration facility is high relative to other mechanical treatment alternatives, primarily due to mixing and the high oxygen requirement associated with the long solids retention time. Energy is consumed by aeration of the activated sludge basin (as well as the aerated sludge holding tank or aerobic digester, if applicable), influent pumping (if applicable), return sludge pumping, mechanical dewatering (if applicable), building heating, cooling, and lighting, and by operating other miscellaneous processes such as clarification and disinfection. However, the aeration/mixing step accounts for the overwhelming majority of power demand. Electrical energy requirements may be expected to be approximately 15,000 kWh/yr for a 0.4 L/s (10,000 gpd) facility, 40,000 kWh/yr for a 2 L/s (50,000 gpd) plant, and 60,000 kWh/yr for a 4 L/s (100,000 gpd) facility.

### 5.12.7 Monitoring

Monitoring consists of sampling and conducting analyses as required by the National Pollutant Discharge Elimination System (NPDES) or other permits, and as is necessary to ensure optimum plant performance. NPDES requirements will vary substantially depending on the size of the facility and the characteristics of the receiving waters. NPDES monitoring requirements may include influent flow, influent BOD and SS, and effluent BOD, SS, chlorine residual, and nutrients (i.e., ammonia, phosphorus). Sampling frequency may range from daily to bimonthly. Operational monitoring generally includes MLSS, aeration basin, dissolved oxygen (DO), pH, settleability, alkalinity, effluent chlorine residual, influent flow, return sludge flow, and sludge waste rate.

### 5.12.8 Residuals

Residuals generated from an extended-aeration facility include screenings and waste sludge. Grit is also generated if the plant is equipped with a separate grit chamber. Grit and screenings must be removed and disposed of promptly because of their putrescible nature. If no separate grit chamber is provided, grit accumulates in the aeration tank, eventually requiring draining and cleaning.

Due to the long solids-retention time, waste sludge production is somewhat less than for conventional activated sludge. Sludge may be wasted to a sludge holding tank or stabilization process such as an aerobic digester and removed from the plant as a liquid, or may be subsequently dewatered by sand bed or other technologies and hauled away as a solid. Options for the handling of waste sludge from mechanical wastewater treatment systems are discussed in greater detail later in this chapter.

## 5.13 Trickling Filter and Modifications

### 5.13.1 Technology Description

The trickling filter process is an attached-growth process in which wastewater that has undergone primary clarification is distributed periodically over an inert media such as rock or plastic. Organisms contained in biological slimes attached to this media are responsible for the breakdown of organic matter. Periodically, portions of this slime layer "slough off" the inert media and are removed by gravity, settling in a secondary clarifier. A typical trickling filter process consists of screening, grit removal, primary clarification, biological treatment with the trickling filter, secondary clarification, and disinfection.

Conventional, standard-rate trickling filters were at one time one of the most popular methods of wastewater treatment, with many such facilities constructed from 1930 to 1950. The conventional trickling filter process is simple and easy to operate. The early plants used stones as the media for biological growth and were designed for discharge of the secondary sludge or "humus" back to the primary clarifier for co-settling. The combined primary and secondary sludge was typically dewatered on sand drying beds, particularly at the smaller facilities.

The conventional trickling filter process is limited with respect to performance. Many older facilities are unable to meet secondary treatment standards (30 mg/L BOD; 30 mg/L SS) on a year-round basis. Finely divided particulate matter with poor settling properties, characteristic of the trickling filter process, often contributes to high SS levels in the effluent. Attention to the design of the final clarifier can overcome this problem. In 1984, EPA relaxed performance standards for existing trickling filter plants, allowing discharge of an effluent containing no more than 45 mg/L SS and 45 mg/L BOD. A conventional existing trickling filter plant would be expected to produce an effluent with less than 40 mg/L BOD and SS 90 percent of time, and less than 30 mg/L BOD and SS 50 percent of the time.

Several modifications to the conventional standard rate of trickling have been developed to overcome the performance limitations of the original process. These include "high-rate" processes and the trickling filter-solids contact (TFSC) process. High-rate trickling filters typically employ higher hydraulic and organic loading rates to the trickling filters and continuous recirculation of effluent through the filter at 50 to 300 percent of the influent flow. Many high-rate systems employ plastic media consisting of either random-dump packing or prefabricated plastic modules that are set in place. Considerable development has occurred in the manufacture of such modular plastic media for trickling filter applications.

The TFSC process is a relatively innovative approach to the trickling filter process. In this process, discharge from the trickling filter is "contacted" with secondary return sludge in an aerated, short-detention-time tank. This allows flocculation and agglomeration of the trickling filter "fines," improving SS and associated BOD removal in the final clarifier. In some cases, round, secondary clarifiers equipped with a central flocculation well are used to improve flocculation and settleability of the biological solids. The process is capable of consistently meeting secondary and some advanced secondary treatment standards.

Another modification is the coupled trickling filter-activated-sludge approach, which employs both attached-growth and suspended-growth biological processes. In this variation, discharge from the trickling filter enters an activated-sludge aeration basin for additional carbonaceous removal and nitrification before final clarification. This modification may be used where ammonia removal (nitrification) is required. The process is particularly stable and capable of consistently achieving a high-quality effluent having BOD and SS levels less than 20 mg/L.

Flow diagrams of the various TFSC processes are shown in Figure 5-9.

### 5.13.2 Applicability and Status

Because of the inability to meet secondary treatment standards consistently, conventional trickling filter facilities are no longer being built. However, many such plants are still in operation, having been constructed to treat wastewater from both small and large communities. Some of these have been abandoned, while others have been upgraded to the TFSC process to improve performance.

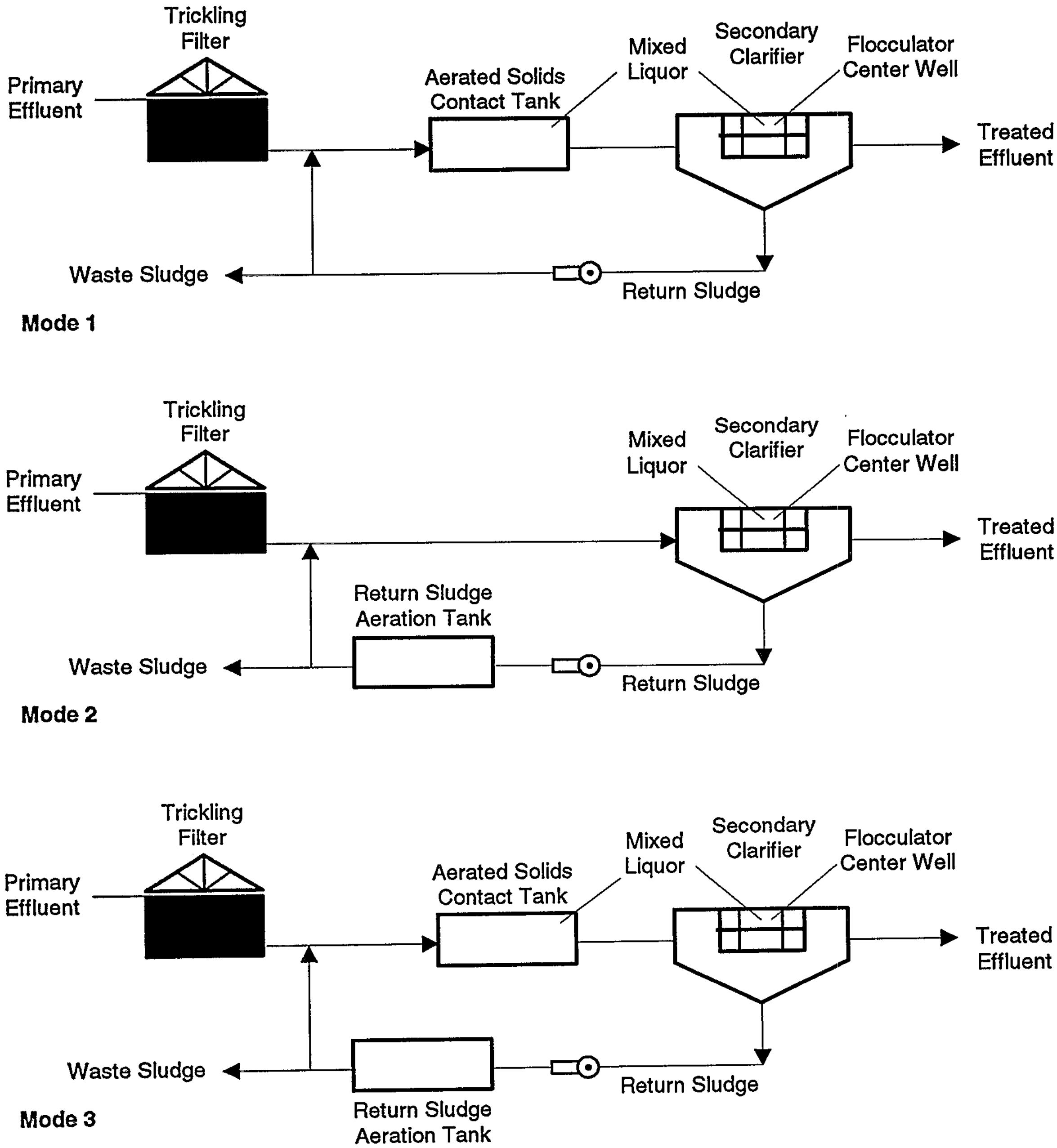



**Figure 5-9. Schematics of Trickling Filter–Solids Contact Processes.**

High-rate trickling filters are also rarely used today for secondary treatment because of these same performance limitations. However they are applicable for use as "roughing" filters ahead of activated-sludge processes to reduce the organic loading to the aeration basin. Such roughing filters are particularly applicable for reduction of high organic loadings resulting from industrial discharges.

In 1991, it was estimated that over 50 TFSC plants were in design construction or operation. Eleven plants were known to be in operation. Although the process was only recently developed (late 1970s), its relatively widespread application allows consideration as a fully developed technology. The TFSC process is clearly appropriate as a mechanical wastewater system for small, sewered communities, both for new systems and as a retrofit to existing trickling filter plants. Features that make TFSC well suited for such applications include its relative simplicity, consistently favorable performance, and low O&M requirements compared to other mechanical treatment alternatives.

### 5.13.3 Advantages/Disadvantages

Advantages of the TFSC process include:

- Applicable for new facilities or upgrading existing trickling filter plants
- Capable of consistently achieving very high-quality effluent (<20 mg/L BOD & SS)
- Relatively simple process
- Low cost and reliable upgrading technique for trickling filters
- Can be designed to provide nitrification

Key disadvantages of this process include:

- Primary clarification required
- Pumping required to douse trickling filter
- Potential for nuisance odors from primary clarifiers, trickling filter, sludge handling
- Moderate O&M requirements; skilled operator necessary.

### 5.13.4 Design Criteria

Available design criteria for the TFSC process are summarized in Table 5-21. Definitive design criteria have not been established, and as a result, a wide variation exists in some design parameters. Table 5-22 provides typical sizing criteria for components of the TFSC process.

There are several design considerations that must be addressed for a successful operation. It is essential to establish conditions in the aerated solids' contact tank and final clarifier that promote favorable flocculation. For this reason, fine bubble diffusers are typically used in the solids contact tank to provide sufficient dissolved oxygen and gentle mixing without excessive turbulence that might shear the floc. Channels or pipes leading to the final clarifier should not impart turbulence to the liquid. Some of the early designs for the process employed flocculating center wells that could be gently mixed using mechanical paddles or air spargers. However, it was found that hydraulic conditions in the center well were such that addition of mixing devices was not necessary.

**Table 5-21. Summary of Available Design Criteria for the TFSC Process**

| Parameter | Trickling Filters | |
|---|---|---|
| | Low rate | High rate |
| Hydraulic loading (mgd/ac) | 1–4 | 10–40 |
| BOD loading (lb/1,000 cu ft/d) | 5–25 | 30–60 |
| Recirculation ratio | 0 | 0.5–3 |
| Media depth (ft) | 6–8 | 4 – 6 |
| | Solids Contact/ Aeration Tank | |
| | Low rate | High rate |
| Detention time (hr) | 0.3–1.5 | 6–12 |
| Solids retention time (days) | 0.5–2.0 | >6 |
| MLSS (mg/L) | 700–3,00 | 1,500–3,500 |
| DO (mg/L) | 1.5–3.5 | 2.0–4.0 |
| | Secondary Clarifier | |
| Overflow rate based on total area (gpd/sq ft) | 300–500 | |
| Sidewater depth (ft) | 15–18 | |
| Flocculator centerwell (% of total area) | 5-16 | |

**Table 5-22. Typical Component Sizing for TFSC Plants**

| Unit Process | At Average Design Flow (mgd) 0.1 | 0.5 | 1.0 |
|---|---|---|---|
| Primary clarifier area (sq ft) | 170 | 850 | 1,700 |
| Primary effluent pumping (mgd) | 0.4 | 2.0 | 4.0 |
| Trickling filter volume (cu ft) | 7,150 | 35,750 | 71,500 |
| Recirculation pumping (mgd) (for high-rate systems) | 0.3 | 1.5 | 3.0 |
| Solids contact basin volume (cu ft) (30-min contact time w/33% return rate) | 370 | 1,850 | 3,700 |
| Secondary clarifier area (sq ft) | 330 | 1,750 | 3,500 |
| Chlorinator capacity (lb/d) | 50 | 250 | 500 |
| Chlorine contact chamber volume (cu ft) | 1,200 | 5,600 | 11,100 |
| Site area (ac) | 1.5 | 3 | 5 |

Design life for a trickling filter plant is 20 years or more. However, mechanical equipment used in the plant is likely to have a service life of 5 to 15 years depending on the type and quality of the equipment.

Trickling filter plants, because of the presence of a primary clarifier, accumulation of biomass on the trickling filter, and the handling of primary sludge, have greater potential for release of objectionable odor than extended-aeration facilities. "Filter flies" may also be a localized nuisance. Buffer distance around the facility should be at least 60 m (200 ft). Fencing must be provided to restrict access.

Mechanical plants serving small communities are typically unattended at night. Plants should be equipped with an alarm system to alert emergency dispatch personnel.

### 5.13.5 Capital Cost Sensitivity

In general, new TFSC processes are more costly than either custom-designed extended-aeration plants or package plants. This is attributable mainly to the additional costs associated with primary clarification and sludge handling. However, use of the TFSC process is probably the most economical means of upgrading an existing trickling filter plant to meet secondary standards. Costs of TFSC plants may be affected by site conditions such as climate, geology, and surrounding aesthetic conditions. Relatively high costs are likely to be associated with sludge dewatering and stabilization.

### 5.13.6 O&M Requirements

O&M requirements for a TFSC facility are relatively high due to the need for skilled and regular operator supervision and the general mechanical complexity of the components. However, performance is less dependent on operator skill and supervision than the extended-aeration process. O&M requirements are summarized in Table 5-23. An inventory of spare parts must be maintained in case of equipment failure, which eventually will occur. Inventories should include pump packaging and seals, bearings, motors, diffusers, chlorinator components, and flow metering equipment. In many cases, lack of spare parts and the long lead times to procure parts and equipment have resulted in long periods of poor performance.

**Table 5-23. O&M Requirements of TFSC Facilities**

**Operations**

- Effluent quality monitoring required by NPDES permit
- Operation assessment analyses (e.g., MLSS, DO, sludge blanket, settleability)
- Regular cleaning of screens, weirs, filter media, skimmer mechanisms, diffusers and other components
- Regular adjustment of trickling filter recirculation rate, sludge return rate and air injection rate
- Regular wasting of solids
- Sludge dewatering, stabilization, and disposal
- Control of disinfectant dosage
- Administration and recordkeeping

**Maintenance**

- Blowers
- Primary effluent and return sludge pumps
- Trickling filter distribution system
- Electrical equipment and instrumentation
- Mechanical dewatering equipment
- Laboratory equipment such as pH, DO meters, chemicals
- Disinfection system

Many small communities lack the skilled personnel necessary to operate properly and maintain a mechanical facility of this type. Thus the TFSC process is unlikely to be the system of choice for flows less than 2 L/s (50,000 gpd). If such a system already exists, the municipality may wish to procure, under contract, the services of an individual or firm qualified to operate and maintain the facility, relieving the municipality of this burden.

Energy consumption for a TFSC facility is higher than a trickling filter plant, but less than an activated sludge facility due to the low power requirement to supply oxygen to the contact tank. This may increase substantially if nitrification is required. Energy is consumed by trickling filter influent and recirculation pumping, aeration of the solids contact or activated-sludge basin, return sludge pumping, mechanical dewatering (if applicable), sludge stabilization, the heating, cooling, and lighting of the building housing the system, and other miscellaneous processes such as clarification and disinfection. Pumping energy may be estimated using the equation:

$$\text{kWh/yr} = 1900 \times (\text{flow, mgd}) \times (\text{discharge head, ft})$$

### 5.13.7 Monitoring

Monitoring consists of sampling and conducting analyses as required by NPDES or other permits, and as is necessary to ensure optimum plant performance. NPDES requirements will vary substantially depending on the size of the facility and the characteristics of the receiving waters. NPDES monitoring requirements may include influent flow, influent BOD and SS, and effluent BOD, SS, chlorine residual, and nutrients (i.e., ammonia, phosphorus). Sampling frequency may range from daily to bimonthly. Operational monitoring generally includes MLSS, aeration basin DO, pH, settleability, alkalinity, effluent chlorine residual, influent flow, trickling filter recirculation flow, return sludge flow, and sludge waste rate.

### 5.13.8 Residuals

Residuals generated from a typical TFSC system include grit and both primary and secondary sludge. Grit and screenings must be removed and disposed of promptly because of their putrescible nature.

Secondary sludge may be blended with primary sludge in the primary tank or pumped to a stabilization process such as an aerobic or anaerobic digester. Sludge may be removed from the plant as a liquid or may be dewatered by sand bed or mechanical devices and hauled away as a solid. Options for the handling of waste sludge from mechanical wastewater treatment systems are discussed in greater detail later in Section 5.16.

## 5.14 Oxidation Ditch

### 5.14.1 Technology Description

The oxidation ditch process is a closed-loop variation of the extended-aeration activated-sludge process. As with extended aeration, the process is characterized by hydraulic retention times of 18 to 30 hours, and solids retention times of 10 to 33 days. The process is highly stable and reliable, and is suitable for relatively small wastewater flows associated with small communities or subdivisions.

An oxidation ditch process typically consists of coarse screening, grit removal, one or more closed-loop aerated channels, secondary clarification, and disinfection. Primary clarification is rarely used. A typical means of aeration is the use of horizontally mounted rotating brush, cage, or disc aerators, which operate at 60 to 110 rpm and provide transfer of oxygen as well as impart velocity to the wastewater. Another common aeration system associated with a proprietary oxidation ditch process is a vertical turbine, nonsparged aerator mounted at 180 degree bends in the channel near the dividing wall. Several novel aeration and mixing devices have also been used with varying degrees of success in regard to oxygen transfer and maintaining solids in suspension. A schematic of the oxidation ditch process is provided in Figure 5-10.

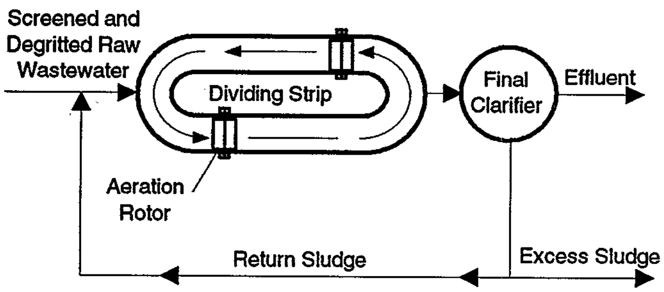



**Figure 5-10. Schematic of an Oxidation Ditch Process.**

Oxidation ditch loops are generally oval in shape. Some larger plants have employed "folded loop" or horseshoe configurations to make efficient use of space. For small facilities, a single channel, 1.2 to 1.8 m (4 to 6 ft) deep with 45 degree sloping sidewalls, is typically used. This configuration allows several options for construction. Often, cast-in-place reinforced concrete is used to form the ditch. However, gunite and asphalt lined ditches have been successfully constructed. Lining is necessary to prevent erosion from the moving wastewater. Experience has shown the cost of gunite lining to be higher than reinforced concrete for this application. Deeper ditches (≥3m) (but smaller in area) with common vertical, concrete walls are also used for larger facilities.

In the 1980s, a number of "in channel" clarifier designs were marketed for oxidation ditch plants. Here a clarification device is placed within the channel to provide a quiescent settling zone, with solids returned back to the ditch through ports or slots at the bottom of the clarifier. These systems eliminate the need for separate final clarifiers and return sludge pumping; however, there are trade-offs: lack of the ability to waste a thickened sludge from a separate clarifier (requiring wasting of a relatively dilute mixed liquor); lack of operational flexibility in adjusting sludge return rates; and the need to increase mixing to overcome additional headloss through the ditch resulting from the restriction of flow around the in-channel clarifier.

The oxidation ditch is capable of reliably producing a very high-quality effluent. A 1978 study by EPA that compared the performance of 29 oxidation ditch plants with competing biological processes demonstrated that the oxidation ditch systems outperformed conventional activated sludge, package plants, trickling filters, and rotating biological contactors. Average annual effluent quality was less than 15 mg/L BOD and SS. A 20-20 effluent was achieved 85 to 90 percent of the time and a 30-30 effluent was achieved 95 percent of the time.

Oxidation ditch plants are capable of substantial nitrogen removal, with 40 to 80 percent nitrogen removal achievable. This favorable performance results from the development of zones of low dissolved oxygen or anoxic conditions between aerators where denitrification can take place. Achieving high nitrogen removal efficiency involves careful placement of aerators and adjustment of variables such as aerator speed, immersion depth, and on/off cycling frequency.

### 5.14.2 Applicability and Status

The oxidation ditch was developed in the Netherlands, with the first plant constructed in 1953 for a community of 400 people. Because of its high performance and reliability it now is used widely throughout the United States. It is well suited for wastewater flows in excess of 0.4 L/s (10,000 gpd). In 1980, over 650 oxidation ditch plants were in use in the United States and Canada. Where a mechanical system is desired that will provide excellent performance, high reliability, and relatively minor operator attention, the oxidation ditch should be considered.

### 5.14.3 Advantages/Disadvantages

Key advantages of the oxidation ditch process include:

- Low sludge production
- Excellent performance
- High reliability
- Nitrogen removal likely
- Relatively low initial cost
- Can be designed for biological phosphorus and nitrogen removal

Key disadvantages of this process include:

- Protection from aerator freezing problems necessary in cold climates
- Relatively high maintenance requirements for aerators
- Potential for rising sludge due to denitrification in final clarifier
- Requires good operator skills and routine monitoring

### 5.14.4 Design Criteria

Table 5-24 summarizes the design criteria for the oxidation ditch process. These criteria would apply to a conventional ditch (extended aeration) designed for carbonaceous BOD removal, without specific considerations for nutrient (i.e., nitrogen, phosphorus) removal. Typical sizing of components of an oxidation ditch plant is summarized in Table 5-25.

Manufacturers of aeration equipment for oxidation ditches have developed design criteria and suggested plant layouts to meet a wide range of performance and site requirements. Some ditches have been designed as a single reactor that operates as both aeration basin and clarifier. The most common oxidation ditch configuration

**Table 5-24. Summary of Design Criteria for the Oxidation Ditch Process**

| Parameter | Range |
|---|---|
| Volumetric loading (lb $BOD_5$/lb/1,000 cu ft) | 10–15 |
| MLSS (mg/L) | 2,100–6,300 |
| F/M (lb $BOD_5$/d/lb MLVSS) | 0.034–0.10 |
| Aeration detention time (hr) (based on average daily flow) | 18–30 |
| Solids retention time (days) | 10–33 |
| Recycle ratio (R) | 0.75–1.5 |
| Volatile fraction of MLSS | 0.6–0.7 |
| Channel velocity (fps) | ≥1.00 |
| Oxygen transfer efficiency for horizontal aerators (lb $O_2$/hp-hr/LF) | 3–5 |

**Table 5-25. Typical Component Sizing for Oxidation Ditch Plants**

| Unit Process | At Average Design Flows (mgd) | | |
|---|---|---|---|
| | 0.1 | 0.5 | 1.0 |
| Aeration basin volume (cu ft) | 13,400 | 66,400 | 134,000 |
| Aerator, installed (hp) | 7.5 | 40 | 60 |
| Secondary clarifier area (sq ft) | 400 | 2,000 | 4,000 |
| Return sludge pumping (mgd) | 0.2 | 1.0 | 1.5 |
| Chlorinator capacity (lb/d) | 50 | 250 | 500 |
| Chlorine contact chamber volume (cu ft) | 1,200 | 6,000 | 11,000 |
| Site area (ac) | 1.0 | 2.5 | 4 |

in the United States is a continuous-flow, single-channel ditch followed by a final clarifier. Another arrangement comprises multiple concentric channels connected by submerged ports. Such plants allow flexibility in operating mode. For example, some four-channel plants are designed to allow operation in the extended-aeration mode, conventional activated sludge-aerobic digester mode, or contact stabilization mode simply by adjustment of gates and valves.

The procedure for design of an oxidation ditch plant is approximately the same as for an extended-aeration activated-sludge plant. Some low-alkalinity wastewaters may require pH adjustment when subjected to long detention times in the aeration basin. Calculation of oxygen requirements should assume that nitrification will occur. Design life of an oxidation ditch plant is 20 years or more, however, mechanical equipment is likely to have a service life of 5 to 15 years depending on the type and quality of the equipment.

### 5.14.5 Capital Cost Sensitivity

Oxidation ditch plants are competitive with other activated sludge processes in the range of 4 to 440 L/s (0.1 to 10.0 mgd) and appear to be particularly cost-effective in the larger size ranges. However, local factors and specific process design can have a major impact on relative construction costs.

Ditch configuration can affect construction costs. For small plants, the shallow ditch with 45 degree sloped sidewalls appears most cost-effective. For plants over 44 L/s (1 mgd), the deep channel with straight sidewalls is cost effective. Ditch construction techniques can also affect capital costs, as discussed elsewhere. As with other mechanical plants, capital costs are influenced by site, geological, and local aesthetic conditions.

### 5.14.6 O&M Requirements

O&M requirements for the oxidation ditch process are summarized in Table 5-26. O&M costs are high relative to natural systems for wastewater treatment. Successful performance requires skilled operators and regular inspection, monitoring, and maintenance.

**Table 5-26. O&M Requirements of Oxidation Ditch Facilities**

**Operations**
- Effluent quality monitoring required by NPDES permit
- Operation assessment analyses (e.g., MLSS, DO, sludge blanket, settleability)
- Regular cleaning of screens, weirs, skimmer mechanisms, tank walls, and other components
- Regular adjustment of sludge return rate and oxygenation rate
- Regular wasting of solids
- Sludge dewatering and disposal
- Control of disinfectant dosage
- Administration and recordkeeping

**Maintenance**
- Mechanical aerators
- Influent and return sludge pumps
- Electrical equipment and instrumentation
- Mechanical dewatering equipment
- Laboratory equipment such as pH, DO meters, chemicals
- Disinfection system

In oxidation ditch plants, aerators and aerator drives account for a major portion of mechanical problems. In an EPA study of oxidation ditch plants, many plants experienced the following aerator-related problems every 2 to 5 years/unit:

- Loss of "teeth" from brush-type aerators
- Bearing problems in gear drives, line shafts, and aerator shafts
- Failure of flexible couplings between line shafts
- Failure of gear reducer and gear reducer shaft seals
- Corrosion of bearings, couplings, and drive units due to continuous wetting from aerator spray

In cold climates, spray from aerators will freeze on equipment, structures, and walkways. Aerators should be covered in moderately cold areas, and also heated in very cold areas.

Many small communities lack the skilled personnel necessary to operate properly and maintain a mechanical facility of this type. Thus the oxidation ditch process is unlikely to be the system of choice for flows less than 2 L/s (50,000 gpd). If such a system already exists, the municipality may wish to procure, under contract, the services of an individual or firm qualified to operate and maintain the facility, relieving the municipality of this burden.

Energy requirements for an oxidation ditch facility include drives for aerators and pumps, sludge dewatering (if applicable), heating, lighting, and miscellaneous uses. Annual electrical energy requirements may be expected to be approximately 72,000 kWh/yr for a 4 L/s (0.1 mgd) facility, 280,000 kWh/yr for a 22 L/s (0.5 mgd) plant, and 500,000 kWh/yr for a 44 L/s (1 mgd) facility.

### 5.14.7 Monitoring

Monitoring consists of sampling and conducting analyses as required by NPDES or other permits, and as is necessary to ensure optimum plant performance. NPDES requirements will vary substantially depending on the size of the facility and the characteristics of the receiving waters. NPDES monitoring requirements may include influent flow, influent BOD and SS, and effluent BOD, SS, chlorine residual, and nutrients (i.e., ammonia, phosphorus). Sampling frequency may range from daily to bimonthly. Operational monitoring generally includes MLSS, aeration basin DO, pH, settleability, alkalinity, effluent chlorine residual, influent flow, return sludge flow, sludge waste rate, and dewatered sludge solids content.

### 5.14.8 Residuals

Residuals generated from an oxidation ditch facility include screenings, grit, and waste sludge. Grit and screenings must be removed and disposed of promptly because of their putrescible nature. If no separate grit chamber is provided, grit accumulates in the oxidation ditch, eventually requiring draining and cleaning.

Due to the long solids retention time, waste sludge production is the same as for an extended-aeration system and somewhat less than for conventional activated sludge. Sludge may be wasted to a sludge holding tank or stabilization process such as an aerobic digester and removed from the plant as a liquid, or may be dewatered by sand bed or other devices and hauled away as a solid. Options for the handling of waste sludge from mechanical wastewater treatment systems are discussed in greater detail later in this chapter.

## 5.15 Sequencing Batch Reactors

### 5.15.1 Technology Description

The sequencing batch reactor (SBR) process is a form of the activated-sludge process in which aeration, sedimentation, and decant functions are combined in a single reactor. Most municipal SBRs consist of two or more parallel tanks. The process employs a five-stage cycle: fill, react, settle, draw, and idle. During the fill stage, wastewater enters the tank and mixes with the settled biological solids remaining from the previous cycle. The tank is mixed during the fill stage and may be aerated. During the react stage, the wastewater and mixed liquor are subjected to aeration, causing oxidation of organic matter. Aeration and mixing are stopped during the settle stage, allowing solids to settle. Clarified supernatant is decanted during the draw stage. After decanting, solids are removed from the bottom of the tank during the idle stage. By this stage the first reactor will have finished its cycle and the other reactor(s) will continue filling. When the second reactor completes its fill stage, it begins its react cycle and influent wastewater is directed to the first reactor.

Modifications to the process include systems that continually accept influent flow, and oxidation ditch systems that function as SBRs without the use of external clarifiers.

A typical SBR process consists of screening, grit removal, SBR cycling, and disinfection. The disinfection system must be designed so that it is either large enough to accommodate the high periodic flow during decanting or preceded by flow equalization. Primary clarification is not commonly practiced. A schematic diagram that includes the five stages of an SBR cycle is provided in Figure 5-11.

Critical components of an SBR system include the aeration/mixing system, the decant system, and the control system. Aeration/mixing systems include jet aeration, fine and coarse bubble diffusers, and mechanical turbine aerators. Of these, the jet aeration system appears to be most common. It has the advantage of being able to mix independently of aeration. This may be important if biological nutrient removal is desired.

A well-designed and reliable decant system is critical to ensuring a high-quality effluent by avoiding discharge of floating solids or MLSS. At least five different proprietary designs are available through vendors of SBR systems. Development of SBR technology in the United States has progressed rapidly since 1980, resulting in many improvements to decanter designs. Decanters include mechanically activated surface skimmers, floating decanters with hydraulically activated valves, floating pipes and orifices equipped with flapper valves operated by air pressure, and fixed-position siphons.

Process control is provided by a programmable logic controller (PLC), which along with associated software is provided by the manufacturer. Programs are typically developed and modified by the vendor to suit the intended application. PLC hardware is generally reliable. Construction is modular, and troubleshooting and module replacement are straightforward.

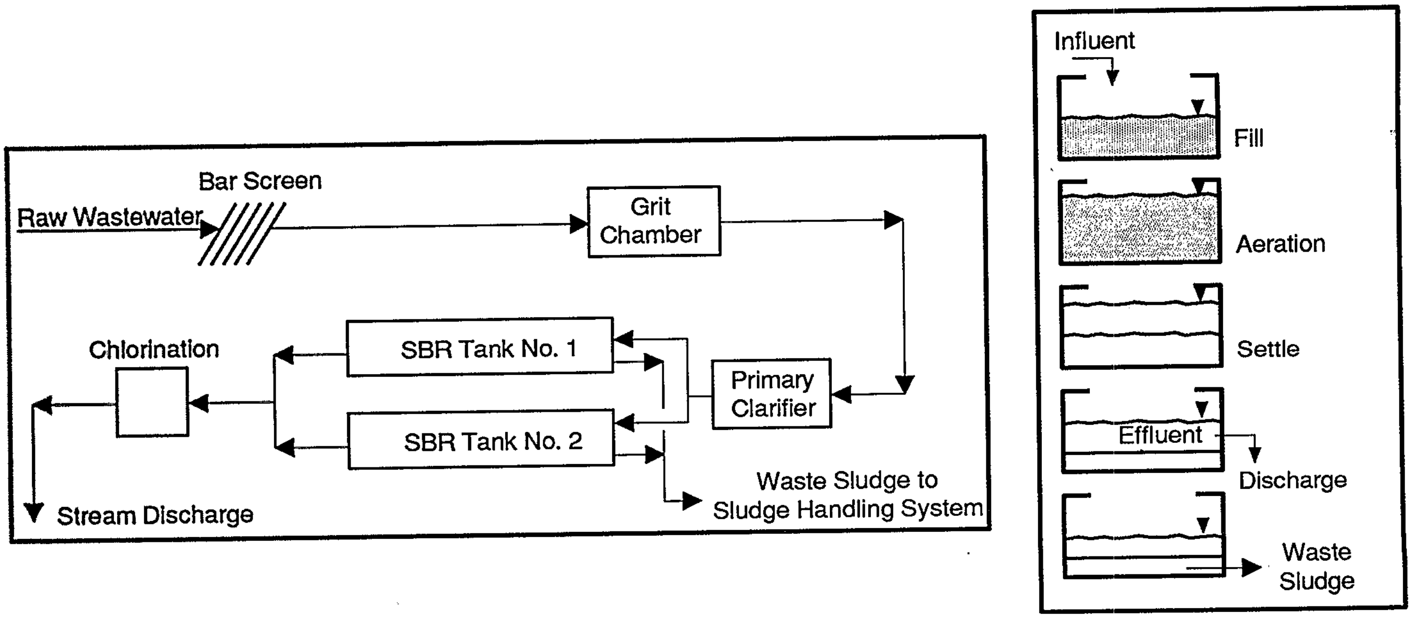



**Figure 5-11. Schematic and Stages for a Sequencing Batch Reactor Process.**

Sludge wasting is accomplished by wasting either aerated MLSS or settled solids. Often, submersible pumps are used to waste solids.

SBRs are capable of producing a high-quality effluent. Data collected by EPA from 13 secondary plants using SBR technology showed effluent average BOD levels of 3.3 to 21.0 mg/L, average SS levels of 3 to 25 mg/L, and average $NH_3$-N levels of 0.3 to 12.0 mg/L. These data are average values representing 7 to 12 months of operating data.

SBRs are also capable of both biological nitrogen and phosphorus removal, which is accomplished by proper reactor sizing and selection of stage lengths and aeration times to achieve the desired distribution of aerobic, anoxic, and anaerobic conditions. Scant data on full-scale operations are available to document the effectiveness for nitrogen and phosphorus removal, yet phosphorus removal appears quite easily achievable.

### 5.15.2 Applicability and Status

The SBR process has widespread application where mechanical treatment of small wastewater flows is desired. Because it provides batch treatment of wastewater, it is ideally suited for the wide variations in flow rates that are typically associated with small communities. Operation in the "fill and draw" mode prevents the "washout" of biological solids that often occurs with extended aeration systems subjected to wide flow variations.

Another advantage of SBR systems for small communities is that they require less operator attention than most mechanical treatment alternatives, yet in most cases they provide a very high-quality effluent. With rare exception, operators of SBR systems are enthusiastic about the technology, preferring it to conventional activated-sludge systems.

Although most municipal applications of SBR technology have been online only since 1980, the concept is well established. Many full-scale fill-and-draw systems were operated between 1914 and 1920, but most were later converted to the continuous-flow mode. The Pasveer ditch was a variation implemented during the early 1960s that employed continuous feeding of wastewater, with intermittent settle and discharge. The system was introduced in Denmark, and 250 plants were in operation by 1980. Similar systems were constructed in Australia beginning in 1965, all employing continuous-feed and intermittent discharge processes.

The first application of SBR technology in the United States was in 1980. Since that time, SBR systems have become more widespread. In 1989, approximately 150 SBR plants were in design or operation; at least 90 facilities were known to be operating. Nearly half of these had design flows less than 11 L/s (0.25 mgd), and a third had flows less than 4 L/s (0.1 mgd).

### 5.15.3 Advantages/Disadvantages

Key advantages of the SBR process include:

- Simple, reliable
- Ideally suited for wide flow variations
- Capable of very high and consistent effluent quality due to quiescent batch settling
- Requires less operator attention than most other mechanical systems
- High operational flexibility allowing capability for nutrient removal, filamentous growth control

The disadvantages of this process include:

- Some problems reported with decant systems
- Improvements to hardware continue to be made as the technology develops
- Requires skilled operator and regulator inspection and maintenance

### 5.15.4 Design Criteria

Available design criteria for the SBR process are summarized in Table 5-27. Vendors of SBR equipment typically provide substantial technical assistance in the design of the particular SBR system. Some vendors offer preengineered SBR package plants. No standard procedures have been developed for sizing an SBR system.

**Table 5-27. Summary of Available Design Criteria for SBR Process**

| Parameter | Range |
|---|---|
| Total tank volume | 0.5–2.0 times average daily flow |
| Number of tanks | Typically 2 or more |
| Tank depth (ft) | 10–20 |
| F/M | 0.04–0.2 |
| SRT (days) | 20–40 |
| Aeration system | Sized to deliver sufficient oxygen during aerated fill and react stage; $O_2$ requirements similar to conventional activated sludge (with nitrification as required) |
| Cycle times (hr) | 4–12 (typical) |

### 5.15.5 Capital Cost Sensitivity

SBR systems are likely to be extremely cost-competitive with other mechanical wastewater treatment systems over a wide range of flows. Unfortunately, scant historical data have been compiled comparing the capital cost of SBR systems with other competing mechanical systems. Clearly, the lack of need for an external secondary clarifier and return sludge pumping system offers potential savings in construction costs. In addition, primary clarification is normally not employed. Design considerations regarding the disinfection system (see Section 5.15.1), however, have the potential to offset some of the inherent advantages of SBR systems.

As with other mechanical systems, capital costs may be affected by site conditions such as climate, geology, and surrounding aesthetics. Costs for sludge stabilization and dewatering may account for a significant percentage of construction costs.

### 5.15.6 O&M Requirements

O&M requirements for an SBR system are expected to be among the lowest of any mechanical wastewater treatment plant. Experience with operating SBR processes has shown excellent reliability and relatively low maintenance. As with other mechanical systems, however, a more skilled operator is necessary. Sludge stabilization and dewatering are likely to constitute a substantial portion of the operating and maintenance costs. O&M requirements of an SBR plant are summarized in Table 5-28.

**Table 5-28. O&M Requirements of SBR Facilities**

**Operations**

- Effluent quality monitoring required by NPDES permit
- Operation analyses (e.g., MLSS, DO, settleability)
- Cleaning of screens, weirs, decant mechanisms, diffusers, and other components
- Adjustment of cycle times as required to optimize performance
- Regular wasting of solids
- Sludge dewatering and disposal
- Control of disinfectant dosage
- Administration and recordkeeping

**Maintenance**

- Blowers, mechanical aerators, or mixing pumps
- Waste sludge pumps
- Electrical equipment, including programmable logic controller
- Mechanical dewatering equipment
- Laboratory equipment such as pH, DO meters, chemicals
- Disinfection system

Energy consumption for an SBR facility is expected to be similar to that for an extended-aeration or oxidation ditch plant, but slightly greater than for a TFSC facility. This variability in energy requirements for the various systems is primarily due to the differences in energy required to transfer oxygen into the wastewater. Energy is consumed by aeration, mechanical sludge dewatering, preliminary treatment, heating, cooling, and lighting of the building housing the system, and other miscellaneous processes.

### 5.15.7 Monitoring

Monitoring consists of sampling and conducting analyses as required by NPDES or other permits, and as is necessary to ensure optimum plant performance. NPDES requirements will vary substantially depending on the size of the facility and the characteristics of the receiving waters. NPDES monitoring requirements may include influent flow, influent BOD and SS, and effluent BOD, SS, chlorine residual, and nutrients (i.e., ammonia, phosphorus). Sampling frequency may range from daily to bimonthly. Operational monitoring generally includes MLSS, aeration basin DO during react stage, pH, settleability, alkalinity, effluent chlorine residual, influent flow, and sludge waste rate.

### 5.15.8 Residuals

Residuals generated from an SBR facility include screenings and waste secondary sludge. Grit is also generated if the plant is equipped with a separate grit chamber. Grit and screenings must be removed and disposed of promptly because of their putrescible nature.

Due to the long solids retention time associated with many SBR facilities and the lack of primary clarification, waste sludge production is relatively low. Sludge may be wasted to a sludge holding tank or stabilization process, such as an aerobic digester, and removed from the plant as a liquid, or may be dewatered by sand bed or other devices and hauled away as a solid. Options for the handling of waste sludge from mechanical wastewater treatment systems are discussed in greater detail later in this chapter.

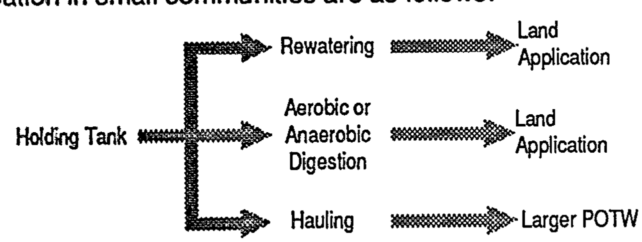
## 5.16 Sludge Handling Alternatives

### 5.16.1 Introduction

Treatment and disposal of sludge generated from the mechanical treatment of wastewater is a major problem facing small communities. Sludge handling can account for 50 percent of the cost of operating a wastewater treatment plant. Many sanitary landfills refuse or are reluctant to accept sewage sludge, and suitable areas for land spreading of stabilized sludge are becoming difficult to find in some urbanized areas of the country.

Regulations promulgated by EPA are quite specific with regard to the degree of stabilization required before sludge can be applied to the land. At a minimum, agricultural use of sludge requires the sludge to be pretreated by aerobic digestion, air drying, anaerobic digestion, or lime stabilization. Additional sludge stabilization can be accomplished by utilizing composting, heat drying, heat treatment, thermophilic aerobic digestion, dry lime stabilization, and other processes described by the EPA.

Sludge handling approaches that have the widest application in small communities are as follows:




Liming or digestion followed by land application is generally the most simple and economical means of handling and disposing of sewage sludge. It does require, however, the availability of land for application within a reasonable proximity of the sewage treatment plant. A significant number of communities employ landfill disposal instead. Virtually all landfills now require sludge to be dewatered to a minimum solids content of 20 to 25 percent.

### 5.16.2 Sludge Holding Tank

The first component of many small community sludge management schemes is a holding tank. The holding tank should be conservatively sized to hold sludge during periods of inclement weather, when dewatering capacity is not available, or should any other problem arise; a volumetric capacity sufficient to accommodate 30 days of sludge wasting is recommended. Most sludge holding tanks are aerated to prevent septic conditions and entrain solids in suspension when required. Diffusers should be of the coarse-bubble type with the capability to be removed for cleaning. They should provide a minimum of 0.4 L of air per $m^3$ of tank volume(s) (25 cfm/1,000 cu ft). Depending on the application, holding tanks should be designed to allow periodic sludge settling and decant of supernatant, increasing solids content and reducing the volume for dewatering and/or disposal. Even if septic conditions are prevented with aeration, odor emissions may be a problem. Simple offgas biofilters composed of soil, compost, or a combination of such materials can provide an economical means of controlling such emissions.

### 5.16.3 Dewatering Beds

Dewatering beds remove sludge moisture by drainage and by evaporation. Underdrainage is collected and returned to the plant headworks. Drying beds typically consist of 15 to 25 cm (6 to 10 in.) of sand placed over 20 to 46 cm (8 to 18 in.) of gravel or stone. Effective size of the sand is 0.3 to 1.2 mm, and uniformity coefficient is less than 5. Gravel is generally graded from 1/8 to 1 in. Underdrains are vitrified clay with open joints or perforated PVC pipe with a minimum diameter of 10 cm (4 in.) and minimum slope of 1 to 2 percent. A cross section of a typical sand drying bed is shown in Figure 5-12.

Sludge is preferably applied to the beds in 20 to 25 cm (8 to 10 in.) lifts. Drying beds are partitioned into sections 3 to 6 m (10 to 20 ft.) wide and 6 to 30 m (20 to 100 ft.) long. Partitions may be constructed using pressure-treated wooden planks, earthen dikes, or concrete walls. Concrete foundations are required around the drying area if it is to be covered (in colder climates).

Feed piping may be ductile iron or PVC. It should be sized to prevent clogging yet maintain adequate cleansing velocities. Provision should be made for flushing and draining the line. Diversion boxes or valves are used to feed the desired bed. Splash plates are usually employed in order to minimize erosion of the sand and improve sludge distribution.

After the sludge has dried to a minimum solids content of 20 to 25 percent, it is removed from the bed. For small operations this is typically done manually, although small front-end loaders are preferable. Sand that may be removed in the process will eventually require replacement. Modern beds are designed with longitudinal concrete strips allowing access by a truck or front-end loader.

For cold or wet climates, glass enclosures can be constructed to improve drying during inclement weather. Ventilation is necessary for moisture control.

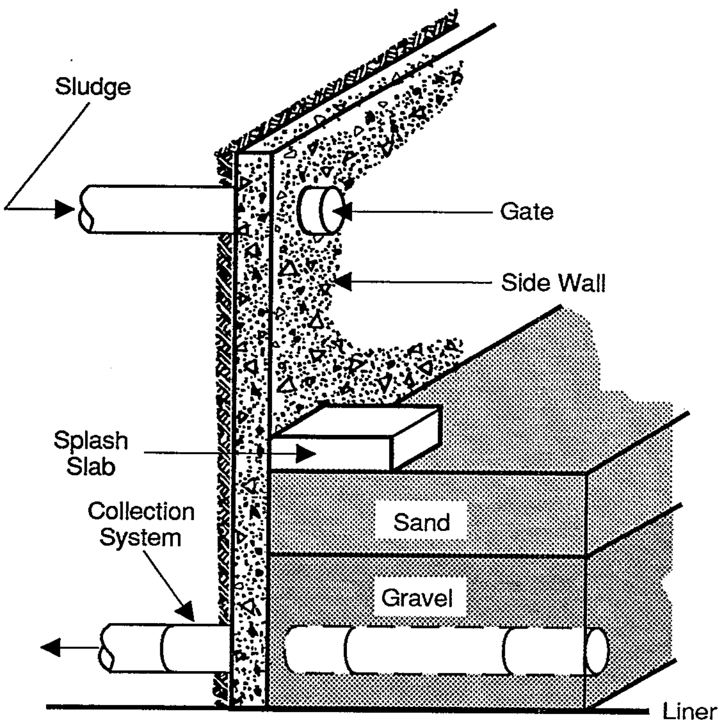



**Figure 5-12. Sand Drying Bed Details.**

Modifications to traditional sand drying beds include use of wedgewire or plastic block bottoms, paved drying beds, vacuum-assisted drying beds, and special mixing devices for areas of intense sunlight. All but the paved bed require chemical (usually polymer) conditioning prior to sludge application, thus limiting these particular modifications to systems operated by skilled O&M staffs. All of the alternative systems can be cleaned by mechanical means, substantially reducing labor costs associated with dewatered sludge removal. Field experience indicates that the use of totally and partially paved drying beds result in shorter drying time than with conventional drying beds. Experience with polymer treatment followed by drying on plastic- or wedgewire-bottom beds has been limited in the United States, but indications are promising. Final solids vary from about 12 percent with immediate removal, to higher percentages with continued drying.

Vacuum-assisted sludge drying (VASD) beds are a relatively recent innovation developed in the mid-1970s. This technology consists of an array of specially made porous blocks or plates with sealed joints. After the polymer-treated sludge has drained supernate, a pump applies a vacuum to the bottom, pulling additional moisture from polymer-conditioned, gravity-drained sludge until it cracks. The major advantage of these alternative systems is shorter drying times. The normal VASD cycle time for digested sludges is considerably less than 24 hours. Final solids are generally 13 to 15 percent, unless dried further before disposal.

Table 5-29 provides a summary of information on sludge dewatering beds.

**Table 5-29. Summary of Information on Sludge Dewatering Beds**

| | |
|---|---|
| **Applicability and status** | Simple, widely-used, effective system |
| **Advantages** | Minimal skill and operator attention required; low construction costs; effectiveness, variable and subject to site conditions |
| **Disadvantages** | Sludge removal is labor intensive; covering may be required in cold or wet climates; odor and vector potential; land requirements are considerable |
| **Design criteria** | Open beds: 2.0 – 2.5 sq ft/capita<br>10 – 28 lb/sq ft/yr<br>Closed beds: 60 – 75 percent of open bed area<br>12 – 40 lb/sq ft/yr |
| **Capital cost sensitivity** | Capital cost may be substantially increased if covering is required |
| **O&M** | Sludge removal is labor intensive, but overall operating costs are low compared to mechanical dewatering. Energy costs are minimal; operation is simple |
| **Monitoring** | As required by disposal or end-use permit; operational monitoring consists of cake solids content measurement |
| **Special considerations** | Odors, flies may be a problem, particularly if the sludge is not well stabilized |

## 5.16.4 Sludge Conditioning

### *5.16.4.1 Aerobic Digestion*

Aerobic digestion of sewage sludge is commonly used by small communities as a means of stabilization prior to land application. If application sites are in reasonably close proximity to the source of sludge, direct application of liquid sludge is clearly the most economical approach. If not, dewatering via sand drying beds may be used to reduce the volume of sludge to be hauled away.

Aerobic digestion as practiced by small communities is typically performed in an open tank. Continuous introduction of air allows biological oxidation of organic matter under aerobic conditions. Aerobic digestion results in a reduction in biodegradable volatile solids, improved dewaterability, and odor reduction. Oxygen may be supplied by mechanical aerators or diffusers.

Solids retention times (SRTs) in aerobic digesters may vary from 30 to 60 days depending on the ambient temperatures of the site and the sludge. Minimum retention times are likely to be dictated by state regulations that specify the degree of stabilization necessary prior to agricultural utilization. EPA guidelines indicate 60 days at 15°C (59°F) to 40 days at 20°C (68°F), with a volatile solids reduction of at least 38 percent. Actual operating experience would suggest that 40 percent VSS reduction can be achieved at lower SRTs and that the minimum SRT to achieve such reduction is dependent on the type of sludge. For example, extended aeration and oxidation ditch sludges typically contain no primary sludge and have already undergone partial stabilization from endogenous respiration.

For small operations, single-tank digesters are commonly used. Periodically, aeration is stopped and solids are allowed to settle for 6 to 12 hours. Clarified supernatant is then decanted and returned to the plant. This procedure maximizes the solids capacity of the digester, and also allows a thickened sludge to be removed, thereby reducing the volume to be hauled away. Generally, supernatant quality from an aerobic digester is favorable, and impacts on the plant from supernatant decant are minimal.

A recent development from Germany is the autothermal thermophilic aerobic digestion process (ATAD). It is just now being introduced into North America. European studies indicate a superior stabilization with greater cost-effectiveness: greater destruction of organisms and reduction of VSS at a price competitive with conventional aerobic digestion (EPA, 1990).

Table 5-30 provides a summary of information on the aerobic digestion process.

**Table 5-30. Summary of Information on Aerobic Digestion of Sludge**

| | | |
|---|---|---|
| **Applicability and status** | Simple, widely used process for sludge stabilization | |
| **Advantages** | Minimal operator attention; low odor potential; simple process; resistant to upset; no chemical required | |
| **Disadvantages** | Very high power consumption; aeration system requires high maintenance; larger volume tank required for cold climate application; foaming potential | |
| **Design criteria** | SRT: | 40 days at 20°C<br>60 days at 15°C |
| | VSS loading: | 0.1–0.4 lb/cu ft/d |
| | Diffused air req'd: | 20–60 cfm/1,000 cu ft |
| | Mech. aeration: | 0.75–1.25 HP/1,000 cu ft |
| | Minimum DO: | 1–2 mg/L |
| **Capital cost sensitivity** | Detention times, digester volumes, and construction costs increase with decreasing sludge temperatures | |
| **Operation and maintenance** | Relatively little operator attention for process except for monitoring and decant/sludge withdrawal operations; high maintenance for mechanical or diffused aeration system; high power requirements for oxygen transfer | |
| **Monitoring** | As required by end-use permit; operational monitoring consists of DO, $VSS_{in}$, $VSS_{out}$, temperature | |

### *5.16.4.2 Anaerobic Digestion*

Anaerobic digestion is nearly as common among existing small U.S. community systems as aerobic digestion. Like the latter, subsequent processing may be a function of proximity of the disposal site. When dewatering is called for, sand beds are the most frequent choice. However, the use of anaerobic digesters for new treatment facilities is becoming far less popular now that designers are recognizing the value of operational simplicity in new facilities.

When used, modern anaerobic digesters are generally designed as two-stage systems, to maximize the stabilization process and gas production. The recovery and use of this gas (primarily methane) is considered the most attractive feature of anaerobic digesters, but the complexity of the recovery and use process makes their feasibility marginal for wastewater systems with capacities in the range of 100,000 to 1 million gpd and unlikely in smaller systems, unless regional energy recovery facilities are in place. In these cases, the sludge from the wastewater treatment plant becomes an additional energy-producing raw material that must merely be transferred from the treatment facility to the energy recovery facility along with the responsibility for disposal.

### *5.16.4.3 Lime Stabilization*

The addition of lime to sledges in sufficient quantities to raise the pH above 12 for 30 minutes has been employed by small municipalities in Europe and in the United States for many years. Lime treatment has several advantages over biological conditioning including low capital cost, ease of use, and improved pathogen reduction. It also serves as an ideal temporary or interruptable supplemental process for periodically overloaded existing digester systems. The chief disadvantages of lime stabilization are the inability of the process to reduce volatile solids (or sludge mass) and the large increase in additional inert solids for dewatering and disposal. Also, subsequent loss of the advantages of alkaline status upon storage must be accounted for in the subsequent sludge processing steps (e.g., odors, slower dewatering rates). Design criteria and other information on lime stabilization in small community treatment plant sludges is provided in Table 5-31.

In recent years, much has been made of new commercial processes for the alkaline treatment of waste sludge "cake." Most of these processes have been applied to larger municipal facilities (>5 mgd capacity). It is doubtful that these systems will have a significant impact on small facilities unless they contribute to a regional sludge man-

**Table 5-31. Summary of Information on Lime Stabilization of Sludge**

| | |
|---|---|
| **Applicability and status** | Simple process |
| **Advantages** | Low capital cost; favorable pathogen destruction; on/off capability; low maintenance requirements. |
| **Disadvantages** | Large increase in sludge solids; no destruction of volatile SS; loss of benefits after storage > 2 wk. |
| **Design criteria** | Dosage = attainment of pH >12 for 30 minutes after mixing; mixing by aeration (preferred) or mechanical means; high-calcium lime yields faster reaction time (hydrated lime preferred) |
| **Capital cost sensitivity** | Lime dosage to meet pH/time requirements, storage time, and mixing requirements directly affect operating costs. Higher calcium content of lime reduces cost. Capital costs of < $150,000 can be anticipated for STPs of mgd. |
| **O&M** | Lime costs of < than $50/day; total O&M costs are primarily in range of $10 to 20/metric tone of solids, the great majority being lime and labor; maintenance of lime feeding and mixing equipment is greatest demand |
| **Monitoring** | pH is the primary parameter for monitoring |

agement facility. In this case the responsibility for such processing would be transferred to the regional facility.

### 5.16.5 Land Application of Sludge

Application of stabilized sludge to the land is commonly practiced by small, rural communities. It is a simple process and provides substantial benefit to agricultural and marginal lands given the value of sewage sludge as a soil conditioner and source of organic matter and nutrients.

Sludge can be applied as a liquid or as a dewatered cake. Many small municipalities favor applying sludge as a liquid since it eliminates the dewatering step and stabilized sludge can be taken directly from the plant to the land. Liquid sludge can be applied by tank truck, subsurface injection, or spray irrigation. Dewatered sludge can be applied by a manure spreader and incorporated into the soil by disking or plowing.

Using the same tank truck both to haul and apply sludge is a simple and economical approach, but it can cause compaction and rutting of the soil. To overcome this problem, spray irrigation is recommended. Equipment used for spray irrigation of sludge is simple and reliable. Another approach is to use specially designed applicator trucks with flotation tires, but such equipment is costly for small communities to purchase and maintain unless they offset costs by receiving sludges from neighboring communities.

One of the major drawbacks of land application is the need to store sludge when weather or soil conditions do not permit application. Sludge is not applied when the soil is wet or frozen. In certain areas of the country, this restriction requires the capability to store sludge for long periods of time.

The rate of sludge application to land may be governed by nutrient loadings or loadings of trace elements such as cadmium. In general, application rates depend on sludge composition, soil/hydrogeological characteristics, climate, vegetation, and cropping practices. Annual application rates have varied from 1 to over 220 t/ha (0.5 to 100 tons/ac) per year. For most small municipalities treating sewage of domestic origin, sludge metals levels are generally low, and application rates are often based on supporting the nitrogen needs of the crop. Typically 900 dry kg (1 ton) of sludge provides 27 kg (60 lb) of nitrogen (50 percent of which is available), 18 kg (40 lb) of phosphate, and 2 kg (5 lb) of potash.

Table 5-32 provides a summary of information on land application of sludge.

**Table 5-32. Summary of Information on Land Application of Sludge**

| | |
|---|---|
| **Applicability and status** | Simple, widely used process of sludge disposal for small communities |
| **Advantages** | Simple; low cost; utilizes sludge as soil conditioner, source of organic matter and nutrients |
| **Disadvantages** | Storage requirements may be considerable to accommodate sludge generation during wet, frozen soil conditions; odor potential; local opposition possible; close monitoring required |
| **Design criteria** | Dependent on sludge composition, soil characteristics, vegetation, cropping practices |
| **Capital cost sensitivity** | Costs may increase substantially if large storage volumes required; cost of specialized application vehicles high, service life is approximately 10 yr |
| **O&M** | Costs sensitive to distances between sludge source and application points; maintenance of vehicles and application equipment required; monitoring costs may be high |
| **Monitoring** | Monitoring requirements dictated by state regulations; requirements increase with size of operation; monitoring required for sludge soil, ground water, crops |

## 5.17 Septage Handling Alternatives

### 5.17.1 Introduction

Providing adequate treatment and disposal alternatives for septic tank pumpings (septage) is a major challenge for small communities. Raw septage is a highly putrescible material that may contain high levels of grit, grease, and debris. It is difficult to dewater, and the characteristics are highly variable. In addition, daily septage generation rates are unpredictable, with substantial seasonal variations occurring due to changes in soil and ground-water conditions. In the past, septage has been commonly applied to the land in a raw state. However, many states are now implementing more stringent regulations requiring stabilization of septage prior to land application in order to minimize the potential for disease transmission, vector attraction, and odor emissions. In some areas, high densities of septic systems, lack of available land, and lack of available sewage treatment plant capacity have justified the construction of high-cost, independent facilities dedicated solely to the treatment of septage.

### 5.17.2 Septage Characteristics

Septage characteristics are highly variable depending on such factors as tank size and design, user habits, pumping frequency, climate and seasonal weather conditions, and presence of appliances such as garbage grinders, washing machines, and water softeners. Table 5-33 provides recommended design values for various characterization parameters.

Ratios of peak monthly to average monthly septage volume that can be expected at a treatment site range from 1.5 to 2.5. Ratio of peak daily to average daily volume can be expected to range from 3.0 to 5.0. Annual septage volumes can be estimated from historical records, if available; by assuming a generation rate of 230 to 380 L/cap/yr (60 to 100 gal/cap/yr); or by assuming a typical tank volume of 3,800 L (1,000 gal) and a pumpout frequency of every three to five years.

### 5.17.3 Overview of Septage Handling Options

Figure 5-13 provides an overview of septage handling options available to a small community. The most common approach is land application; co-treatment at a sewage treatment plant is also widely employed. For small communities, independent facilities are usually limited to simple, low-cost approaches such as stabilization lagoons or lime stabilization systems.

### 5.17.4 Land Application

Land application of septage employs the same techniques used for land application of liquid sewage sludge discussed earlier in this chapter. Where stabilization of liquid septage is required prior to land application, lime stabilization should be considered, since it is one of the most simple and economical techniques to meet applicable stabilization criteria. Lime stabilization involves addition and mixing of sufficient lime to achieve a pH of 12 for at least 30 minutes. To achieve pH of 12, lime dosages can be expected to be 1,500 to 3,500 mg/L as $Ca(OH)_2$, which is hydrated lime. (Lime stabilization is discussed in more detail earlier in this chapter.)

**Table 5-33. Suggested Design Values for Septage Characteristics[a]**

| Parameter | Value (mg/L)[b] |
|---|---|
| TS | 40,000 |
| TVS | 25,000 |
| TSS | 15,000 |
| VSS | 10,000 |
| COD | 20,000 |
| $BOD_5$ | 7,000 |
| TKN | 15,000 |
| $NH_4$-N | 700 |
| Total P | 150 |
| Alkalinity | 1,000 |
| Grease | 8,000 |
| pH | 6.0 |
| LAS | 150 |
| Heavy metals | |
| Zinc | 10 |
| Copper | 5 |
| Lead | 1 |
| Cadmium | 0.1 |
| Mercury | 0.005 |
| Toxic organics | |
| Methyl alcohol | 16 |
| Isopropyl alcohol | 14 |
| Acetone | 11 |
| Methyl ethyl ketone | 4 |
| Tuluene | 0.2 |

[a]EPA 1984a; 1991.
[b] Except for pH

TS = total solids
TVS = total volatile solids
TSS = total suspended solids
VSS = volatile suspended solids
COD=chemical oxygen demand
BOD = biochemical oxygen demand
TKN = total kjeldahl nitogen
$NH_4$-N = ammonia nitogen
P = phophate
LAS = linear alkyl sulphonate

### 5.17.5 Co-treatment at a Sewage Treatment Plant

Many septage haulers dispose of septage at wastewater treatment plants. This is an acceptable alternative if the plant has sufficient capacity and is equipped to handle the material.

The most common approach is to add the septage to the headworks of the plant. In many plants, this allows the septage to undergo screening and grit removal, which is

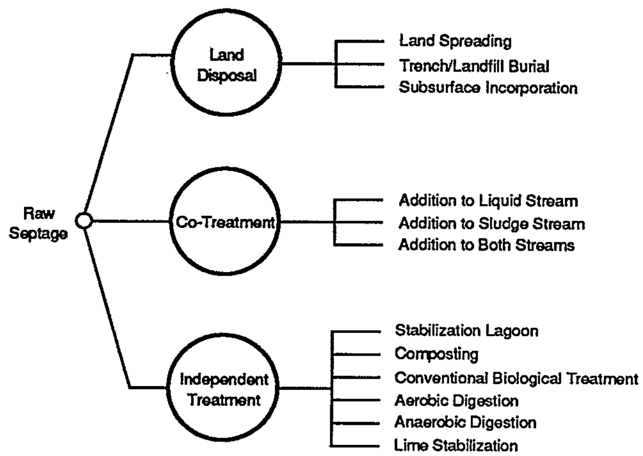



**Figure 5-13. Basic Septage Management Options.**

important to remove debris and grit that can foul equipment or cause excessive wear on pump components. Larger plants (> 88 L/s (2 mgd)) often allow haulers to discharge directly to an influent channel or manhole immediately upstream of the headworks. However, for small plants, this can result in a substantial "shock load" to the biological system, particularly if the plant is not equipped with a primary clarifier. Primary clarifiers allow removal of many of the organic septage solids, damping the organic loading to the aeration basin. For small plants lacking primary clarifiers or for larger facilities receiving high volumes of septage, a septage holding/equalization tank is strongly recommended. The tank should be sized to handle twice the peak daily volume and should be equipped with an air mixing system to maintain solids in suspension. A solids handling pump is then used to transfer septage to the headworks at a predetermined rate. A timer and level control system can be used to establish these feed patterns. Because odor emissions from a septage holding tank are a major concern, provisions should be made for a biofilter or other simple odor-control device.

Details of a septage receiving pad may be found elsewhere (EPA, 1984; Rezek and Cooper, 1980). However, provision should be made for a trash rack and washdown facilities. It is important that a log be maintained of all septage deliveries; at a minimum, entries should include hauler's name, time of delivery, volume, and source of septage. Also, a sample of each load received at the facility should be taken.

Septage may also be treated as a sludge, and handled as part of the solids management scheme at the sewage treatment plant. This can be accomplished effectively by adding septage upstream of a primary clarifier; however, for small extended-aeration plants or oxidation ditches without primary clarification, handling septage in the solids handling train deserves consideration. Septage may be added to the stabilization process, such as an aerobic digester, or to the sludge holding tank prior to dewatering. Separate screening and grit removal is recommended for either approach, however. Septage is not easily dewatered, and blending with treatment plant sludge prior to dewatering may have major adverse impacts on the dewatering unit's performance. For this reason, addition of septage to the stabilization process is the preferred method of handling this waste in the solids train.

Table 5-34 provides a summary of information on septage handling at a wastewater treatment plant. Table 5-35 lists specific factors to be evaluated for co-treatment of septage and watewater.

### 5.17.6 Independent Septage Treatment Facilities

Independent facilities for the treatment of septage may be warranted if land is not available for land spreading, or if adequate treatment plant capacity is not available within a reasonable proximity (16 to 32 km (10 to 20 mi)) to the sources of septage. Independent facilities can vary in scope from simple lime stabilization systems to complex mechanical septage treatment plants comprising multiple physical and biological processes.

Independent septage treatment facilities using technology compatible with the capabilities of a small community would most likely utilize aerobic digestion, lime stabilization, or lagoons.

#### *5.17.6.1 Aerobic Digestion*

Aerobic digestion of septage is very similar to aerobic digestion of sewage sludge (see discussion of the aerobic digestion process, earlier in this chapter).

An independent septage treatment facility using the aerobic digestion process should employ screening and grit removal as part of a preliminary treatment scheme. Residuals from these processes require regular disposal.

Although the aerobic digestion process is a relatively simple means of stabilization, there are several important concerns related to its use at an independent septage treatment facility. First, power costs are likely to be quite high to accomplish the transfer of oxygen. Second, supernatant decanted from the digester must be disposed of properly. Supernatant may require additional treatment prior to introduction to a subsurface disposal system as well as storage prior to use in an irrigation system. Various options for an independent aerobic digestion facility are shown in Figure 5.14.

#### *5.17.6.2 Lime Stabilization*

Lime stabilization is likely to be among the most cost-effective options for stabilization of septage to meet land application criteria and/or conditioning of septage prior to

**Table 5-34. Summary of Information on Co-treatment of Septage at a Sewage Treatment Plant**

| | |
|---|---|
| **Applicability and status** | Commonly used technique for septage disposal |
| **Advantages** | Sewage treatment plants generally equipped to handle such waste; compatible with sewage for biological treatment; maintains waste treatment operations at one location |
| **Disadvantages** | Some plants may have insufficient capacity to handle large volumes; increases aeration, scum, and sludge handling requirements; requires adequate preliminary treatment systems; may increase odor emissions |
| **Design criteria** | Screening and grit removal should be provided; organic loading capacity of plant must be evaluated; primary clarification desirable to minimize impact on biological system; separate receiving station and equalization recommended |
| **Capital cost sensitivity** | Additional capital cost may be only for receiving station, equalization/holding tank, odor control system |
| **O&M** | Additional O&M costs likely for increased aeration, sludge and scum handling requirements, grit and screenings disposal, cleanup |
| **Monitoring** | Septage volumes, characteristics, normal wastewater plant monitoring |

**Table 5-35. Factors To Be Considered in Evaluating Co-treatment of Septage**

**Design Factors**

- Types of unit processes at wastewater treatment plant
- Design capacity (hydraulic and organic) of the plant
- Proposed location of septage input
- Volume of septage (average, peak)
- Mode of addition (slug or continuous loading)
- Ratio of existing loadings to design loadings (hydraulic and organic)

**Potential Impacts**

- Increased organic loading to biological process
- Increased solids loading to sludge handling processes
- Increased hydraulic loading
- Scum buildup on primary clarifier surfaces
- Foaming problems in aeration basins
- Increased odor emissions from headworks, primary clarifiers
- Potential upsets due to toxic substances present in nondomestic septage
- Potential to exceed effluent limitations for BOD, TSS, nutrients

dewatering. The process is simple and requires a minimum of operator skill and attention.

The process involves addition of sufficient lime or other alkaline material to raise the pH to 12 for a period of 30 minutes. This destroys pathogenic organisms, improves dewaterability, and reduces objectionable odors. Lime can be added in the form of quicklime, CaO, or $Ca(OH)_2$. In addition, other alkaline materials such as lime kiln dust or cement kiln dust can be used to elevate the pH.

Lime stabilization of septage is typically conducted on a batch basis using a single- or two-tank system. For small operations, lime is added manually to the tank containing screened, degritted septage. The tank contents are then mixed; air mixing appears to be the most effective means of mixing lime with liquid septage. The pH of the liquid is monitored several times during the lime addition process. When a pH of 12 is reached, the contents are maintained at that pH for a period of 30 minutes. At this pH, release of ammonia ($NH_3$) can be expected.

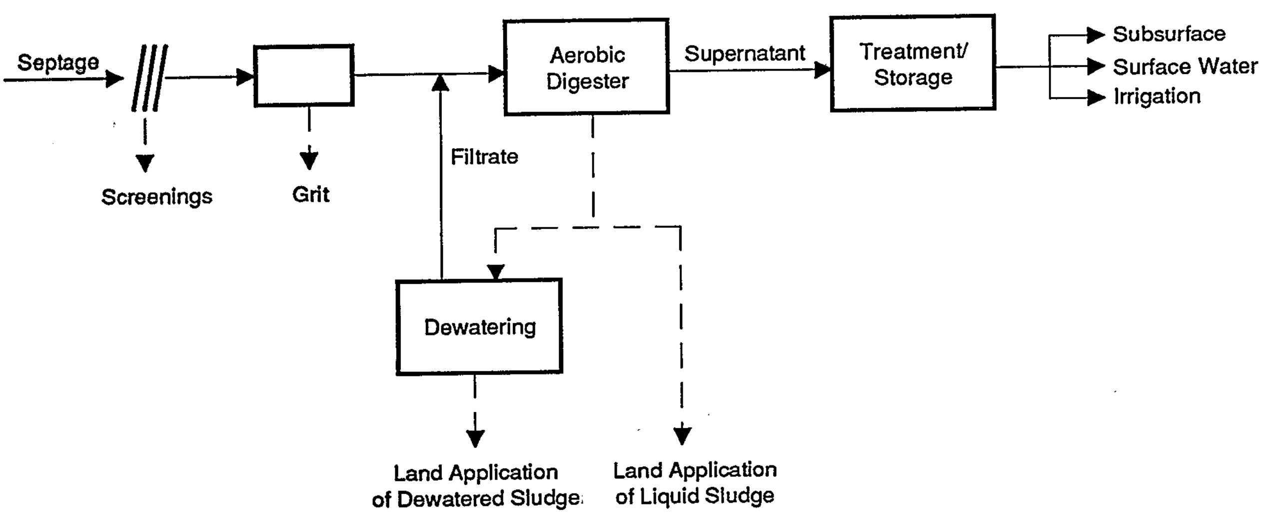



**Figure 5-14. Alternatives for an Independent Aerobic Digestion Facility for Septage Treatment.**

The stabilized sludge may be applied to the land as a liquid or dewatered first, using sand drying beds. If dewatered, treatment and disposal of the filtrate must be considered.

Information on lime stabilization of septage is summarized in Table 5-36.

**Table 5-36. Summary of Information on Lime Stabilization of Septage**

| | |
|---|---|
| **Applicability and status** | Simple, fully developed technology with broad application for small communities |
| **Advantages** | Simple batch processes; minimal operator skills; economical; meets EPA pathogen-reduction criteria; reduces objectionable odors; improves dewatering |
| **Disadvantages** | No organic destruction; increases mass of solids to be handled; mechanical lime feed systems, if used, require high degree of operator attention; dusty operation; potential ammonia releases |
| **Design criteria** | Screening, grit removal recommended. Air mix tank, batch process<br>pH = 12.0 for 30 minutes<br>Dosage: 1,500–3,500 mg/L as $Ca(OH)_2$ |
| **O&M** | Generally low O&M for small batch operations; larger, mechanized operations may have high O&M requirements; relatively low power consumption |
| **Construction issues** | If dewatering is part of treatment scheme, facilities must be constructed to treat/dispose of filtrate |
| **Monitoring** | As required by disposal permit; operational monitoring consists of pH measurement; solids content of cake, if applicable |

### *5.17.6.3 Lagoons*

Lagoons are commonly used for the treatment of septage, particularly in the northeastern United States. Properly designed and sited, lagoons provide consistent performance and are easy to operate. The simplest system consists of two earthen basins in a series. Raw septage is discharged into the primary lagoon. Partially clarified effluent from the first cell is discharged into a second lagoon that also acts as a percolation pond. Liquid is disposed of through a combination of infiltration and evaporation. Figure 5-15 shows various configurations for septage lagoons, including those that discharge to the ground water using percolation ponds or infiltration basins and those that produce an effluent that is subjected to further treatment and/or is applied to the land. Depending upon State regulations and subsurface geological and ground-water conditions, lining of the lagoons may be required, and the type that discharges to ground water may be prohibited. Concrete, asphalt, or clay liners are recommended over membrane liners due to the potential for damage to membrane liners during removal of accumulated solids. Lagoons are often constructed above grade with earthen embankments to minimize excavation costs.

Although performance data are limited, existing data suggest that septage be treated in two lagoons in a series prior to treatment in infiltration beds and percolation ponds, and before land application or further treatment and surface discharge. One of the major operating considerations with septage lagoons is accumulation of solids. Lagoons may require periodical dredging, and percolation ponds may require draining and cleaning to restore infiltrative capacity. For these reasons, use of two parallel lagoon systems is recommended.

To minimize odor emissions during the discharge of septage from hauler trucks, the material should be dis-

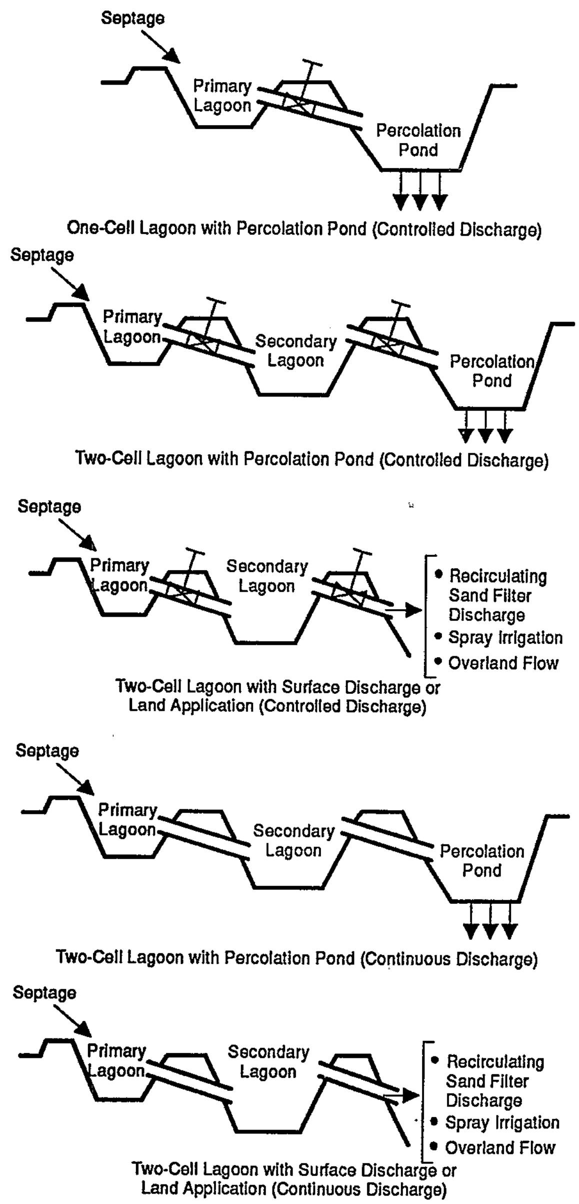



**Figure 5-15. Septage Lagoon Variations.**

charged into a concrete chamber with a tight-fitting manhole or hatch. Septage then flows by gravity into the first cell of the lagoon, below the water surface to avoid turbulence. Because of the large exposed surface area, odor emissions may be a problem. Large buffer zones (300 ft (90 m)) are recommended. Addition of lime to maintain a pH 7 may help reduce objectionable odors.

A summary of information on septage lagoons is provided in Table 5-37.

**Table 5-37. Summary of Information on Septage Lagoons**

| | |
|---|---|
| **Applicability** | Suitable for septage disposal where geological conditions, buffer areas are adequate |
| **Advantages** | Relatively low construction cost; consistent performance; simple operation |
| **Disadvantages** | Potential for ground water contamination and odor problems; large land area required; ground-water monitoring essential; eventually, dredging required to remove accumulated solids |
| **Design criteria** | Minimum 2 lagoons in series followed by percolation beds, land application, treatment/discharge.<br>Two parallel systems recommended<br>Percolation bed loading: 1 gpd/sq ft<br>Minimum ground-water separation distance: 4 ft (to top of ground-water mound)<br>Detention time (min.): 20 days in lagoons<br>Lagoon depth (min.): 3 ft<br>Lining: as required by state: concrete, asphalt, clay<br>pH control: 6.8 - 7.2 using lime |
| **Capital cost sensitivity** | Cost increase with lagoon lining requirements and need for further treatment prior to surface discharge |
| **O&M** | Does not require skilled operator; dredging/cleaning operations are labor intensive; ground-water monitoring necessary; lime addition may reduce odor emmisions |
| **Monitoring** | Ground water monitoring required; effluent monitoring may be necessary depending on final disposal; minimal operational monitoring |
| **Residuals** | Lagoon dredgings; scrapings from infiltration basins |

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National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
(703) 487-4650

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