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Summary Report

Small Community Water and Wastewater Treatment

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Summary Report

Small Community Water and Wastewater Treatment

Prepared for:

U.S. Environmental Protection Agency
Center for Environmental Research Information
26 West Martin Luther King Drive
Cincinnati, OH 45268

Prepared by:

Science Applications International Corporation
501 Office Center Drive
Fort Washington, PA 19034

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INTRODUCTION

OVERVIEW

Small communities face a variety of environmental infrastructure and public health issues. One of the largest concerns of a small community is the provision of safe drinking water and reliable wastewater collection and treatment. Due to their size, small communities are often limited in their ability to address this technically complex and potentially costly issue.

Many organizations exist to provide technical and financial services to small communities as they struggle with their drinking water and wastewater problems. These "service providers", through a network of local, county, state, and regional offices, are uniquely qualified to assist small communities in identifying the most cost-effective solutions to their problems. Four such organizations are: the National Rural Water Association (NRWA); the Rural Community Assistance Program (RCAP); the Coalition of Environmental Training Centers (CETC); and the U.S. Department of Agriculture, Cooperative Extension Service (USDA-CES). Traditionally, these organizations work with the U.S. Environmental Protection Agency (U.S. EPA) to provide valuable technical assistance to small communities dealing with many different environmental and public health issues.

PROJECT DESCRIPTION

During the summer of 1991, two pilot workshops were conducted to test a unique concept for providing assistance to small communities. The U.S. EPA's Office of Water (OW) and Office of Research and Development (ORD) collaborated to develop a two-day workshop designed to assist small communities dealing with the many new drinking water and wastewater treatment requirements. Different U.S. EPA perspectives on small-community drinking water and wastewater problems were provided by the Office of Ground Water and Drinking Water (OGWDW); the Office of Wastewater Enforcement and Compliance (OWEC); the Risk Reduction Engineering Laboratory (RREL); and the Office of Technology Transfer and Regulatory Support (OTTRS).

This cooperative effort produced a workshop that combined technical presentations, open discussions, and group exercises to achieve the following objectives:

- To provide technical information on drinking water and wastewater technologies suited for small-community applications
- To provide a forum for small-community decision makers (mayors, council members, town managers) and drinking water and wastewater system operators to foster improved communications at the local level
- To use the talents of local, county, and state service providers (NRWA, RCAP, CETC, USDA-CES) to develop and present technical information and to provide important local perspectives on drinking water and wastewater issues
- To improve communication among small communities and the network of government agencies and non-government organizations that provide technical and financial assistance
- To encourage communication and cooperation among small communities to develop and implement mutually beneficial solutions to drinking water and wastewater problems

Pilot workshops were conducted in Lafayette, Louisiana and Eugene, Oregon. In both locations, local, county, state, and regional service providers were involved to provide local insights into the problems faced by the participating small communities. About 15 to 20 small communities were represented at each workshop.

The following issues were addressed during the two-day workshop:

- Drinking water and wastewater regulations
- Building a good management team
- Importance of well trained operators
- Drinking water distribution and treatment systems
- Wastewater collection and treatment systems
- Funding and maintaining financial health
- Small community self-assessment

Four Utah Communities Work Together to Reduce the Cost of Complying with New Drinking Water Sampling and Analyses Requirements

The 1986 amendments to the Safe Drinking Water Act resulted in new sampling and analyses requirements for small communities. Many small communities have a limited understanding of the technical aspects of the new requirements and the costs for additional sampling and analyses can be high.

Several communities in Washington County, Utah have banded together to help each other to comply with new volatile organic compound (VOC) testing requirements. Assisted by the Rural Community Assistance Corporation (RCAC) and the Five County Association of Governments (FCAG), the communities identified state-certified laboratories and negotiated the best price for the VOC analyses. The laboratories offered discount rates for the testing, provided the group submit samples as one entity.

RCAC and FCAG provided technical assistance on proper sampling procedures to the system operators in these communities and also facilitated several meetings for the system operators to discuss other issues of mutual interest. The communities have continued to meet on a quarterly basis to discuss problems and have received technical presentations from state and U.S. EPA speakers.

PROJECT RESULTS

Comments supplied by the participants indicated they felt the pilot workshops were successful. While many of the participants valued the technical information presented, many also valued the workshop's emphasis on the importance of teamwork and communication within and among communities. Many participants also thought the service providers supplied previously unknown information on available technical and financial resources.

The workshop materials developed for use in these pilot workshops have been distributed to the service providers, who have been encouraged to incorporate all or part of the workshop into their ongoing small-community outreach efforts. This Summary Report, which documents the results of the workshops, is another product of the pilot workshops.

To continue the momentum created by the workshops, this report presents information on drinking water and wastewater technologies suited to small-community applications. Six case studies illustrate the use of effective communication, available technical and financial services, and cost-effective technologies for the solution of small-community drinking water and wastewater problems. This report also provides a directory of state and regional locations of the U.S. EPA, NRWA, RCAP, CETC, and USDA-CES.

SUMMARY REPORT DESCRIPTION

The Technology Overview section of this report presents summaries of drinking water and wastewater technologies suited to small communities. These overviews do not include every technology suitable for small-community application. Rather, the overviews present technical and cost information on those technologies most widely used. As every small-community drinking water and wastewater problem is unique, the selection of the most appropriate technology should be based on the site-specific environmental, public health, and financial constraints each small community faces.

The Technology Overviews - Wastewater Treatment summaries present information on collection systems, treatment technologies, and sludge treatment and disposal methods. The Technology Overviews - Drinking Water Treatment summaries present information on the most widely applied treatment technologies. Each technology overview presents a process description and discussions of operation and maintenance requirements, the limitations of the technology, and financial considerations.

The Case Studies section of this report presents case studies that show how six small communities addressed their unique drinking water and wastewater problems. Case studies were selected that best illustrate the use of cost-effective technologies and available technical and financial assistance.

Case Studies - Wastewater presents three case studies:

- West Monroe, New York

  This small community effectively used assistance from the New York State Department of Environmental Conservation's Self-Help Program and the Oswego County Department of Health to evaluate numerous wastewater collection and treatment options. West Monroe selected low pressure sewers and a package treatment plant employing the extended aeration activated sludge process.

- Mapleton, Oregon

  This small community received valuable assistance from the Oregon Department of Environmental Quality, the Lane County Board of Health, and the Oregon Rural Community Assistance Program. Mapleton selected a two-cell recirculating filter to treat wastewater received from a system of new sewers (service lines, laterals, and mains) and new septic tanks.

- Portville, New York

  This village was willing to step back from the process of selecting an appropriate treatment system, evaluate its progress, and change direction based on its knowledge of small community problems. While this case study presents a successful project and many positive experiences, it also illustrates some of the pitfalls that can be encountered by small communities.

Case Studies - Drinking Water presents three case studies:

- Los Ybanez, Texas

  This small community worked effectively with the Community Resources Group, Inc., a private, non-profit rural development organization, to obtain funding and technical assistance. The selection of a reverse osmosis system was largely based on information provided by a neighboring community.

- Westfir, Oregon

  This community obtained assistance from the Rural Community Assistance Corporation and the Lane County Housing Authority and Community Services Agency. Westfir ultimately selected slow sand filters to treat its drinking water. The community also solved other related problems by installing new intake pumps, a sodium hypochlorite disinfection system, and new water supply pumps.

- Mockingbird Hill, Arkansas

  This very small community received financial and technical assistance from the Farmers Home Administration and the Arkansas Rural Water Association. The community eventually selected an air stripping and package precipitation treatment system to deal with ground water with high levels of hydrogen sulfide and dissolved and suspended solids.

The Resource Directory section of this report presents listings of state and regional organizations that can provide a wide variety of technical and financial services to small communities. These organizations can be contacted as small communities address their drinking water and wastewater problems and start to identify the technical and financial resources available to them.

TECHNOLOGY OVERVIEWS – WASTEWATER TREATMENT

PRESSURE SEWERS

TECHNOLOGY APPLICATIONS

Pressure sewer systems provide small communities with an economical alternative to more expensive conventional gravity sewer systems. Pressure sewers are most cost-effective in areas where housing density is low and the terrain is rolling. They can also be effective where the terrain is extremely flat, significant bedrock is present, or the groundwater table is high.

PROCESS DESCRIPTION

The components of pressure sewer systems may be subdivided into the sewer main components and onsite system components. These subsystems are briefly described in the following paragraphs.

Sewer main components typically consist of the following:

- Sewer mains are usually installed using PVC or polyethylene pipe ranging in diameter from 2 to 8 inches. The actual diameter selected depends on the number of homes to be served and the need to maintain a cleansing velocity of 2 feet per second. Pressure mains are installed at a minimum depth of 30 inches. Deeper installations are necessary in colder climates or high traffic locations. Since the small diameter plastic pipe used is somewhat flexible, the sewer mains can be routed around obstacles.

- Isolation valves may be installed almost anywhere in the system. They are usually located at intersections or terminal ends of mains; on long, steep grades; at bridge or stream crossings; or as part of the design of other facilities, such as cleanouts or flowmeters.

- Cleanouts may be installed where pipe sizes change, at the terminal ends of mains, or in service laterals.

- Air release valves are usually installed at high points in the sewer mains to vent excess air that may accumulate at these points. Valves designed for drinking water systems have not worked well in these applications and should not be used.

Two types of onsite systems may be installed: Septic Tank Effluent Pump (STEP) units or Grinder Pumps (GPs). These systems consist of the following:

- STEP systems (see Figure 1) typically consist of a septic tank and submersible pump at each service connection. The septic tanks are usually 1,000- gallon prefabricated concrete or fiberglass units with internal baffles. The tanks must be watertight to prevent the infiltration of groundwater. Grit, settleable solids, and grease are removed from the raw wastewater in the septic tanks. Centrifugal submersible pumps are used to pump the septic tank effluent to the sewer main. STEP pumps may be installed inside the septic tank using an internal vault or in a pump vault external to the septic tank. The pumps may be 1/3 to 1/2 horsepower (HP). The discharge line from the pump is equipped with a check valve and gate valve. Electrical service is required at each service connection. In most installations, three mercury float liquid level sensors are installed in the pump vaults: two to turn the pump on and off and one to trigger a high water alarm.

- GP installations (see Figure 2) do not include a septic tank. Instead, the building sewer is connected to a fiberglass pump vault, 3 feet in diameter with a liquid capacity of 40 gallons. A centrifugal or progressing cavity GP is suspended inside the pump vault. The vault also contains liquid level sensors, as described previously, to operate the pump. GPs shred or reduce the size of wastewater solids, which results in a pumpable slurry. The pumps are usually 1 to 2 HP and require 220 V electrical service. Pump discharge lines to the sewer main are generally plastic, 1.25 inches in diameter, and contain a check valve and gate valve.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

Small systems of approximately 300 homes or less do not normally require a full-time staff. Service can be performed by personnel from the municipal public works or highway department who have received instruction in system O&M requirements. The routine O&M requirements of both STEP and GP systems are minimal. The majority of the system maintenance activities are in response to homeowner service calls. Most of the service calls are due to electrical control problems or pump blockages. STEP systems also require periodic pumping (once every 5 to 7 years) of the septic tank contents.

Due to the inherent septic nature of the wastewater present in pressure sewers, the system personnel must take appropriate safety precautions whenever they un-

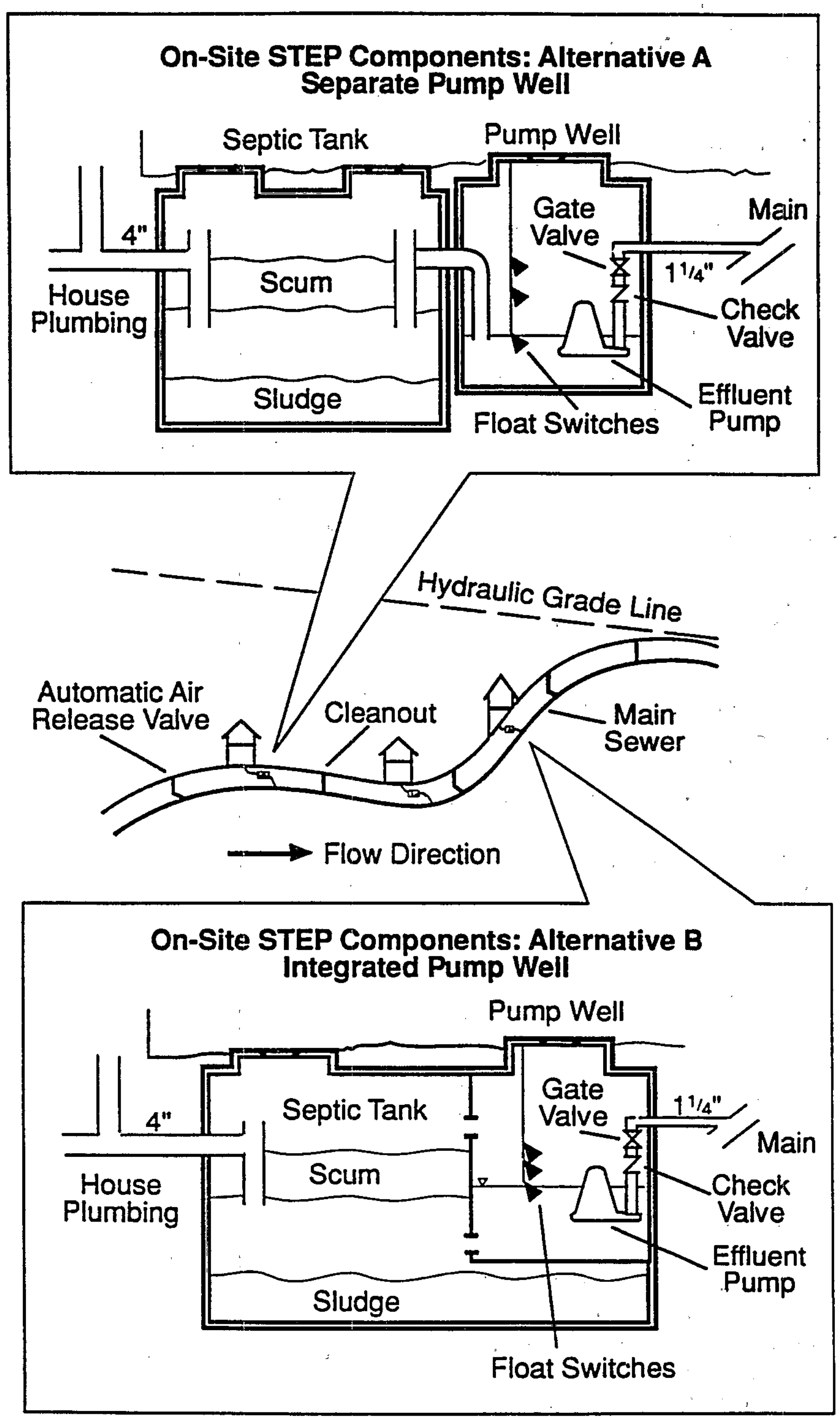

Figure 1. Schematic of a STEP pressure sewer system.

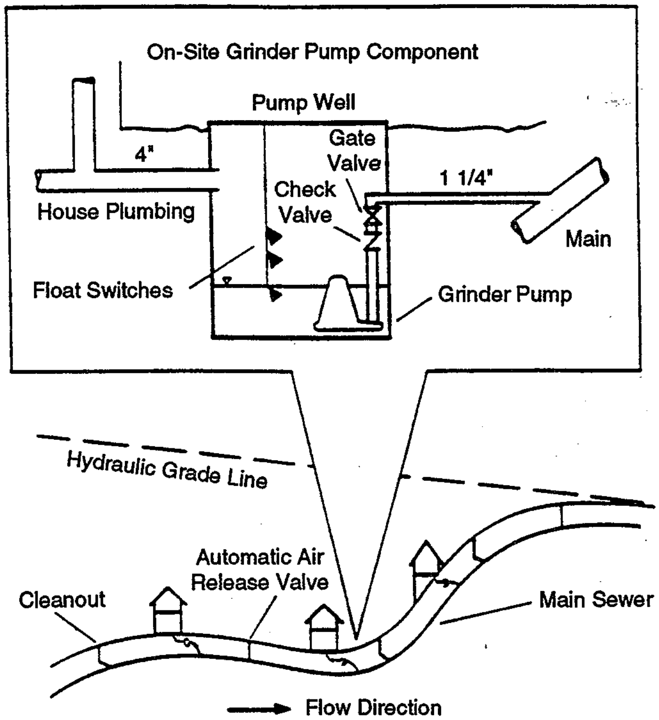

Figure 2. Schematic of a GP pressure sewer system.

dertake maintenance activities to minimize exposure to toxic gases, such as hydrogen sulfide, which may be present in the sewer lines, pump vaults, or septic tanks. Occasionally, odor problems may develop in pressure sewer systems. In many cases, improper house venting has been the cause of odor problems. The addition of chemicals such as chlorine or hydrogen peroxide which are strong oxidizing agents may be necessary to control odor in situations where venting is not the cause of the odor problem.

Generally, it is in the best interests of the municipality and the homeowners to have the municipality or sewer utility responsible for the O&M of all system components. General easement agreements are needed to permit access to onsite components, such as septic tanks, STEP units, or GP units located on private property.

TECHNOLOGY LIMITATIONS

Pressure sewers are usually not as cost-effective as conventional gravity sewers when the treatment plant is at a lower elevation than the collection system and the service area undulations are of low relief. Due to the septic nature of the wastewater generated by these systems, corrosion-resistant materials, such as plastic pipe and fittings, must be used throughout the system. Both STEP and GP systems may experience problems due to interruptions in the electrical power service, pump blockages, or float switch malfunction caused by excessive grease accumulation in the pump vault. STEP systems may also experience sewer main blockages due to the failure to periodically remove sludge from the septic tanks. Excessive infiltration or inflow problems may occur from building sewers, leaking septic tanks or pump vault covers, and connections from building roof and basement drains.

FINANCIAL CONSIDERATIONS

Cost estimates for construction vary widely depending on site-specific conditions. The feasibility of using a community's public works staff and equipment to install all or parts of a pressure sewer system should be considered to reduce installation costs. Accurate O&M costs for pressure sewers have not been well documented. Gross estimates for construction and O&M costs are presented in Table 1. These estimates were prepared by updating cost data presented in EPA's Innovative and Alternative Technology Assessment Manual. Communities planning to install pressure sewers should check with nearby communities using these systems to obtain more reliable cost information.

Table 1. Estimated Construction and O&M Costs for Low Pressure Sewers (1992 $)

Component	Construction Costs ($)	Annual O&M Costs ($)
Sewer Mains (PVC)		175-350/mile
1-3 Inch Diameter	5.30/foot	
4-6 Inch Diameter	6.20-8.20/foot	
STEP Units Septic Tank, Pump, Controls, Service Line, etc.	2,110-4,000	90
GP Units GP, Controls, Pump Vault, Service Line, etc.	2,820-4,140	130

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Innovative and Alternative Technology Assessment Manual. EPA/430/9-78-009, U.S. Environmental Protection Agency, Washington, D.C., 1980.

VACUUM SEWERS

TECHNOLOGY APPLICATIONS

Vacuum sewers can be a cost-effective alternative to conventional gravity sewers under one or more of the following conditions: flat terrain or rolling terrain with small elevation changes, high water table, low population density, or the presence of bedrock at shallow depths. Vacuum sewer systems do not experience the odor and corrosion problems inherent in small diameter and pressure sewer systems.

PROCESS DESCRIPTION

A vacuum sewer system (see Figure 1) has three major subsystems: the central collection station, the collection network, and the onsite facilities. Vacuum is generated at the central collection station and is transmitted by the collection network throughout the area to be served. Sewage from conventional plumbing fixtures flows by gravity to an onsite holding tank. When about 10 gallons of sewage has been collected, the vacuum interface valve opens for a few seconds allowing the sewage and a volume of air to be sucked through the service pipe and into the main. The difference between the atmospheric pressure behind the sewage and the vacuum ahead provides the primary propulsive force. The fact that both air and sewage flow simultaneously produces high velocities which prevent blockages. Following valve closure, the system returns to equilibrium and the sewage comes to rest at the low points of the collection network. After several valve cycles, the sewage reaches the central collection tank, which is under vacuum. When the sewage reaches a certain level, a conventional non-clog sewage pump discharges it through a force main to a treatment plant or gravity interceptor.

The vacuum interface is a unique component of a vacuum sewer system. These valves operate automatically using pneumatic controls. The onsite facilities do not use any electricity. The valve is placed in a valve pit (see Figure 2) buried above the holding tank.

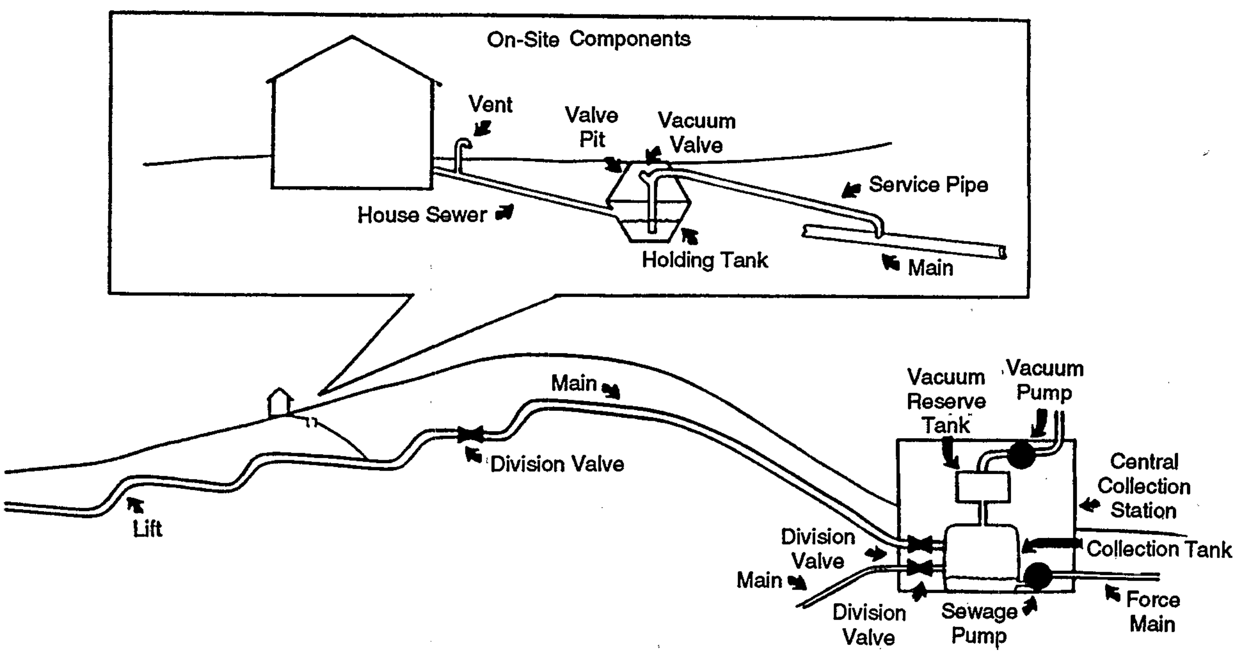

Figure 1. Vacuum sewers.

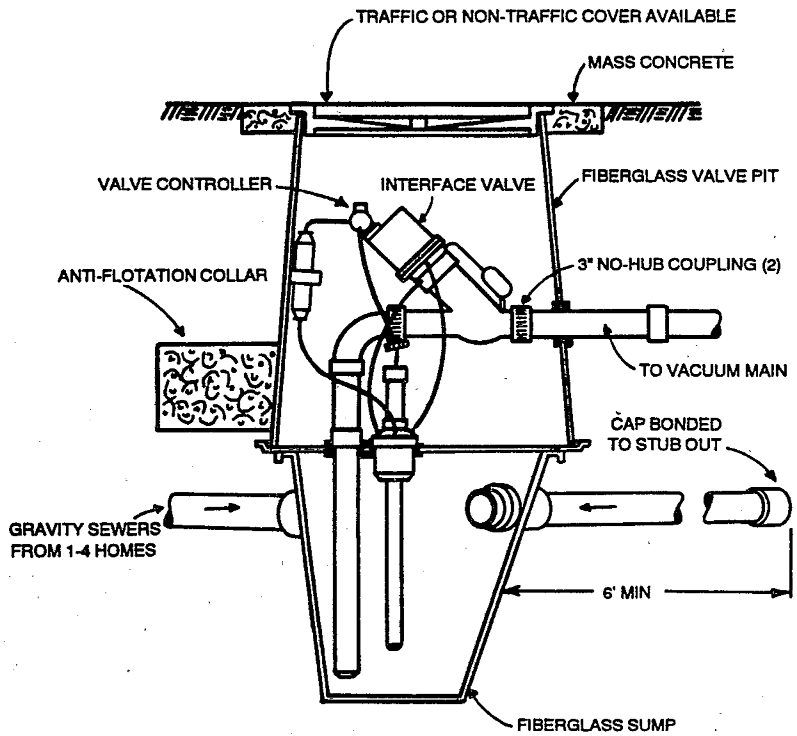

Figure 2. Vacuum sewers.

Plastic pipe is used throughout vacuum sewer systems. The gravity flow house sewer is usually 4-inch diameter pipe. It contains an external vent to admit air when the valve cycles, preventing the house plumbing traps from being sucked dry. Typical service connections are 3-inch pipe; mains range in size from 4 to 10 inches depending on the flow and layout. Joints are either solvent-welded or vacuum-certified rubber ring types.

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 %, the pipe parallels the ground. Otherwise the pipe is laid with a downward slope of about 0.2 % until the depth becomes excessive. When this occurs, a lift formed by two 45-degree elbows and a short length of pipe (see Figure 3) is inserted to gain elevation. The typical lift raises the pipe by 2 feet or less, but higher lifts have been used. Division valves are usually placed at main junctions and at about 1,500-foot intervals to facilitate troubleshooting and repairs. Service lines or tributary mains always join the continuing main from above through a Y-connection.

Several mains may be served by a single collection station. Each main is connected directly to a collection tank through a division valve. Air flows through 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 pumps have been used. Automatic controls cycle the vacuum pumps alternately to maintain the vacuum in the desired range, usually 18 to 23 feet 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.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

Operator training provided by the system manufacturer is critical to proper O&M of vacuum sewer systems. The training program should begin during the construction phase of the project. This will permit the operator to become familiar with all the system components, including the locations of sewer mains, valve pits, division valves, and other key components.

Typical operation of vacuum sewer systems includes daily checks of the vacuum station to monitor sewage pump run times, vacuum pump oil and block temperatures, and vacuum gauge readings. The standby emergency generator should be checked and exercised weekly. At least twice a year, the sewer main division valves should be inspected and exercised. All service connection vacuum valves should be inspected annually and manually cycled

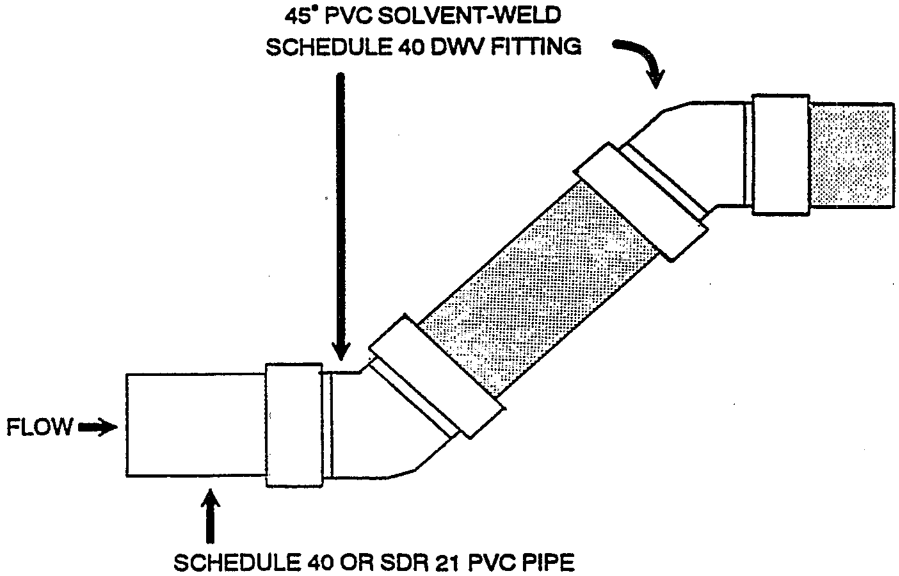

Figure 3. Vacuum sewers.

to verify proper operation. The vacuum valves should be rebuilt once every 5 to 10 years. The seals and diaphragm of the controller/sensor for the vacuum valves should be replaced every 5 years.

TECHNOLOGY LIMITATIONS

Vacuum sewers are not recommended for hilly terrain or areas with significant elevation changes; vacuum sewers have a maximum lift capability of 15 to 20 feet. The installation and O&M requirements of vacuum sewers are more rigorous than small diameter or pressure sewer systems. Energy consumption may also be greater for vacuum sewers than for pressure and small diameter gravity sewer systems. Grease accumulation on control probes located in the vacuum station sewage collection tank may disrupt the vacuum pump and sewage pump operating cycles.

FINANCIAL CONSIDERATIONS

Vacuum system costs are highly site-specific. The following costs are estimates based on a 1989 telephone survey of 32 of 42 U.S. vacuum systems, bid tabulations, and information from manufacturers and design engineers and are in December 1989 dollars (ENR Construction Cost Index=4679). These costs were obtained from U.S. Environmental Protection Agency Municipal Wastewater Treatment Technology Fact Sheets (1990) for vacuum sewers which updates the original material presented in EPA's Innovative and Alternative Technology Assessment Manual.

Based on data from 17 systems, the total construction costs of a vacuum system may range from $7,000 to $18,000 per 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-inch interface valve, pit, cover	$2,000.00
• 4-inch house vent	$60.00
• 4-inch gravity flow house sewer	$5.00/foot
• 3-inch vacuum service pipe	$7.00/foot
• 4-inch vacuum main	$8.00/foot
• 6-inch vacuum main	$11.00/foot
• 8-inch vacuum main	$14.00/foot
• 10-inch vacuum main	$19.00/foot
• 4-inch division valve	$350.00
• 6-inch division valve	$500.00
• 8-inch division valve	$700.00
• 10-inch division valve	$1,000.00
• 4-inch cleanout	$150.00
• 6-inch cleanout	$180.00
• 150-gpm prefabricated central collection station, including building, excluding land	$116,000.00

Accurate O&M costs for vacuum sewers have not been well-documented. However, for planning purposes, the following figures (in February 1992 dollars) may be used to estimate annual O&M costs:

- Annual O&M costs per valve pit $5.32
- Annual O&M cost per central collection station $2,540.00
- Power consumption 250 kWh/yr/customer

The following formula may be used to compute total estimated vacuum sewer system annual O&M costs:

C = 2540*NS + 205*LR*NS + 0.5*LR*NDV + 5.3*NIV + 1.2*LR*NIV + 250*NIV*ER

where:

C	=	Annual O&M costs (February 1992 dollars)
NS	=	Number of central collection stations
LR	=	Labor rate including fringe benefits and overhead (February 1992 $/hour)
NDV	=	Number of division valves
NIV	=	Number of vacuum interface valves
ER	=	Electric power rate (February 1992 $/kWh)

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Innovative and Alternative Technology Assessment Manual. EPA/430/9-78-009, U.S. Environmental Protection Agency, Washington, D.C., 1980.

TECHNOLOGY APPLICATIONS

Small Diameter Gravity Sewers (SDGS) are suitable for low-density residential and commercial developments. SDGS systems collect the effluent from septic tanks and transport the wastewater to a community treatment plant by gravity. Unlike conventional large diameter gravity sewers, SDGS do not require straight alignment and deep excavation and may be installed on a variable gradient provided that there is enough elevation head to maintain flow in the desired direction. SDGS can be installed where the terrain is too flat for conventional gravity sewers.

PROCESS DESCRIPTION

Typical SDGS systems consist of the following components: building sewer, septic (interceptor) tank, service laterals, collector mains, cleanouts, manholes, and vents (see Figure 1). Some systems may also include lift stations when elevation differences do not permit gravity flow in the collection mains. When the collector main hydraulic gradient is greater than a service lateral, a Septic Tank Effluent Pump (STEP) unit may be installed in the individual service lateral to overcome the head differential. A brief summary of SDGS system components follows.

- Building sewers convey raw wastewater from the building to the septic or interceptor tank. To prevent blockages, no smaller than 4-inch diameter pipe should be used.
- Septic tanks are critical components of SDGS systems. They permit the removal of a significant portion of settleable solids and grease from the raw wastewater. Septic tanks also provide for flow equalization and storage of the settled solids. The tanks are typically 1,000-gallon prefabricated watertight concrete or fiberglass units with internal baffles. Larger tank volumes are necessary when multiple dwellings are connected to a single tank.
- Service laterals convey the septic tank effluent to the collector mains. They are typically 3- or 4-inch plastic pipe. The diameter of the lateral should not exceed the diameter of the collector main. Service laterals in low-lying areas may also contain check valves to prevent backups during peak flow periods.
- Collector mains convey the settled wastewater to either a lift station, conventional gravity sewers, or community treatment plant. To resist corrosion from sulfides present in the wastewater, plastic pipe is typically used. It should be at least 3 to 4 inches in diameter. Deep excavation is usually not required, because the mains may be installed with variable or inflective gradients. Where the pipe is not buried below pavement or subject to traffic loadings, the minimum recommended burial depth is 30 inches.
- Cleanouts, or less frequently manholes, provide access to the mains for periodic maintenance and inspection. Cleanouts are usually installed at intervals of 400 to 1,000 feet, at high points, and at upstream terminal sections of the mains.
- Air release valves and vents are required to maintain free-flow conditions in the collector mains. Venting is usually accomplished through the building plumbing stack vent. Air release valves or ventilated cleanouts are installed at high points in the mains.
- Lift stations are installed in the collector main system when elevation differences will not permit gravity flow. STEP units can be installed in service laterals located at lower elevations than the collector mains.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

O&M requirements for SDGS systems are usually minimal, especially if there are no STEP units or lift stations present in the system. Periodic flushing of low-velocity segments of the collector mains may be required. The septic tanks must be pumped periodically to prevent solids from entering the collector mains. It is generally recommended that pumping be performed once every 3 to 5 years. However, the actual operating experience of SDGS systems indicates that once every 7 to 10 years is adequate. Lift stations should be checked on a daily or weekly basis. A daily log should be kept on all operating checks, maintenance performed, and service calls. Regular flow monitoring is useful in evaluating whether inflow and infiltration problems are developing.

The municipality or sewer utility should be responsible for O&M of all of the SDGS system components to ensure a

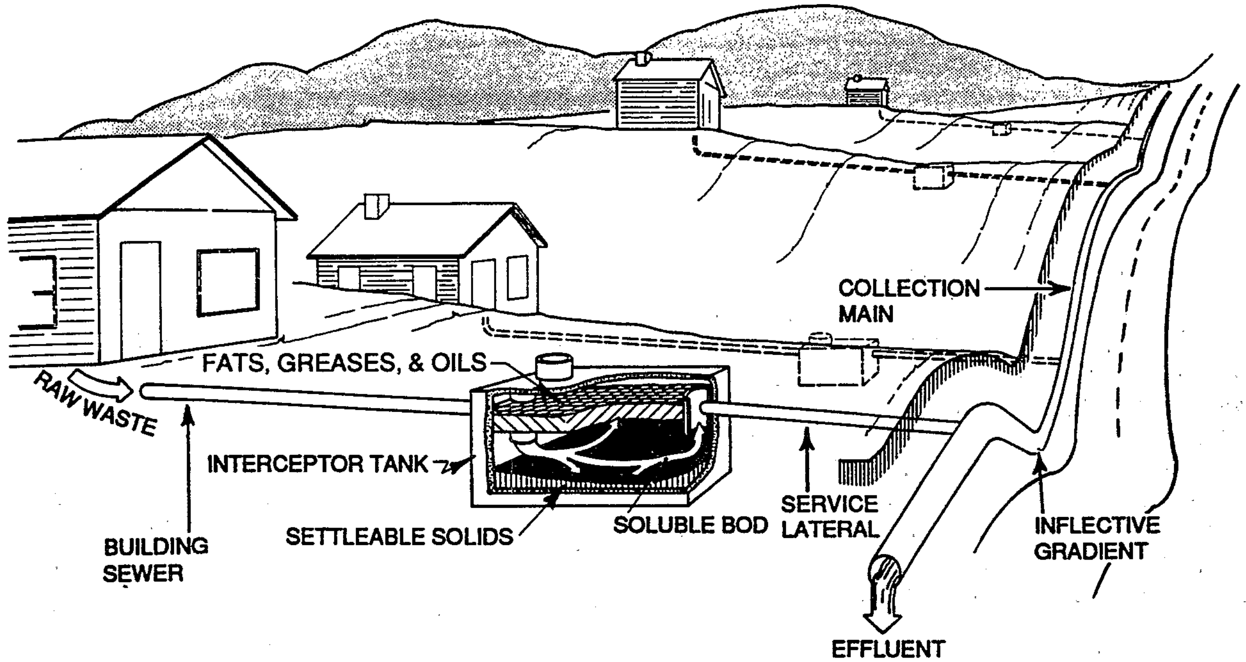

Figure 1. SDGS system.

high degree of system reliability. General easement agreements are needed to permit access to components, such as septic tanks or STEP units located on private property.

TECHNOLOGY LIMITATIONS

SDGS systems may be prone to frequent blockages if the individual septic tanks are not pumped at regular intervals. Connection of existing septic tanks to newly installed SDGS systems is undesirable, since many of the older tanks are not watertight and may also receive flow from building roof and basement drains.

The septic wastewater conveyed by SDGS systems can result in severe odor and corrosion problems. The injection of air or oxygen and chemical oxidizers, such as chlorine, hydrogen peroxide, or potassium permanganate, at various points in the collection system may be necessary to control odor and corrosion. Corrosion-resistant materials, such as plastic pipe, must also be used throughout the system.

In general, SDGS systems are not cost-effective unless the topography allows the sewer profile to stay close to the ground surface without using a large number of lift stations.

FINANCIAL CONSIDERATIONS

Construction costs will vary widely, depending mostly on topography, housing density, and subsurface conditions in the service area. Construction cost figures per foot of main installed have been reported in EPA's Manual on Alternative Wastewater Collection Systems to range from $25.51 to $94.70 (February 1992 dollars). The costs of installing the collection mains and septic tanks usually account for more than 50 percent of total construction costs. Insufficient data exist to estimate O&M costs. However, it has been reported that most systems charge a flat rate of $10 to $20 per month for each connection to pay for administrative, O&M, and financing expenses. Alternatively, planning estimates of $50 per year per septic tank and $430 per year per mile of piping may be used to estimate annual O&M costs as per EPA's Wastewater Treatment Technology Fact Sheets (1990) for small diameter gravity sewers.

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PACKAGE TREATMENT PLANTS

TECHNOLOGY APPLICATIONS

Prefabricated and pre-engineered treatment plants, known as package plants, have been widely used in situations where activated sludge treatment was desired and where flows were generally less than 30,000 gallons per day (gpd). These plants are capable of providing satisfactory treatment of wastewater flow from developments generating normal domestic wastewater with reasonably consistent flow patterns. Skillfully operated package plants can achieve high levels of Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS) removal with effluent concentrations ranging from 20 to 30 mg/L. These systems require the presence of a qualified operator on a daily basis.

PROCESS DESCRIPTION

Activated sludge package treatment plants typically include a bar screen, comminutor, aeration tank, aeration system (diffused air or mechanical), clarifier, and disinfection and sludge handling/disposal components. Primary clarification is not usually employed in package plant designs. Typically, the aeration tank and final clarifier are prefabricated into a dual compartment, circular or rectangular steel tank (see Figure 1).

Within the aeration tank, the wastewater and activated sludge (usually referred to as Mixed Liquor Suspended Solids or MLSS) are mixed and aerated together. Because primary settling is not provided, the aeration tank should have sufficient agitation to keep the heavier solids which are normally removed by primary settling in suspension. The bacteria present in the MLSS feed on the organic content of the wastewater as it flows through the tank. The bacteria use this organic material for food and energy to live and to reproduce. A portion of the organic material is converted to carbon dioxide; the remainder is used to produce new bacteria cells. After the aeration tank, the activated sludge is removed from the wastewater by gravity settling in the final clarifier. The settled sludge is returned to the aeration tank to maintain a sufficient population of bacteria to treat the wastewater. As more wastewater is treated, additional activated sludge is generated. Some sludge must be periodically wasted (removed) from the system to prevent an overload of solids. A solids overload typically results in excessive solids loss from the final clarifier; this, in turn, results in excessive TSS and BOD_5 concentrations in the plant effluent.

Package plants are most commonly designed as an extended aeration activated sludge process. The aeration tanks are normally sized to provide an average hydraulic detention time of 18 to 36 hours. As a result of the long detention time, extended aeration systems generate a somewhat more stabilized sludge than other variations of the activated sludge process.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

O&M requirements are the same as those for extended aeration activated sludge systems. Package treatment plants should be checked daily by a qualified operator experienced in the operation of biological treatment systems. Depending upon the size of the facility, the operator should be present from 2 to 4 hours per day. Adequate time should be allocated for process control, sampling, maintenance, and recordkeeping. To properly operate the treatment facility, the following operating parameters should be monitored daily:

- MLSS concentrations (weekly)
- Dissolved oxygen (DO) levels in the aeration tank
- Final clarifier sludge blanket depth
- Return activated sludge rate
- Waste activated sludge rate

These parameters should be checked against predetermined target values to evaluate the performance of the system. Typical operational target ranges for package treatment plants are as follows:

• Solids Retention Time (SRT)	20 to 30 days
• MLSS	3,000 to 6,000 mg/L
• Food/Microorganism Ratio (F/M)	0.05 to 0.15
• DO	1.5 to 2.5 mg/L
• pH	7.0 to 8.0

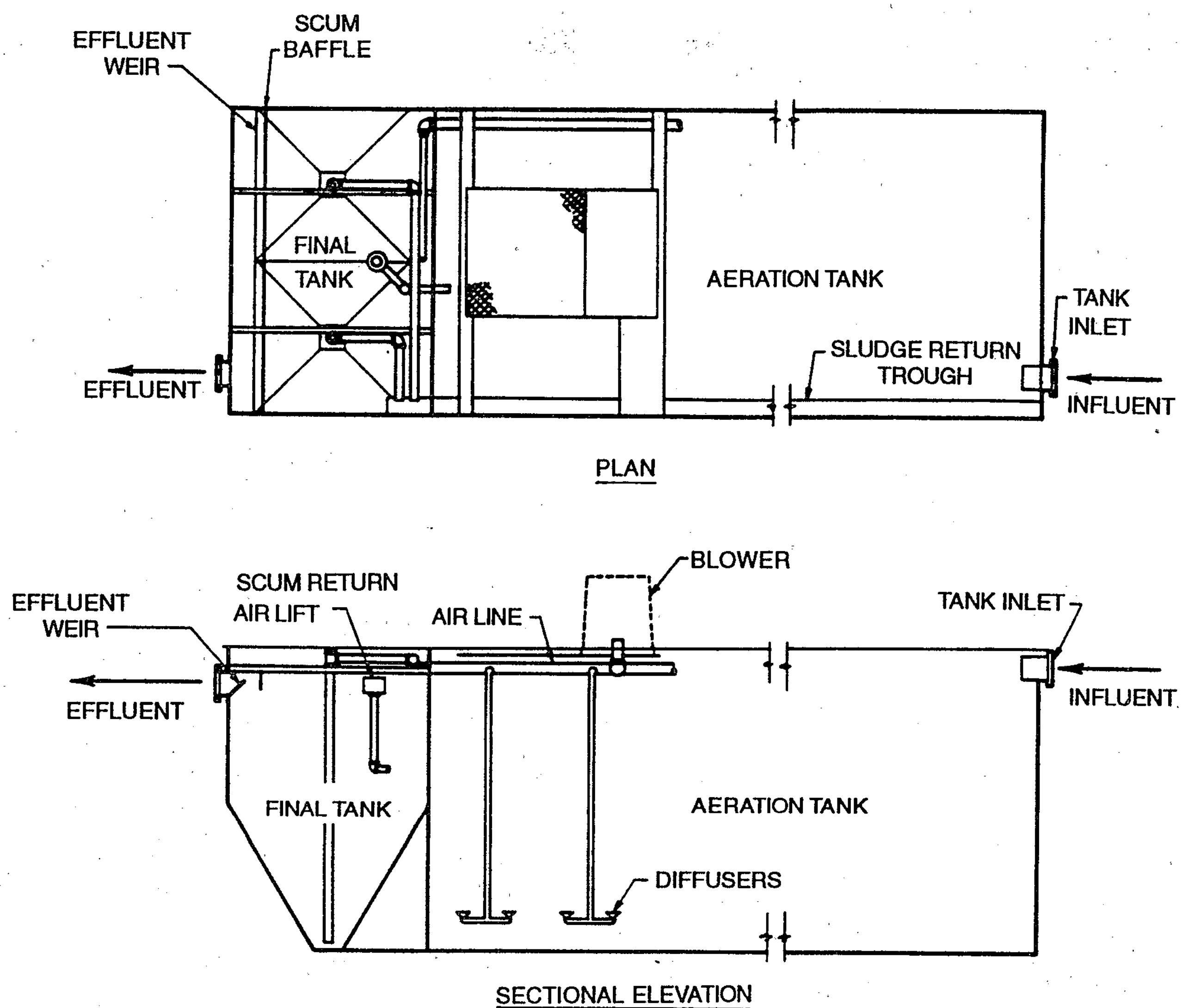

Figure 1. Extended aeration treatment plant with air diffusers.

Regular preventive maintenance (PM) is required to keep the equipment in good operating condition. A formalized PM program should be established based on equipment manufacturer recommendations. This program should include a listing of all equipment, required PM tasks, and the frequency of each task to be performed. Equipment which requires routine maintenance typically includes pumps, air blowers, and diffusers. Most of the package plant clarifiers are equipped with air lifts to pump the settled activated sludge back to the aeration tanks or to sludge handling and disposal equipment. These air lifts tend to clog frequently due to the very dense sludge solids and various debris; therefore, they must be checked daily.

A scheduling system for required tests and process observations should be established. A schedule showing all regular and intermittent maintenance procedures along with emergency procedures should be posted. Access to a complete and relatively easy to understand O&M manual should be provided. This manual should be reviewed by all operating personnel. Adequate training of operators and assistant operators should be provided.

RESIDUES GENERATED

The characteristically long detention times and high sludge age (about 20 to 30 days) of package treatment plants generally result in a sludge low in volatile solids. Sludge is sometimes aerated in a digester or separate aerated storage basin included in the prefabricated plant. Stabilized solids are often dewatered on sand drying beds and eventually disposed of at a landfill or by land application.

TECHNOLOGY LIMITATIONS

Properly operated package treatment plants offer an added measure of performance over other biological processes but are subject to some of the same limitations that other activated sludge treatment processes face. These limitations include hydraulic shock loads due to large flow variations, denitrification in final clarifier causing solids carryover, difficult control of MLSS, upsets due to cold weather, and sensitive operational control. The characteristically long detention time in package plants usually results in the effluent containing significant concentrations of fine suspended solids termed "pin-floc." A filtration unit may be needed to remove the fine suspended solids to meet the permit limitations for BOD_5 and TSS.

FINANCIAL CONSIDERATIONS

The costs for package treatment plants are broken down in Table 1 into construction and O&M costs. The construction costs include screening or comminution, extended aeration package unit, disinfection facilities, and sludge drying beds. They do not include costs of land, engineering, laboratory, legal services, and financing. O&M costs include labor, utilities, chemicals, and maintenance materials.

Table 1. Package Treatment Plant Construction and O&M Costs (1992 $)

Plant Capacity (gpd)	Construction Costs ($)	O&M Costs ($)
10,000	60,000 - 83,000	8,600 - 13,000

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TRICKLING FILTER PLANTS

TECHNOLOGY APPLICATIONS

Trickling filters can be used for the aerobic treatment of domestic and industrial wastewater. Trickling filter plants are capable of achieving a high level of Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS) removal, typically 85-percent removal. Additional treatment may be required to meet stricter discharge standards. When designed for nitrification, trickling filters can achieve further BOD_5 and TSS removal (final concentration less than 30 mg/L). Trickling filters have been popular for use in small systems because they perform well with a minimum of skilled technical supervision and have lower operating costs than activated sludge systems.

PROCESS DESCRIPTION

Trickling filters accept wastewater that has been previously treated by primary sedimentation. The wastewater is applied through a distribution system to a bed of rock or plastic media. As the wastewater trickles down over the media, a bacterial slime forms upon the media, which removes organic matter from the wastewater. The growing slime layer is constantly scoured by the wastewater applied. Once the slime layer has reached its critical thickness, outer portions of the layer begin to slough off and are discharged to the underdrain system. The wastewater and solids collected in the underdrain system are transported to a secondary settling tank where the solids and wastewater are separated. In practice, a portion of the treated wastewater is usually recycled back to the trickling filter. Recirculation often aids in the dilution of incoming wastewater and improves the quality of the final effluent. More equalized hydraulic loads, better distribution over filter media, and less clogging contribute to increased treatment efficiency when recirculation is used.

Trickling filter systems will typically include screens, grit removal tank, primary clarifier, trickling filter, secondary clarifier, disinfection system, and sludge treatment and disposal components (see Figure 1). The trickling filters themselves include a distribution system to apply wastewater to the filter media; filter media to provide a surface area for the growth of microorganisms; and an underdrain system to support the media, provide drainage, and permit the circulation of air for aerobic conditions.

The distribution system usually consists of distributor arms that evenly distribute the wastewater to the media. Most distributors rotate by the reaction force of the wastewater discharging from the distributor arm orifices.

Filter media vary in types and include river rock or granite media varying in size from 3 to 5 inches diameter, redwood media, and plastic media. The depth of rock media filters is usually limited to 5 - 10 feet. Depths for plastic or redwood media vary between 20 and 40 feet.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

Trickling filter plants should be checked daily by personnel experienced in the operation of biological attached growth processes. Adequate time should be allocated for process control, sampling and testing, maintenance, and recordkeeping. Operating checks that should be performed on a daily basis include the following:

- Check distributor orifices, top of filter media, underdrain system, and vent pipes for plugging.
- Check biological growth layer on filter media.
- Monitor recirculation rate.
- Check quality of filter effluent prior to final clarifier treatment.
- Perform sampling and testing as required by discharge permit.
- Check sludge blanket in clarifier.

These parameters should be checked against predetermined target values to evaluate the performance of the system. Typical design and operation parameters for low-rate and high-rate trickling filters are listed in Table 1.

Regular preventive maintenance (PM) is required to keep all equipment in good operating condition. A PM program should be developed based on the equipment manufacturer recommendations. This program should include a list of all equipment, required PM tasks, and frequency of tasks that should be performed. Typical equipment that requires routine maintenance includes rotary distributor bearings, distribution arms, orifices, and recirculation pumps.

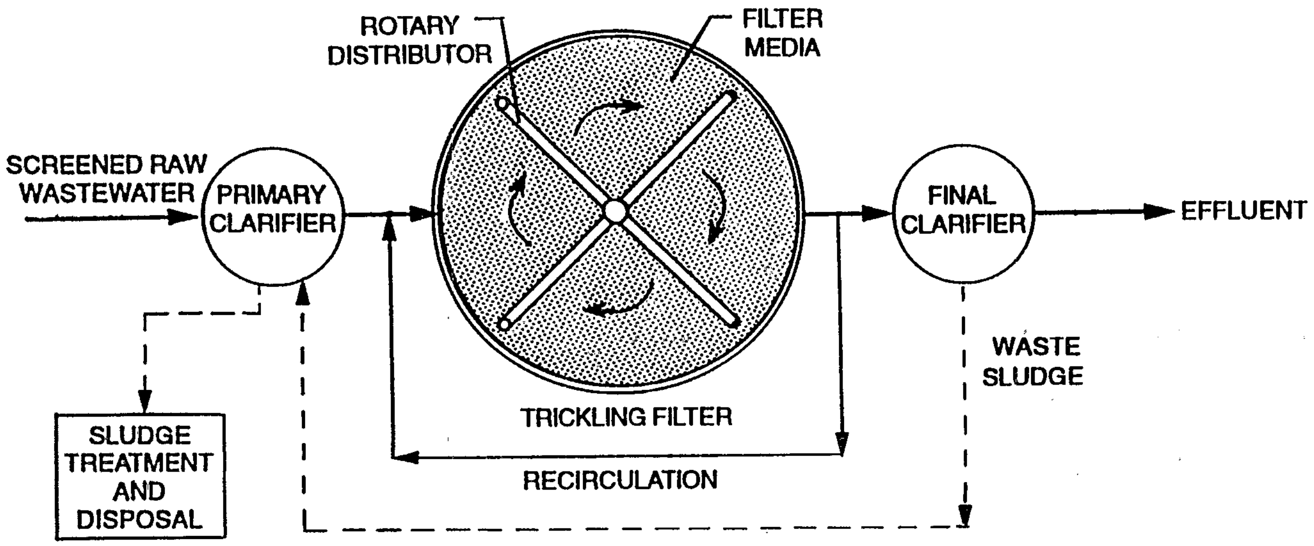

Figure 1. Trickling filter treatment system.

Table 1. Operational Parameters

Parameter	Low-Rate	High-Rate
Hydraulic Loading gpd*/ft^2	25-100	200-1000
Organic Loading lb BOD_5/1,000ft^3	5-25	25-300
Depth, Feet	6-8	15-40
Filter Media	Rock	Plastic

*Gallons per day

RESIDUES GENERATED

Sludge produced by the trickling filter process originates from the primary clarifiers and the biomass or solids that are continually sloughed off from the filter media and collected in the secondary clarifier. The sludge must be digested either aerobically or anaerobically before it is disposed of, usually in a landfill or by land application. Inadequate solids removal from the trickling filter process will result in a poor effluent quality.

TECHNOLOGY LIMITATIONS

As with any biological treatment process, trickling filters are adversely affected by hydraulic and organic overloads. Cold weather also significantly reduces the treatment efficiency of trickling filters. Trickling filters alone are generally unable to meet stringent effluent limitations without the installation of additional treatment processes.

FINANCIAL CONSIDERATIONS

The estimated construction cost as obtained from a single supplier for a trickling filter with a design capacity of 100,000 gpd and consisting of plastic media, steel (bolted) tank, and rotary distributor is $100,000. Costs do not include pumping, clarifiers, engineering, and legal services.

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OXIDATION DITCHES

TECHNOLOGY APPLICATIONS

Oxidation ditch technology is applicable in any situation where activated sludge treatment is appropriate and where the flow is greater than 50,000 gallons per day (gpd). These plants are capable of consistently achieving high levels of Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS) removal with effluent concentrations as low as 10 to 15 mg/L, even in extremely cold climates. High levels of nitrification (95 to 99%) are possible with proper operation. Total nitrogen removals up to 80 percent may be achieved by maintaining oxygen-rich (aerobic) and oxygen-deficient (anoxic) zones around the ditch. Nitrification will take place in the aerobic zones, and denitrification will occur in the anoxic zones. Increased operator attention or automatic control packages are required to produce high levels of nitrogen removal.

PROCESS DESCRIPTION

An oxidation ditch is a variation of the extended aeration activated sludge process that uses a continuously recirculating closed loop channel or channels as an aeration basin. The aeration basin is normally sized to provide an 18- to 24-hour hydraulic detention time. The long detention time provides protection against shock loads and results in high levels of treatment and reduced sludge production.

The components of an oxidation ditch system will typically include screening, grit removal, oxidation ditch, secondary clarification, and sludge handling (see Figure 1). Sludge handling often consists only of a periodically aerated tank, sometimes with a dewatering bed, and rarely includes digestion. Primary clarification is not usually included in the oxidation ditch plant design. The typical oxidation ditch aeration basin is a single channel; multiple interconnected concentric channels can be used for larger systems. The oval configuration or "racetrack" is the most common channel configuration. Other channel configurations which have been used include circular, ell, or horseshoe patterns.

Mechanical aerators are commonly used for mixing, aeration, and circulation of the activated sludge. Generally, these are horizontal brush, cage, or disc-type aerators designed specifically for oxidation ditches. Occasionally, vertical turbine aerators will be used. The aerators must supply the required oxygen to the channel and impart a sufficient velocity in the channel (greater than 1.0 foot per second) to keep the contents in suspension. Oxygen transfer capabilities of an aerator will vary depending upon the particular design. The number of aerators provided depends on the size, configuration, and oxygen requirements of the plant. A minimum of two aerators should be installed.

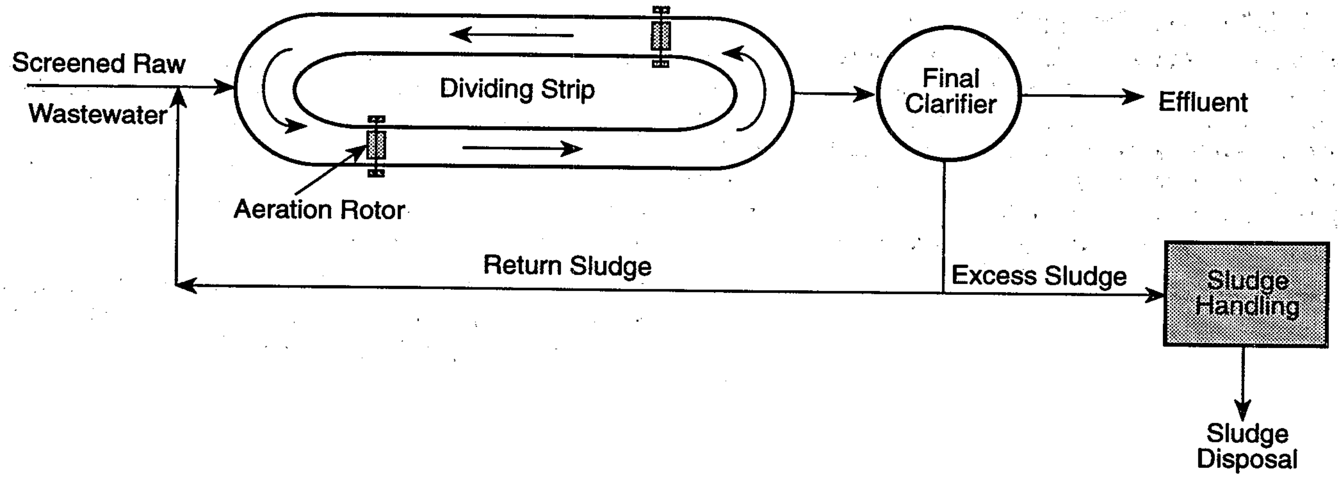

Figure 1. Oxidation ditch flow diagram.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

O&M requirements are very similar to those for extended aeration activated sludge systems. Oxidation ditch plants should be checked daily by personnel who are experienced in the operation of biological treatment systems. Depending on the size of the facility, the operator should be present from 2 to 8 hours per day. Adequate time should be allocated for process control, sampling, maintenance, and recordkeeping. To properly operate the treatment system, the following operating parameters should be monitored at least weekly:

- 30-minute sludge settling volume
- Mixed liquor suspended solids concentration in the ditch
- Dissolved oxygen levels in the ditch
- Final clarifier sludge blanket depth
- Return activated sludge rate
- Waste activated sludge rate

These parameters should be checked against predetermined target values to evaluate the performance of the system. Typical operational target ranges for oxidation ditches are as follows:

- Solids Retention Time (SRT) 10 to 20 days
- Mixed Liquor Suspended Solids (MLSS) 2,000 to 5,000 mg/L
- Food/Microorganism Ratio 0.05 to 0.15
- Dissolved Oxygen (Minimum level) 1.5 to 2.0 mg/L
- pH 6.0 to 9.0

Regular preventative maintenance (PM) is required to maintain the equipment in optimal condition. A formalized preventative maintenance program should be established based upon the recommendations of the equipment manufacturers. This program should include a listing of all equipment, the required PM tasks, the frequency the tasks should be performed, and a record of their performance. Typical equipment which requires routine preventative maintenance includes pumps, aerators, motors, and drives.

RESIDUES GENERATED

The characteristically long detention times and high sludge ages of oxidation ditches generally result in slightly reduced waste sludge production and a sludge with a low volatile content. If the volatile content of the waste sludge is low enough (below 50 percent), the waste sludge may be directly applied to drying beds for dewatering, prior to final disposal.

TECHNOLOGY LIMITATIONS

Oxidation ditches offer an added measure of reliability and performance over other biological processes but are subject to some of the same limitations that other activated sludge treatment processes face. These limitations include plant upsets due to high organic loadings from industries, high hydraulic loadings from excessive inflow and infiltration, changes in the types of microorganisms present, inadequate solids removal, and poor operational control.

FINANCIAL CONSIDERATIONS

The costs for the oxidation ditches are broken down into construction and O&M costs. The capacity of the plants ranges from 50,000 to 500,000 gpd. The construction cost includes the oxidation ditch, clarifier, pumps, building, laboratory, and sludge drying beds. It does not include costs for land, engineering, legal services, or financing. Operation and maintenance costs include labor, utilities, chemicals, and maintenance materials. The cost breakdown is shown in Table 1.

Table 1. Oxidation Ditch Construction and Annual O&M Costs (1992$)

Plant Capacity (gpd)	Construction Costs ($)	Annual O&M Costs ($)
50,000	342,000	38,000
150,000	418,000	53,200
500,000	722,000	79,800

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SEQUENCING BATCH REACTORS

TECHNOLOGY APPLICATIONS

Sequencing Batch Reactors (SBRs) are an excellent alternative to conventional activated sludge treatment plants. SBRs are capable of achieving high levels of Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS) removal (greater than 90 percent) with effluent concentrations as low as 10 mg/L. In addition to BOD_5 and TSS removal, nitrification, denitrification, and phosphorus removal are possible with modifications to the plant operation. SBRs offer additional features applicable to small communities. These features include easy installation, simple operation, lower maintenance than most activated sludge variations, and energy efficiency.

PROCESS DESCRIPTION

SBRs are a variation of the conventional activated sludge treatment system in which equalization, aeration, clarification, and sludge wasting processes are carried out sequentially in the same tank. SBRs consist of a single tank equipped with an inlet for raw wastewater, air diffusers with associated blowers and piping for aeration, a sludge draw-off mechanism at the bottom to waste sludge, a decant mechanism to remove supernatant after settling, and a control mechanism to time and sequence processes. SBRs operate in cycles of five periods carried out in sequence as follows: FILL, REACT (aeration), SETTLE (clarification), DRAW (decant), and IDLE (sludge wasting). These processes are controlled by time to achieve the objectives of operation. They are discussed further in the following paragraphs. Figure 1 shows a typical single cycle.

FILL

The purpose of the FILL operation is to add raw wastewater to the reactor. During the FILL phase, performance standards may require alternating conditions of low and high dissolved oxygen (DO) concentrations. Periods of aeration during FILL are critical to the development of organisms with good settling characteristics. Conversely, periods of zero DO (anaerobic conditions) or low DO (anoxic conditions) are necessary for biological nutrient removal of nitrogen and phosphorus.

REACT

The purpose of the REACT phase is to complete the reactions initiated during the FILL stage. Depending on design type, influent flow may be diverted to another reactor during this phase and aeration continues on a constant basis or influent flow may continue during this period separated by long distances, baffles, etc., in other designs. Organic removal occurs during this stage. Nitrification (ammonia removal) may also occur during this phase if loading is low enough compared to Mixed Liquor Suspended Solids (MLSS) (i.e., high Solids Retention Time [SRT]).

SETTLE

The purpose of the SETTLE phase is to allow solids separation to occur in the system while providing a clarified supernatant to be discharged as effluent. In the SETTLE mode, reactor contents are completely quiescent, eliminating the short-circuiting of continuous flow clarifiers.

DRAW

The purpose of the DRAW phase is to remove the clarified supernatant from the reactor as final effluent. Floating and adjustable weirs are the most popular decanting mechanisms for this phase of treatment, but submersible pumps are also used.

IDLE

The purpose of the IDLE phase is to provide time for one reactor to complete its fill cycle prior to switching to another unit. IDLE is not a necessary phase and can be eliminated. Depending upon the process and treatment goals, aeration, mixing, or sludge wasting can occur during the IDLE phase.

Continuous influent types of SBRs do not have an IDLE phase.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

O&M requirements for SBRs are minimal compared to other conventional activated sludge treatment systems. However, SBR plants should be checked daily by personnel experienced in operating biological treatment systems. Depending on the size of the facility and the com-

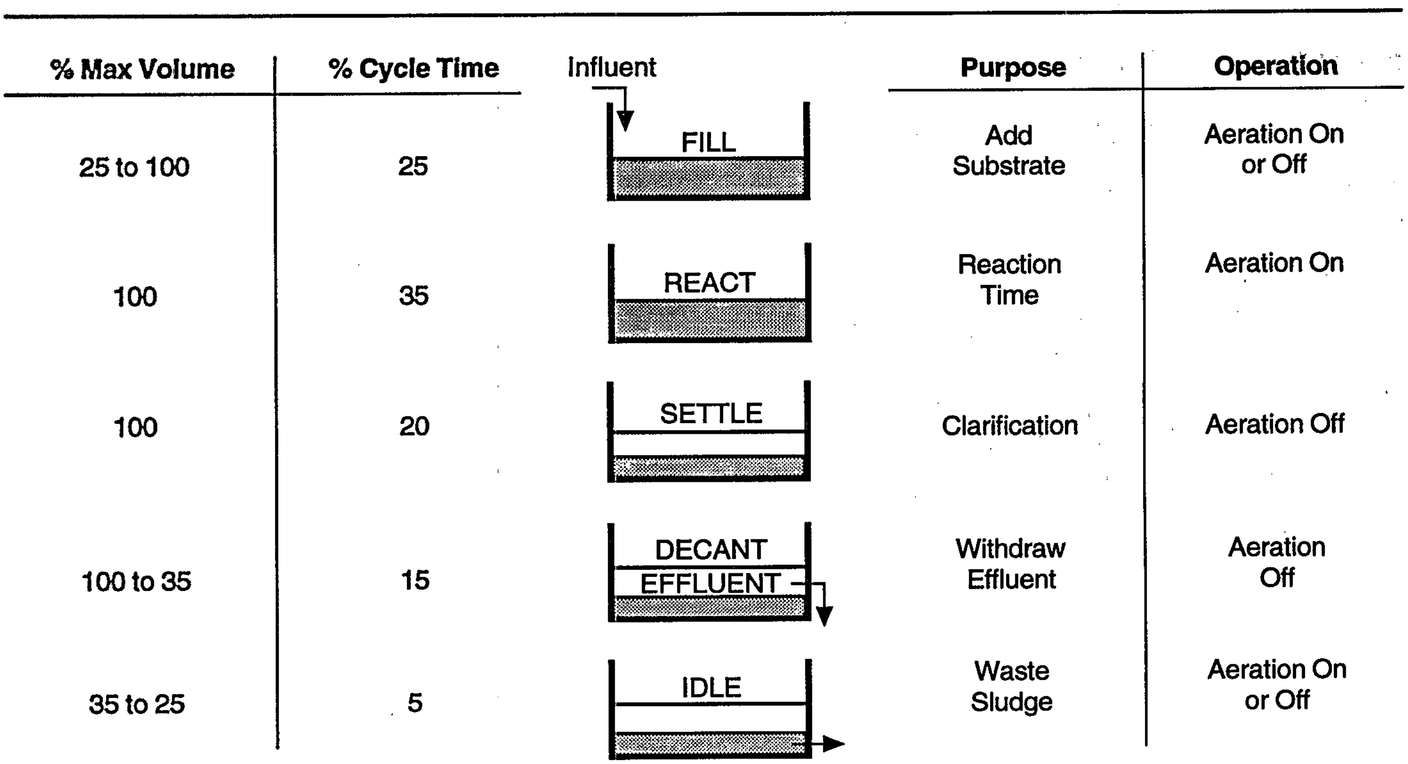

Figure 1. Typical SBR operating sequence.

plexity of treatment processes (i.e., nitrogen and phosphorus removal), the operator should be present from 2 to 8 hours per day. Adequate time should be allocated for process control, sampling, O&M, and daily recordkeeping. Unless these plants receive at least some daily attention and maintenance from a qualified operator, effluent quality will eventually become unsatisfactory. To operate the treatment system properly, the following operating parameters should be monitored at least weekly:

- 30 to 60 minute sludge settling volume
- MLSS concentration
- DO concentrations
- Decant heights

These parameters should be checked against predetermined target values to evaluate the performance of the system. Typical operating target ranges for SBRs are as follows:

- SRT — 20 to 30 days
- MLSS — 2,000 to 6,000 mg/L
- Food/Microorganism Ratio (F/M) — 0.08 to 0.16
- pH — 7.0 to 8.0

Regular preventive maintenance (PM) is required to keep the equipment in optimal condition. A formalized PM program should be established based upon the recommendations of the equipment manufacturers. This program should include a listing of all equipment, required PM tasks, and the frequency with which the tasks should be performed. Typical equipment that requires routine PM includes pumps, blowers, air diffusers, and automatic controllers.

RESIDUES GENERATED

Generally, SBRs generate the same quantities of sludge as extended aeration activated sludge facilities. Excess or waste activated sludge may be typically aerobically digested, dewatered on drying beds, and applied to the land.

TECHNOLOGY LIMITATIONS

SBRs, especially systems designed to remove nitrogen and phosphorus, require the presence of an experienced operator. Performance depends heavily on the reliability of automatic controllers for valves, pumps, aeration systems, and decanting systems. Cold weather will also affect the performance of SBRs.

FINANCIAL CONSIDERATIONS

Construction costs for SBRs treating flows less than 100,000 gallons per day (gpd) were obtained from a single manufacturer. O&M costs were not available. However, in general, SBRs are recognized as being economical to operate and maintain. Estimated construction costs (1992$) for systems having design capacities of 10,000 gpd, 50,000 gpd, and 75,000 gpd are $76,000, $136,000, and $165,000, respectively. The cost of each system includes two SBRs and an aerobic digester of steel tank construction, piping, valves, controls, and a concrete support pad. The estimated construction costs do not include the cost of land, engineering, legal, or financial fees.

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LAGOONS

TECHNOLOGY APPLICATIONS

Lagoons are the most commonly employed wastewater treatment technology, especially for small communities. They provide a simple and economical means of treating a community's wastewater. However, due to increasingly stringent permit limitations, additional downstream treatment processes, such as intermittent sand filters, may be necessary to polish the lagoon effluent. Lagoon systems are also used to pretreat and store wastewater in land application or constructed wetlands systems.

PROCESS DESCRIPTION

Lagoon systems are natural treatment processes in which bacteria and algae reduce the organic content of a community's wastewater. During daylight hours, the algae release oxygen into the lagoon, and the bacteria use this oxygen for respiration. A healthy lagoon will exhibit a green color due to the large algae population that develops. Additional aeration may be provided by wind action or in some cases by a mechanical or diffused aeration system. Facultative lagoon systems operate at very long detention times, ranging from 20 to 150 days depending on design and climate conditions. Aerated lagoons generally require shorter detention times. Lagoons operated in cold climates require longer detention times. As shown in Figure 1, a lagoon system typically includes screening, lagoon, and disinfection components.

Lagoon systems for domestic wastewater treatment are typically categorized into two types: stabilization ponds or aerated lagoons. In stabilization ponds, aerobic conditions are maintained by algae and wind action. The ponds must be kept shallow (3 to 5 feet deep) to permit the adequate mixing needed to maintain aerobic conditions. Stabilization ponds operate best with detention times in excess of 30 days. As a result of their shallow depths and long detention time requirements, stabilization ponds require large amounts of land, usually 1 acre for every 200 people served. To avoid organic overload, ponds should be sized to treat a Biochemical Oxygen Demand (BOD_5) loading of 15 to 50 lbs/acre/day, depending on climate.

Aerated lagoons can treat a much higher organic loading than stabilization ponds. This is due primarily to the supplemental aeration process. Air is introduced into the lagoons via mechanical surface aerators or subsurface diffusers. Aerated lagoons require one-tenth to one-third of the land required for stabilization ponds and have shorter detention periods (3 to10 days in warm climates). The average depth of an aerated lagoon is between 6 and 10 feet. Shorter detention times and increased depths reduce the land requirements. This is an attractive alternative where land costs are high or large amounts of land are unavailable.

Lagoon systems are usually designed with a minimum of three separate cells or basins connected in series. Sometimes, the first cell is larger than the other cells and receives the bulk of the organic load. The remaining two cells act as polishing ponds or settling basins for suspended solids removal. Algae cells compose most of the

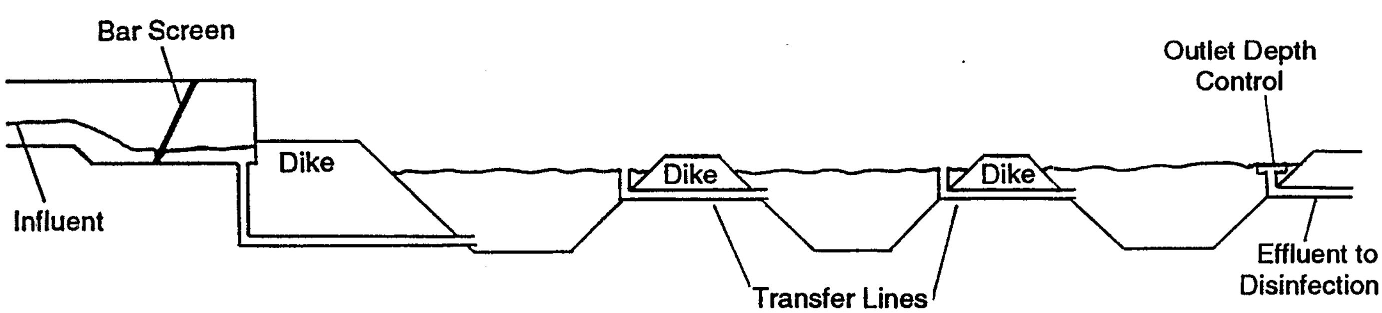

Figure 1. Wastewater treatment lagoon system.

suspended solids that must be removed from lagoons. In aerated lagoon systems, the last cell in a series is usually left unaerated to provide for suspended solids removal. Additional downstream treatment, such as intermittent sand filters, may be necessary to produce an effluent containing less than 10 mg/L BOD_5 or Total Suspended Solids (TSS). Floating baffles may also be used to updgrade an existing large single-cell lagoon into several smaller cells to prevent short-circuiting.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

As a result of their simple design characteristics, lagoons have few O&M requirements. The O&M of a lagoon normally involves daily site inspections; routine site maintenance, such as grass cutting, and erosion and aquatic weed control; and periodic sample collection and testing. Typically, a full-time maintenance staff is not required. Many small communities use their public works staff to operate and maintain the lagoon system. This type of arrangement can work, provided the community officials permit the public works staff to be trained properly and to spend sufficient time at the treatment facility. In addition to routine monitoring activities, the lagoons should be checked at least once each year for sludge accumulation. Excessive sludge deposits in lagoons may result in reduced detention times and increased BOD_5 and TSS levels in the effluent. Sludge may need to be removed every 5 to 10 years depending on actual site conditions.

Operating parameters should be monitored weekly and sometimes daily in order for proper treatment to occur. Operating checks should include the following:

- Check dikes for erosion, leaks, and signs of burrowing animals.
- Keep inlet and outlet structures clear of obstructions.
- Check pond surface and outlet structures for scum build-up.
- Monitor pond for temperature, color, pH, dissolved oxygen (DO), and suspended solids.
- Monitor screening devices.
- Sample effluent and test as required by permit.
- Monitor condition of mechanical equipment.

A Preventive Maintenance (PM) program should be initiated for all equipment according to manufacturer recommendations. Records should be kept for the influent and effluent BOD_5, TSS, and pH. Generally, for lagoons, the pH ranges from 6.5 to 10.5. Pond conditions including DO, solids content, and sludge depths should also be recorded. The lagoon DO should be greater than 1.0 mg/L, and it should be checked at several locations within the lagoon.

RESIDUES GENERATED

During the normal course of treatment, algae and bacteria cells eventually die and settle to the bottom of the lagoon along with incoming setteable solids forming a sludge layer. Since there is little or no oxygen at the bottom of the lagoon, the sludge is partially stabilized by anaerobic digestion. Eventually the sludge accumulation in the lagoon becomes excessive and must be removed. Depending on state regulations, the sludge may be disposed of at landfills or by land application. The sludge should be tested for toxic pollutants, such as heavy metals, before it can be applied to the land.

TECHNOLOGY LIMITATIONS

Lagoons are low-cost and simple biological treatment systems suitable for domestic wastewater treatment where strict discharge limits are not imposed. Additional downstream treatment processes must be installed to provide further reductions in TSS concentrations, especially during the warmer months when algal activity increases. Lagoon systems have far greater land requirements than mechanical treatment systems. This may restrict their application in communities where the cost of land is prohibitive or where insufficient land is available. Like most biological treatment systems, lagoons are adversely affected by cold weather and organic and/or hydraulic overloads.

FINANCIAL CONSIDERATIONS

The costs of lagoons are broken down into construction and O&M costs in Table 1. The construction costs for a lagoon usually depend on excavation prices. The construction costs include excavation of the lagoon, associated aeration equipment, pumps, piping, and disinfection. They do not include costs for land, legal and engineering services, or financing. O&M costs will vary with location as a function of labor and utilities required for treatment. O&M costs include labor, utilities, and maintenance materials.

Table 1. Aerated Lagoons Construction and Annual Operational Costs (1992$)

Plant Capacity (gpd)	Construction Costs ($)	Annual O&M Costs ($)
10,000	47,000 -71,000	4,300 - 6,500
100,000	118,000 - 353,000	17,200 - 25,800

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INTERMITTENT SAND FILTERS

TECHNOLOGY APPLICATIONS

Intermittent sand filters are most often used to provide further treatment of lagoon or septic tank effluent. These filters are ideally suited for populations of less than 1,000 people and where flow is less than 100,000 gallons per day (gpd). These facilities are capable of consistently achieving high removals of Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS) removal with effluent concentrations normally in the range of 5 to 10 mg/L. High levels of nitrification (90 to 95%) can also be realized with proper operation. Intermittent sand filters are especially applicable to small communities due to low cost and minimal operator requirements.

PROCESS DESCRIPTIONS

Intermittent sand filters (see Figure 1) are variations of fixed film biological treatment systems. They consist of two types of processes: single-pass (where wastewater travels through the filter once) and recirculating (where the wastewater travels through the filter several times). Single-pass filters can be further divided into buried and open types. Recirculating filters are similar in design to the open types except a coarser media is used and a portion of the filtered effluent is piped back to the dosing tank and reapplied to the filter.

Basically, an open intermittent sand filter consists of a bed of sand, usually 30 to 36 inches deep, resting upon a layer of gravel containing an underdrainage system of open joint/perforated pipe. The filter floor (usually earthen) is graded and sloped to provide necessary drainage. Side walls extend approximately 18 inches above the sand filter surface. The total bed area is usually divided into two or more smaller filters. Each individual filter is dosed on an alternating cycle. This allows the filter to drain completely after each dose which is necessary to maintain aerobic conditions.

The recirculating intermittent sand filter utilizes a recirculation tank where most of the discharge from the sand filter is diverted and mixed with pretreated wastewater to be reapplied to the filter. This system dilutes the wastewater stream being applied to the filter while improving filter performance and decreasing clogging. A recirculating filter normally operates in the range of 2.0 to 3.0 gpd/ft^2. Open single-pass filters are operated at rates as high as 10 gpd/ft^2.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

This section pertains to open and recirculating sand filters since buried filters are not readily accessible. The hydraulic loading to the filters should be checked daily. To maintain aerobic conditions, the filters should drain completely before the next dosing cycle begins. Hydraulically overloaded filters will typically exhibit septic (anaerobic) conditions and reduced effluent quality. For recirculating filters, a recirculation ratio of 3:1 to 5:1 should be maintained for proper operation. The dosing interval should be adjusted so that each dose results in about 2 to 4 inches of wastewater uniformly distributed across the filter.

Maintenance of sand filters is primarily concerned with keeping the filter inlet distribution channels clear of any debris, cleaning or raking the upper layer of the filter bed, and replenishing sand lost from the filter bed. The filter surface should be cleaned whenever the top layer of sand begins to show signs of clogging. Weeds and grasses should not be allowed to grow in the filter. Sand should be replaced periodically to maintain a sand depth in excess of 24 inches.

RESIDUES GENERATED

Some sludge will be generated whenever the surface of the filter is cleaned. The sludge and sand removed should be properly disposed of in accordance with state regulations.

TECHNOLOGY LIMITATIONS

Intermittent sand filters are easily upset by excessive hydraulic loading. These filters are also unable to handle excessive solids levels in the influent wastewater. Cold weather may also adversely affect intermittent sand filters; sometimes, removable covers are installed to prevent freezing. Intermittent sand filters have intermediate land requirements, lower than land treatment systems but much more than mechanical plants.

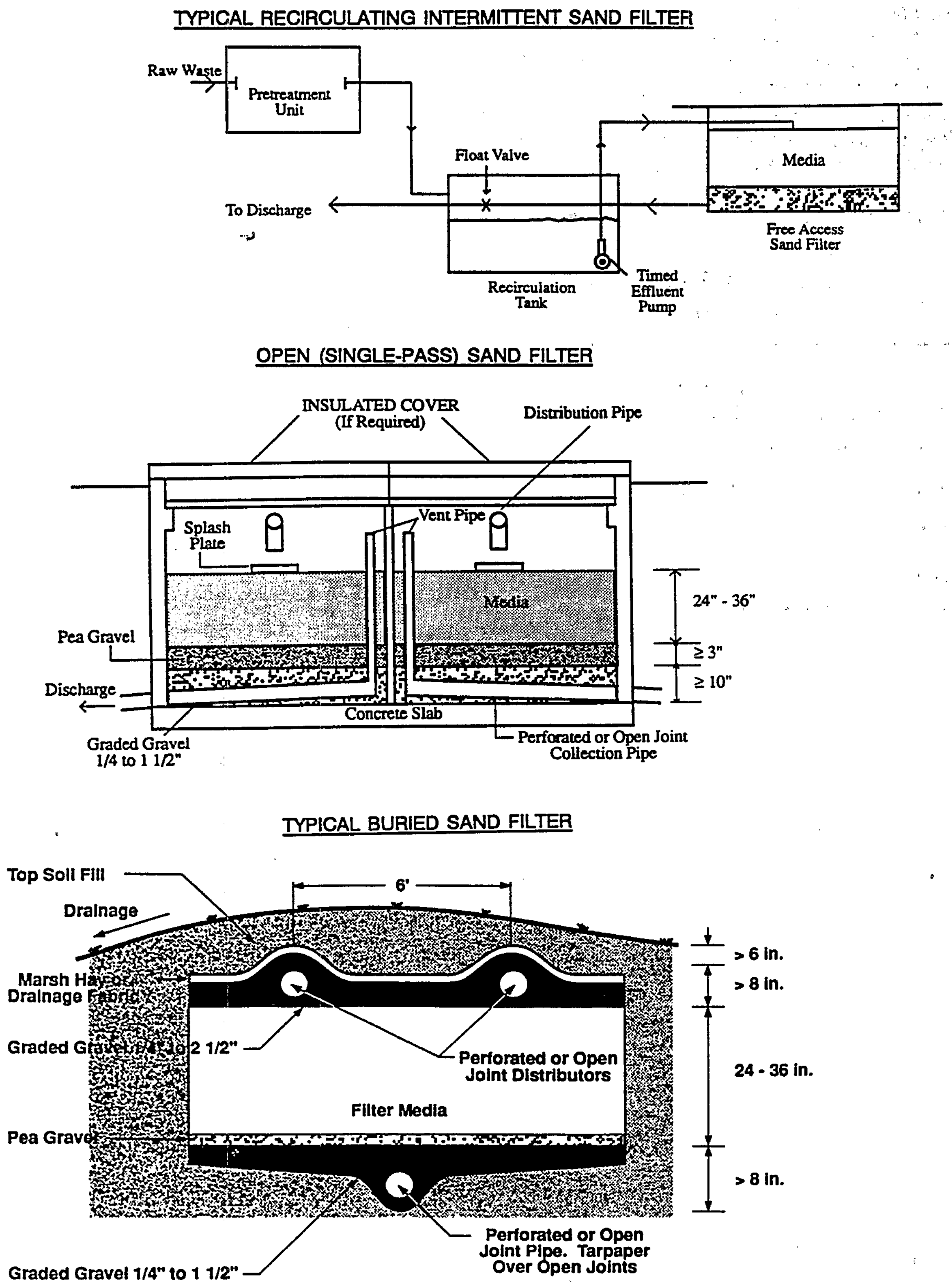

Figure 1. Schematic showing the three types of intermittent sand filters.

FINANCIAL CONSIDERATIONS

The costs for intermittent sand filter systems are broken down in Table 1 into construction and O&M costs. The capacity of the plants ranges from 10,000 to 100,000 gpd. The construction costs include the following components: concrete, sand, gravel, distribution system, and pumps. They do not include the cost of land, engineering, legal and financing, pretreatment, and disinfection costs. O&M costs include labor, utilities, maintenance materials, and associated chemicals.

Table 1. Intermittent Sand Filters Construction and O&M (1992 $)

Capacity (gpd)	Construction Costs ($)	Annual O&M Costs ($)
10,000	42,000	3,500
50,000	200,000	10,000
100,000	450,000	12,500

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TECHNOLOGY APPLICATIONS

Land application systems can be utilized to provide further treatment of secondary effluents and/or a means of ultimate effluent disposal. Benefits of land treatment include nutrient recovery, cash crop production, groundwater recharge, and water conservation (if used for irrigation of landscaped areas). These systems are highly desirable in areas where surface water discharge requirements are strict and land is relatively inexpensive. Soil characteristics play a significant role in the siting of land treatment systems.

PROCESS DESCRIPTIONS

In land treatment systems, pretreated wastewater is applied to the site by either overland flow (OF), rapid infiltration (RI), or slow rate irrigation (SRI) methods. Treatment is provided by natural processes as the wastewater effluent flows through the soil and vegetation. Organic material is removed within the top inch of soil. Nitrogen is removed primarily by plant uptake in SRI and by nitrification-denitrification in OF and RI systems. Phosphorus is removed by adsorption in varying degrees by OF, SRI, and RI systems. A portion of the wastewater is lost to evaporation and evapotranspiration; the remainder returns to the surface waters via overland flow or to the groundwater via percolation during irrigation. The major components of land application systems are summarized in Table 1. Land treatment methods are discussed in the following paragraphs.

In the overland flow method, wastewater is applied at the top of a gently sloping (2 to 8%) hill and allowed to flow over the surface of the ground to the bottom of the hill where it is collected, disinfected, and discharged to the receiving water (see Figure 1). The suspended solids in the wastewater settle out and attach themselves to surface vegetation where they are decomposed. Organic matter is consumed by bacteria on grass and soil, while nutrients are absorbed by the grasses, which are harvested periodically. The effluent produced generally exceeds requirements of most secondary treatment systems.

Overland flow systems have been effectively used to treat preliminary or primary effluent, as well as secondary effluent. Overland flow systems provide a high level of treatment with minimal use of mechanical equipment. Typical removal and treatment levels to be expected are listed below:

Parameter	Percent Removal
Biochemical Oxygen Demand (BOD_5)	85 to 92
Total Suspended Solids (TSS)	85 to 92
Nitrogen	60 to 80
Phosphorus	20 to 50

For spray irrigation, wastewater is pumped through a network of pipes to the various fields. The effluent is applied utilizing sprinklers or spray nozzles. The sprinklers are either fixed or mounted on a rotary distributor.

In spray irrigation systems, the rate at which the effluent is applied depends on the weather, stage of plant growth, and soil drainage characteristics. Since wastewater can-

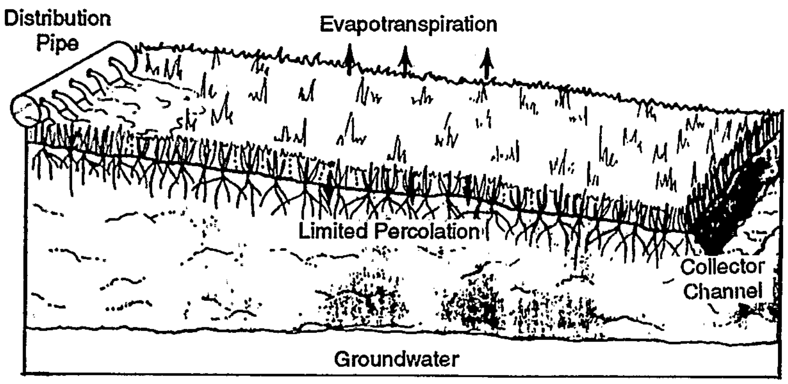

Figure 1. Overland flow method.

Table 1. Components of Land Application Systems

- Preliminary or secondary equivalent treatment
- Transmission to the land application site
- Wastewater storage for non-irrigation periods
- Distribution network over the irrigated area (SRI only)
- System to recover the treated wastewater (OF only)
- Crop harvesting system

not be applied continuously, there must be a provision for storing effluent for periods of as long as 90 days. Usually, this is accomplished using lagoons or stabilization ponds. Wastewater can be applied to cultivated croplands, orchards, pastures, meadowlands, and woodlands. Wastewater is removed by percolation and evapotranspiration (see Figure 2). Typical removal levels from secondary effluent by irrigation are provided below:

Parameter	Percent Removal
BOD_5	90 to 99
Chemical Oxygen Demand (COD)	80 to 90
TSS	90 to 98
Nitrogen	75 to 95
Total Phosphorus	95

For rapid infiltration systems, pretreated wastewater is applied to highly permeable soils by distributing in basins (see Figure 3). Additional treatment occurs by filtration, adsorption, and microbial action as the wastewater percolates through the soil matrix. Wastewater is continuously applied to the basins for periods lasting from several hours to one week. During the resting period, wastewater continues to drain through the soil matrix. Alternating periods of flooding and drying maintain the infiltration capacity of the soil matrix. Vegetation is usually not planted in RI systems. Depending on the system design, treated wastewater either percolates to the groundwater or collects in underdrains for reuse or surface discharge. Of the three land treatment methods, RI systems have the lowest land requirements. Typical removal and treatment levels to be expected are listed below:

Parameter	Percent Removal
BOD_5	85 to 99
TSS	85 to 99
Nitrogen	50
Phosphorus	70 to 95

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

O&M requirements are nominal and typically consist of the following elements: monitoring of wastewater application rates, pump maintenance, pipeline maintenance, and crop harvesting. Pump maintenance involves routine lubrication and inspection and equipment repair as required. For permanently installed pipelines and spray irrigation systems, pipeline maintenance is minimal and involves the occasional flushing of lines or draining of the system at the end of the irrigation season. Since spray nozzles will become clogged with solids, they should be checked and cleaned frequently.

In RI basins, a biological mat forms on the surfaces of the infiltration areas which reduces the infiltration rate. The surfaces should be disked or harrowed to break up the mat. This should be performed at regular intervals or whenever it is observed that routine periods of drying do not restore infiltration rates to acceptable levels.

Once an operating schedule is established (rate and duration of wastewater application), operation becomes routine. However, daily attention by the operator is critical. Automatic timers can be placed on pumps to eliminate the daily responsibility of turning the system on

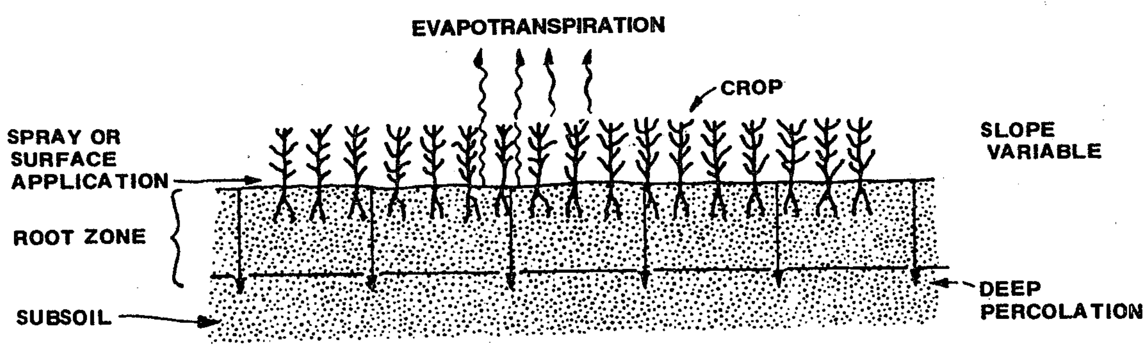

Figure 2. Schematic of spray irrigation.

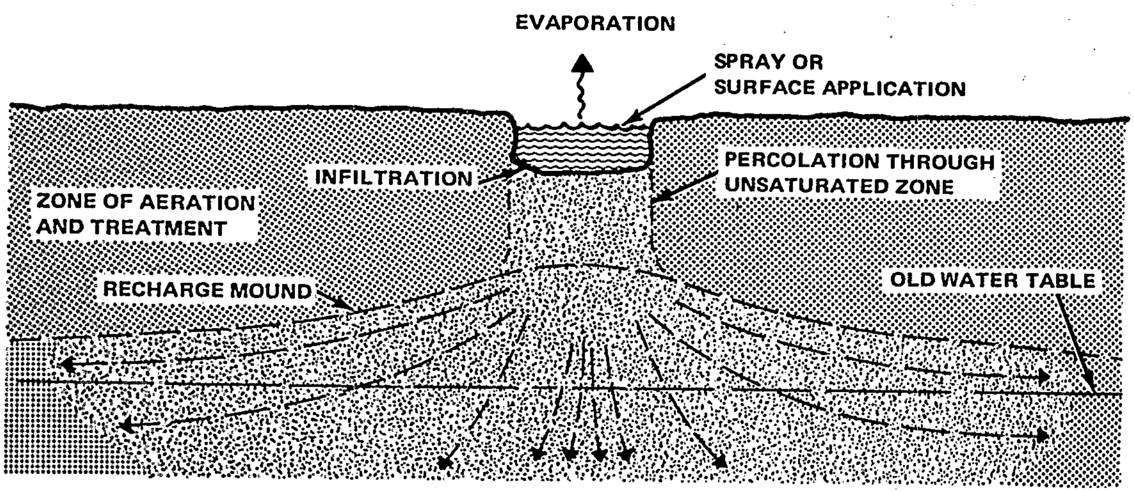

Figure 3. Rapid infiltration.

Table 2. Comparison of Design and Operating Parameters, Land Treatment Systems

Parameter	Irrigation	Overland Flow	Rapid Infiltration
Weekly Application Rate (inches)	0.5 to 4.0	2.4 to 6.0	4 to 96
Annual Application Rate (feet)	2 to 18	8 to 40	20 to 410
Estimated Land Required for 100,000 gpd (acres)	20 to 25	5 to 10	1 to 7
Minimum Preapplication Treatment Requirements	Lagoons	Screening and Grit Removal	Lagoons
Climate Restrictions	Storage Needed for Cold and Wet Climates	Storage Needed for Cold and Wet Climates	Cold Weather May Reduce Hydraulic Loading Cycles
Slope	<20%	Smooth Slopes of 2-8%	Not Critical
Soil Permeability	Slow to Moderate	Impermeable (clays, silts, soils with impermeable barriers)	Rapid

or off. Highly skilled operators are not usually required, but these systems should be checked by personnel who have experience in agricultural techniques. Adequate time should be allocated for process control, sampling (if required), maintenance, and recordkeeping.

Crop management may be one of the most important operating requirements for OF and SRI systems. The application rate of the wastewater must be regulated to match not only crop requirements (especially for nitrogen), but soil drainage properties as well. Application rates must also be regulated to suit changing weather conditions. Excessive application rates can result in crop failure, erosion, groundwater contamination (irrigation systems), surface runoff (irrigation systems), and short-circuiting (overland flow).

When evaluating the performance of the treatment systems, common design features for proper operation should be observed. These are presented in Table 2.

RESIDUES GENERATED

Land treatment systems generate residuals (rags, grit) and sludge from the associated preapplication treatment processes. Periodically, grass crops must be mowed and harvested from the application sites. The frequency of harvesting will vary depending on climate conditions but generally should be at least two to three times per year if substantial nutrient removal is desired.

TECHNOLOGY LIMITATIONS

Land treatment systems are limited by soil type, topography, climate (cold weather and annual precipitation), crop selection, and land availability. In addition to the preceding factors, spray irrigation systems may also be affected by wind conditions, spray nozzle clogging, and reduced infiltration rates caused by sealing of the soil. High flotation tires are required for mowing and harvesting equipment used in overland flow systems. These systems may also require preapplication disinfection depending on state regulations.

Groundwater quality may be impacted by nitrate contamination, especially for rapid infiltration systems. RI systems may also be adversely impacted by inadequate TSS removals by upstream treatment units.

FINANCIAL CONSIDERATIONS

The construction and O&M costs for spray irrigation and overland flow land treatment system methods are presented in Tables 3 and 4, respectively. The capacity of the plants ranges from 10,000 to 100,000 gpd. The construction costs for spray irrigation systems include expenses for pumps, distribution piping, and fixed rotary sprinklers. Costs for pretreatment facilities, storage lagoons, and land are not included. The construction costs for overland flow systems pertain to a complete system, including disinfection and discharge facilities; land costs are not included.

Table 3. Typical Construction and O&M Costs for Spray Irrigation (1992 $)

Plant Capacity (gpd)	Construction Costs ($/gpd)	O&M Costs ($/1,000 gal)
10,000	5.50 to 10.95	0.92 to 1.83
100,000	1.83 to 5.50	0.19 to 0.52

Table 4. Typical Construction and O&M Costs for Overland Flow Systems (1992 $)

Plant Capacity (gpd)	Construction Costs ($/gpd)	O&M Costs ($/1,000 gal)
10,000	7.20 to 14.40	1.46 to 2.92
100,000	1.46 to 2.92	0.37 to 0.73

The estimated construction and annual O&M costs for a 100,000 gpd rapid infiltration system is reported in the literature to be $79,200 and $8,000, respectively. The construction costs do not include pretreatment or storage facilities.

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SUBMERGED BED CONSTRUCTED WETLANDS

TECHNOLOGY APPLICATIONS

Constructed wetlands (CW) treatment technology is applicable to situations where direct stream discharge is either prohibited or restricted. Wetlands systems may also be utilized as polishing or tertiary treatment units, accepting effluent from prior treatment facilities (i.e., lagoons). The capacities of these systems range from 500 gallons per day (gpd) to 1 million gallons per day (MGD) with the great majority being less than 100,000 gpd. Submerged bed systems can produce an effluent which contains low concentrations of Biochemical Oxygen Demand (BOD_5) and Total Suspended Solids (TSS), typically below 20 mg/L.

PROCESS DESCRIPTION

As shown in Figure 1, submerged bed constructed wetlands systems typically consist of single or multiple channels or trenches with impermeable linings. The channels permit plug-flow conditions. Wastewater is distributed to the channels by perforated or grated pipe. The channels contain a layer of gravel to support the growth of emergent vegetation such as cattails, rushes, and reeds.

The majority of the wastewater BOD_5 and suspended solids are reduced by either sedimentation or filtration as the wastewater passes through the beds. The treated wastewater is typically discharged from the channel by an outlet pipe which can be adjusted to vary the depth of the water level in the channel. A general criterion of 5 m^2/person is presently employed to establish the needed bed area.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

Operating requirements for constructed wetlands are minimal. The water level in the channels may need to be periodically adjusted to regulate the detention time in the system. Excessive detention times can result in an unnecessary capital expense. Shorter detention times can result in inadequate treatment. It is important that the wastewater is equally distributed to all of the channels. In general, harvesting of the wetlands vegetation is not practiced, but newer designs suggest better treatment may result from such practice. The inlet sections of the channels should be checked regularly for excessive solids buildup or the accumulation of debris.

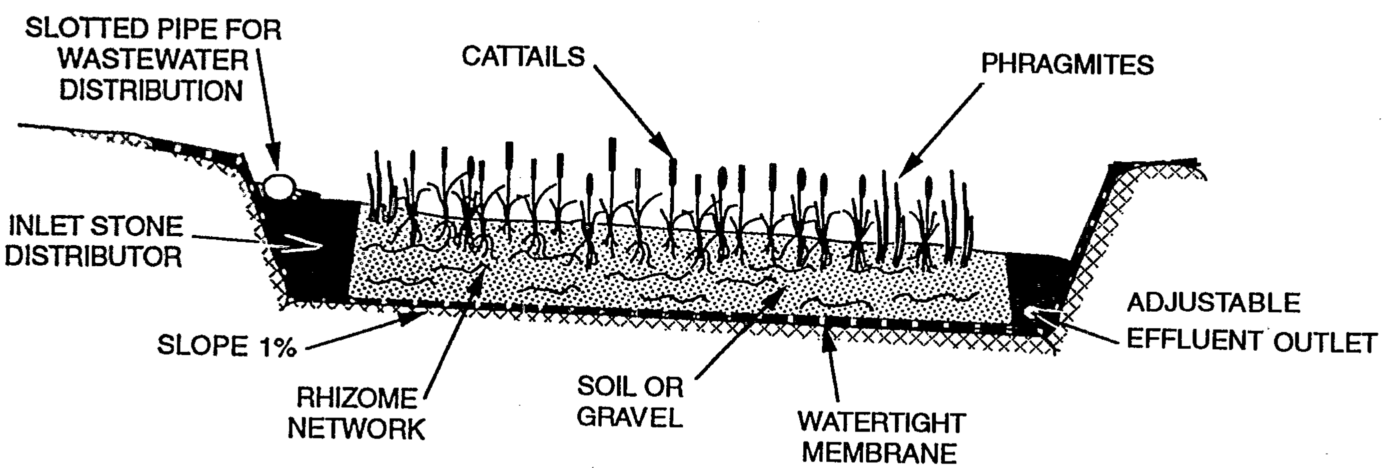

Figure 1. Submerged bed constructed wetland.

The influent and effluent should be monitored for process control and permit requirements. The BOD_5 loading to the system should be checked periodically to avoid an organic overload. Generally, primary or septic tank treatment should precede the submerged wetlands.

RESIDUES GENERATED

Constructed wetlands systems do not generate sludge as do conventional biological treatment systems. However, the aquatic vegetation may need to be periodically harvested. The ultimate service life on a CW system prior to solids overloading is unknown.

TECHNOLOGY LIMITATIONS

Constructed wetlands should not be used to treat raw wastewater. Most U.S. applications have been limited to the treatment of secondary effluents. Pretreatment of raw wastewater is necessary to prevent organic overload and to prevent excessive accumulations of solids, especially at the inlet ends of the channels.

FINANCIAL CONSIDERATIONS

Due to a wide variety of designs and existing site conditions, the costs of constructing and operating wetlands treatment systems vary greatly. Since land requirements are significant, they are an important factor in site selection.

The costs for wetland treatment systems are broken down in Table 1 into construction and O&M costs. The construction costs include the inlet and outlet structures, gravel filter bed, pretreatment, and final disinfection. They do not include the costs of land, engineering, laboratory, legal services, or financing. O&M costs include labor, utilities, chemicals, and equipment maintenance.

Table 1. Typical Costs For Wetlands Treatment Systems (1992 $)

	Construction Costs ($/gpd)	O&M Costs ($/1000 gal.)
Artifical Wetlands System	0.58 - 2.36	0.12 - 0.58
Pretreatment	1.18 - 3.53	0.58 - 1.18
Disinfection	0.88 - 1.18	0.24 - 0.35

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TECHNOLOGY APPLICATIONS

Several of the wastewater treatment processes previously described (e.g., oxidation ditches, trickling filter plants, package treatment plants, SBRs and lagoons) generate sludge that must receive proper treatment and disposal. Before disposal, sludge must be stabilized to remove pathogens and reduce the organic content. After stabilization, some communities dewater the sludge to reduce its total volume for disposal. Small communities should dispose of their stabilized sludge by application to the land wherever possible.

PROCESS DESCRIPTIONS

Stabilization processes, dewatering, and land applications practices are briefly described in the following paragraphs.

Stabilization

For small communities, sludge stabilization is typically accomplished by aerobic digestion or lime application. In the aerobic digestion process, excess sludge from oxidation ditches or package plants is pumped to an uncovered and unheated aerobic digester. The sludge is retained in the digester for 20 to 30 days (three to four times longer for cold climates) to reduce the Volatile Suspended Solids (VSS) content and pathogens. The digester contents are aerated and mixed during the digestion period. In the lime application method of sludge stabilization, lime is added to raw or digested sludges. Enough lime must be added to the sludge to raise the pH to greater than 12 to reduce pathogens. For raw sludge, the pH should be raised to 12.4 and then kept above 11 for 14 days to stabilize the sludge, as well as to kill pathogens.

Dewatering

Uncovered sand drying beds are commonly used by small communities to remove excess water from sludge prior to disposal. Some beds consist of an impervious clay bottom or liner upon which underdrain piping is placed. The underdrain piping is covered by 6 to 12 inches of graded gravel. The upper layer of the bed contains 12 to 18 inches of sand. In normal use, approximately 8 to 12 inches of sludge are applied to the entire bed surface. Some of the liquid drains from the sludge, collects in the underdrains, and is returned as influent to the wastewater treatment plant while the rest evaporates. Alternative designs, such as vacuum-assisted, chemically assisted, asphalt bottoms, may also be applicable.

Land Application

Beneficial sludge constituents include nitrogen, phosphorus, potassium, and certain trace metals that act as fertilizer nutrients, and organic material that serves as a soil conditioner. Therefore, sludge can be an excellent supplement to commercial fertilizers and soil amendments. The method by which the sludge is applied to land and the application rate depend on the characteristics of the sludge and soil, as well as the type of crop. Three categories of crops are usually grown: agronomic or row crops, forage crops and grasses, and forested systems. Liquid sludge can be applied to either the land surface (by spreading or spraying) or to the land subsurface (by injection, disking, or plowing). Dewatered sludge cannot be pumped or sprayed and is typically spread over the land surface and then plowed or disked into the soil.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

The following paragraphs discuss O&M requirements for the various processes presented above.

Stabilization

Aerobic digesters are relatively simple to operate and maintain. The process should be monitored at least weekly for dissolved oxygen (DO), pH, and VSS reduction. The DO should be kept between 1.0 and 2.0 mg/L. The pH in the digester should be between 6.5 and 8.0. A properly operated digester should achieve a VSS reduction of at least 40 percent after 15 to 20 days in warm climates. Cold weather operation may require much more time to achieve adequate VSS reduction. Periodically, the air supply is turned off to allow the sludge to settle. The supernatant is then decanted to the head of the plant. The sludge may also be removed at this time if adequate VSS reduction has been achieved.

Dewatering

O&M requirements for sand drying beds primarily involve dried sludge removal from the bed and maintenance of the sand beds. In most small plants, dried sludge is removed manually and represents a very large labor demand. A small quantity of sand is lost each time sludge is removed and must be replaced periodically. The surface of the bed must be raked and kept level. Weeds and grasses must also be removed from the beds.

Land Application

The land application of sludge requires good coordination between the treatment plant management and farming operations management. The scheduling of sludge applications should not interfere with farming operations (i.e., planting, tilling, harvesting) but should meet the removal requirements of the treatment plant. The application rate should be in accordance with any state permit limits to prevent excessive accumulation of nutrients or heavy metals in the soil. All site runoff should be contained. Cold and wet weather will generally prevent land application operations. The quality of the sludge must be tested periodically for nutrients, minerals, heavy metals, and pesticides.

TECHNOLOGY LIMITATIONS

The technology limitations for stabilization, dewatering, and disposal are highlighted in the following paragraphs.

Stabilization

Aerobic digesters are energy-intensive operations. They are adversely affected by excessive solids loading, cold weather, and low pH (< 6.0).

Dewatering

The efficiency of sand drying beds is significantly affected by wet weather, freezing temperatures, intense sunlight, and poor maintenance practices. Holding tanks are necessary to store sludge during periods of inclement weather.

Land Application

Land application of sludge is subject to crop management requirements and climate conditions. Holding tanks or lagoons are necessary to store sludge. For dedicated land disposal, land may have to be purchased or leased; buildup of metals in the soil may limit future use of the land. Odors and site runoff may limit land application operations. Prior to application, sludge must first be treated by a stabilization process, such as digestion, sand drying beds, composting, or lime stabilization.

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TECHNOLOGY OVERVIEWS – DRINKING WATER TREATMENT

COAGULATION/FLOCCULATION

TECHNOLOGY APPLICATIONS

Coagulation and flocculation processes are used to prepare non-settleable (colloidal) and dissolved solids for removal from a drinking water supply. Colloidal and dissolved solids may adversely affect the quality of the drinking water supply and make it unsuitable for use due to the presence of turbidity, color, taste, heavy metals, or microbiological contamination. Coagulation and flocculation processes are installed upstream of sedimentation or filtration processes; colloidal and dissolved solids are not removed by sedimentation or filtration processes alone.

PROCESS DESCRIPTION

During the coagulation step, chemicals referred to as coagulants are added to the water in a rapid mix tank (see Figure 1). Rapid mixing is important to ensure uniform distribution of the coagulant throughout the water. The coagulant neutralizes the electrical charge on colloidal particles; this permits the small particles to begin to stick together to form larger particles. Examples of coagulants are aluminum sulfate (alum) and ferric sulfate. Polymers are sometimes added to improve the coagulation process. The rapid mix step generally takes place in 30 to 60 seconds within a small tank equipped with a mechanical mixer. Hydraulic jumps or in-line static mixers have also been used to mix the coagulating chemicals and water.

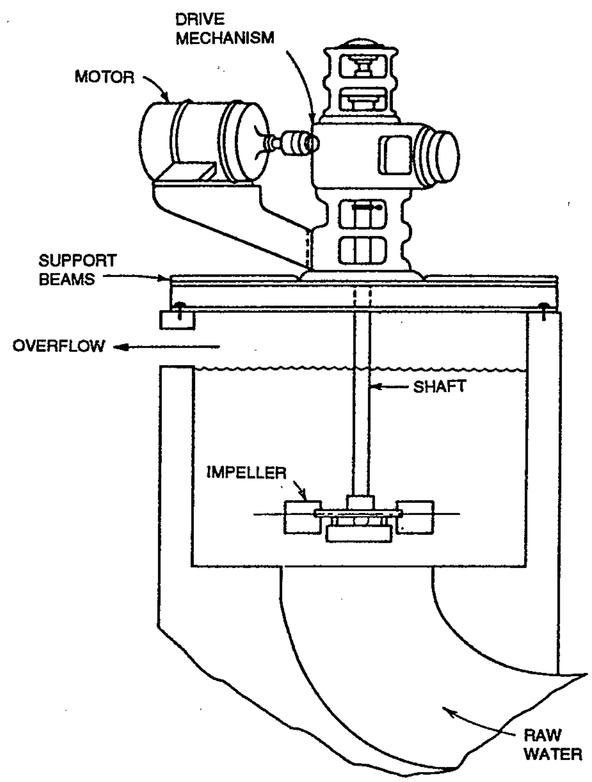

Figure 1. Rapid mix tank.

Immediately following the rapid mix tank is a flocculation basin (see Figure 2). During flocculation, the coagulated water is gently mixed in a basin for a period of 30 to 60 minutes. The gentle mixing allows the suspended particles to collide and form heavier particles called floc. The floc particles can be ultimately removed from the water by gravity settling and/or filtration.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

The proper operation of coagulation/flocculation processes requires regular monitoring by the operating personnel. The monitoring frequency will be determined by actual site conditions, i.e., size of treatment system and variability of raw water quality. Parameters to be monitored regularly and recommended monitoring locations are presented in Table 1. The operator should also visually check at least daily the floc particle size and distribution in the flocculation basin to maintain adequate mixing rates. Adequate mixing is demonstrated by the presence of a well-formed floc and uniform distribution of the floc throughout the flocculation basin. Records of the treated water flowrate, chemical feed pump settings, and chemical feed rates should also be maintained.

The optimal coagulant feed rate depends on the type of coagulant used and the raw water quality and temperature. Typical dosages for alum range from 15 to 100 mg/L; ferric sulfate dosages range from 10 to 50 mg/L. Lower water temperatures require higher coagulant dosages. Operators should periodically perform jar tests to determine the optimal coagulant dosage.

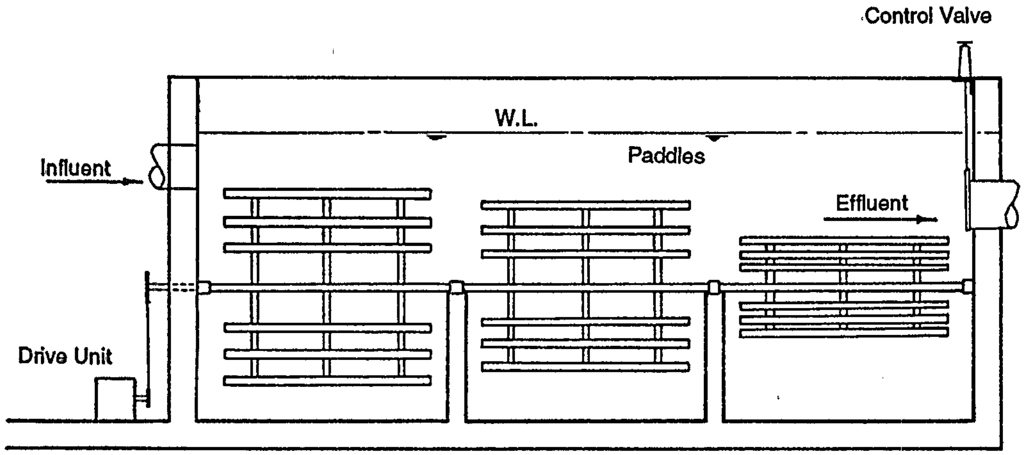

Figure 2. Flocculation basin with horizontal paddle wheel agitators.

Table 1. Recommended Process Monitoring for Coagulation / Flocculation Processes

Parameter	Location
Turbidity	Raw water
Temperature	Raw water
pH	Raw water, flocculator effluent
Alkalinity	Raw water, flocculator effluent
Color	Raw water

TECHNOLOGY LIMITATIONS

Coagulation/flocculation processes are adversely affected by low water temperatures, insufficient alkalinity, inadequate mixing, and a highly variable raw water quality. Continuous, uniform raw water flowrates are necessary for good performance. Low-turbidity waters are difficult to treat and may result in inadequate floc formation. Well-trained operators are needed for reliable operation.

FINANCIAL CONSIDERATIONS

The costs for coagulation/flocculation systems include the basic chemical feed system (mix tank, mixer, metering pump, valves, and piping), the rapid mix system, and the flocculator. Construction cost estimates for chemical feed systems range from $1,830 to $11,030. Annual O&M costs will be dependent on the chemical used and the application rate. Estimated annual O&M costs range from $2,620 to $10,950. The costs presented above are for systems with maximum chemical feed rates of 10 lbs/day up to 1,000 lbs/day.

For small community systems with flows less than 100,000 gallons per day (gpd), the estimated construction costs for adding rapid mix facilities range from $23,000 to $31,800. The O&M costs do not vary significantly for flows under 100,000 gpd and are estimated to be approximately $5,000 per year.

Estimated construction costs for flocculators range from $17,600 to $32,000 for systems treating flows under 100,000 gpd. O&M costs are estimated to be $2,000 per year.

Costs presented in 1992$.

For most small communities, coagulation/flocculation systems are included as part of a package treatment system also consisting of sedimentation and filtration. Costs of package treatment systems are presented under the filtration overview.

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SEDIMENTATION

TECHNOLOGY APPLICATIONS

In sedimentation basins or clarifiers, settleable solids are removed from raw water by gravity settling. Non-settleable (colloidal) solids can also be removed from water by sedimentation with the aid of coagulants added to upstream coagulation/flocculation treatment units. The coagulants convert the colloidal solids to large, dense, and settleable flocs which are removed by settling in sedimentation basins. Sedimentation also helps to reduce the solids loading to filtration systems which results in longer filter runs.

PROCESS DESCRIPTION

Sedimentation may take place in rectangular, circular, or square basins or tanks. The tanks are sized to provide sufficient time for gravity settling to take place. Inlet structures are provided to reduce the velocity of the water and to distribute the water uniformly across the tank. The outlet or effluent section of sedimentation tanks contain weirs that control the exit velocity of the clarified water to prevent short-circuiting. Sludge collection mechanisms may also be provided to remove the settled solids from the bottom of the basin.

Modifications to conventional sedimentation basins have included the installation of tube or plate settlers and the solids-contact process. The installation of tube settlers in existing sedimentation basins results in improved settleable solids removal efficiencies. In many cases, the flow capacity of existing basins has been increased by 100 percent following the installation of tube settlers. Tube or plate settlers are made of plastic and placed in modules which are 2 to 3 feet in length. The tubes or plates are installed at an incline about 60 degrees from the horizontal. As water flows upward through the modules, solids settle out on the plates and eventually move downward and into the basin from which they are removed.

Solids-contact basins or upflow clarifiers (see Figure 1) combine coagulation, flocculation, and sedimentation into a single basin. Coagulation and flocculation take place in the reaction zone where the water and the coagulating chemicals first enter the basin. The flocculated solids are allowed to settle and accumulate around the reaction zone. Water exiting the reaction zone passes through the accumulated solids (or "sludge blanket") which act as a filter trapping smaller floc particles. After passing through the sludge blanket, the water flows up and over the weirs located at the top of the clarifier. A portion of the settled solids is removed from the clarifier for disposal while the remainder is recycled back to the reaction zone. The recycled sludge speeds up the coagulation/flocculation process and reduces coagulant dosage requirements.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

Sedimentation basins should be checked daily to ensure adequate solids removal. Typically, the raw water and individual basin effluent should be checked for turbidity. The temperature of the raw water should also be monitored since low water temperatures will reduce the settling rate of particles. During cold-weather operation increased dosages of coagulants may be needed or the flowrate may be reduced to increase the detention time in the basin. The overflow weirs must be kept clean and level to prevent short-circuiting and poor solids removal.

The sludge blanket level should also be checked daily in all operating basins. Sludge should be withdrawn frequently from conventional sedimentation basins to prevent excessive accumulations of sludge. Excessive accumulations result in significant solids loss from the basins and blinding of downstream filtration processes. Solids-contact clarifiers require careful monitoring of the sludge blanket for the following reasons: (a) a sufficient blanket level must be maintained to ensure that all water passes through the blanket prior to exiting the clarifier; and (b) the sludge removal and recirculation rates are adjusted based on the level of the sludge blanket.

Sedimentation basins should be drained annually for inspection. Inlet baffles should be cleaned of any algae or solids accumulation. Effluent weirs must be level. Basins equipped with tube or plate settlers may need to be drained more frequently to flush out accumulations of solids.

TECHNOLOGY LIMITATIONS

Depending on the design characteristics, some sedimentation basins may be upset by sudden increases in flow rates. Low water temperatures may result in reduced solids removal due to a reduction in settling rates or the production of density currents which cause short-circuiting. Inadequate inlet baffles or uneven effluent weirs can result in poor solids removal due to short-circuiting.

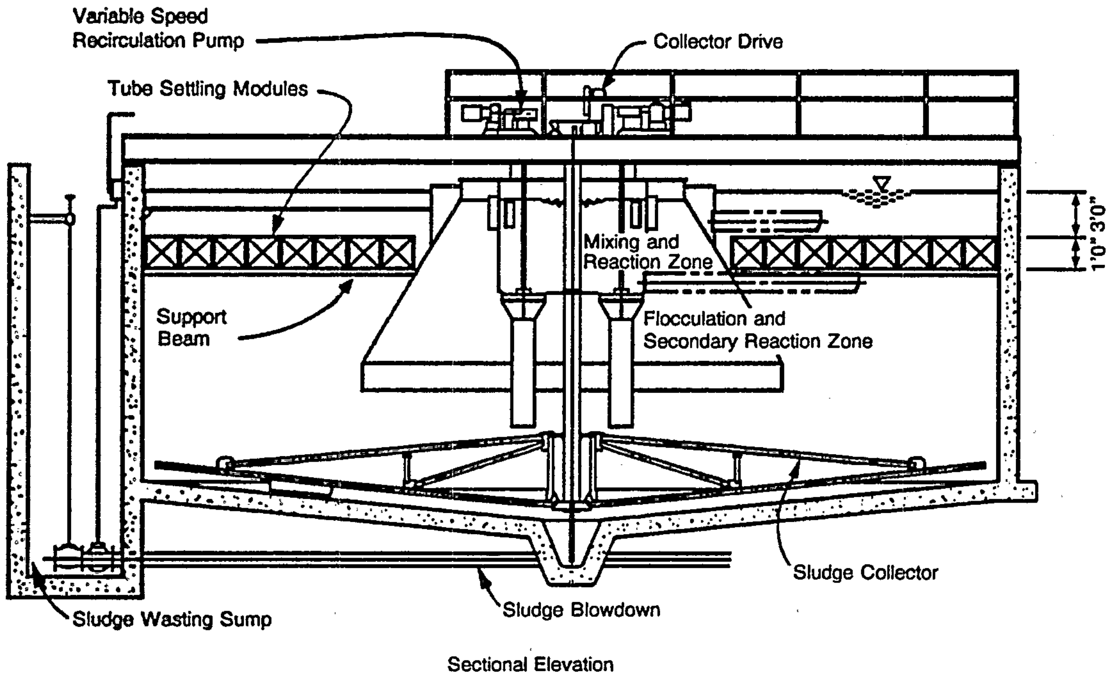

Figure 1. Solids contact clarifier with tube settlers.

SLUDGE HANDLING AND DISPOSAL

Sludge removed from sedimentation basins can be disposed of by landfilling or land application. The discharge of water treatment plant sludge to receiving waters is not considered an acceptable practice, unless approval has been obtained from a state or federal environmental regulatory agency and a National Pollutant Discharge Permit (NPDES) has been issued to regulate the discharge of pollutants.

The quantity of sludge generated will depend upon the quality of the raw water supply and the quantity and type of coagulants added to the raw water. For most small communities, sludge is dewatered prior to disposal. Dewatering may be accomplished at relatively low operation and maintenance (O&M) costs by pumping sludge into lagoons or sand drying beds. More efficient dewatering may also be accomplished by using belt filter presses, pressure filters, or centrifuges; however, these units also have higher O&M costs than the simpler drying beds or lagoons.

FINANCIAL CONSIDERATIONS

For most small community systems, sedimentation basins are included as part of a package treatment system consisting of coagulation/flocculation, sedimentation, and filtration. Costs of package systems are presented under the filtration overview.

Existing sedimentation basin performance and capacity may be increased by the addition of tube or plate settling modules. For basins treating flows under 100,00 gpd, the estimated costs for installing the modules range from $2,000 to $4,050.

Costs presented in 1992$.

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FILTRATION

TECHNOLOGY APPLICATIONS

Filtration processes are utilized to reduce turbidity and microorganism levels in a community's surface water supply. Filtration can be very effective in removing pathogenic microorganisms, including Giardia cysts and viruses. Conventional filtration processes are normally preceded by coagulation, flocculation, and sedimentation. Direct filtration processes are preceded by coagulation and flocculation only; the floc is removed directly by the filters.

PROCESS DESCRIPTION

Numerous filtration technologies are available for drinking water treatment. This section will limit discussion to four technologies most appropriate to small-community systems.

Slow sand filters (see Figure 1) consist of a bed of fine sand approximately 3 to 4 feet deep supported by a 1 foot layer of gravel and an underdrain system. The effective size of the sand ranges from 0.25 to 0.35 mm, with a uniformity coefficient of 2 to 3. Slow sand filters are designed to operate at very low application rates (0.03 to 0.10 gallons per minute/ft^2 of filter bed area) and therefore have relatively extensive land requirements. Slow sand filters are operated under continuous, submerged conditions maintained by adjusting a control valve located on the discharge line from the underdrain system.

Biological processes, and chemical/physical processes common to various types of filters occur at the surface of the filter bed. A biological slime or mat referred to as "schmutzdecke" forms on the surface of the bed which traps small particles and degrades organic material present in the raw water. Slow sand filters are limited to treating surface waters with turbidity levels less than 20 nephelometric turbidity units (NTU) due to the surface biological mat which forms as well as the small void spaces in the bed. Generally, water applied to slow sand filters is not pretreated by coagulation/flocculation and sedimentation processes.

Recent studies have demonstrated that certain modifications to slow sand filters can result in improved performance and possibly extend the application range to more turbid waters. Some of these modifications require further full-scale investigations to support the results of these studies and to develop more comprehensive design criteria. Slow sand filter modifications investigated include several pretreatment steps, such as roughing filters and preozonation, to extend the application range to lower quality waters; filter mats to increase filter runs and simplify cleaning procedures; surface amendments to remove organic precursors and control disinfection by-product formation; and harrowing techniques to reduce cleaning costs and filter "ripening" periods.

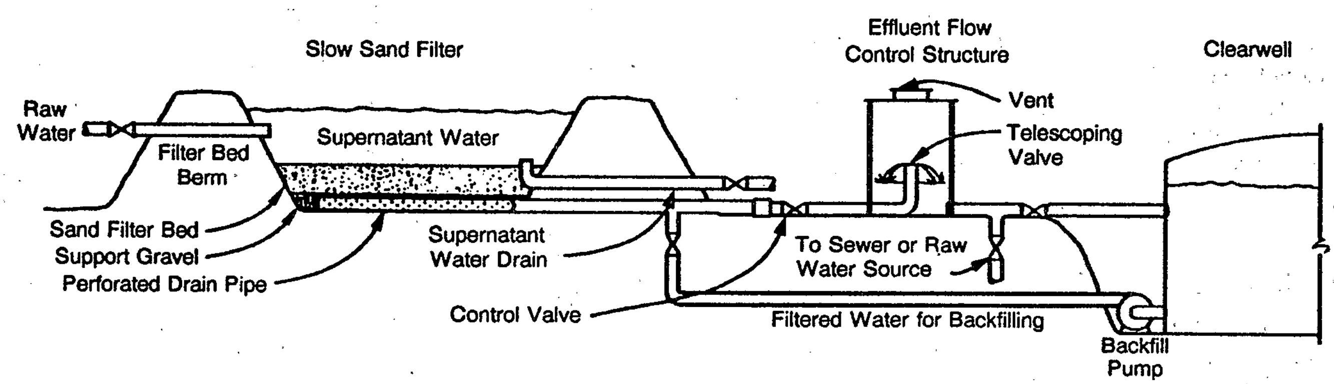

Figure 1. Typical unhoused slow sand filter installation.

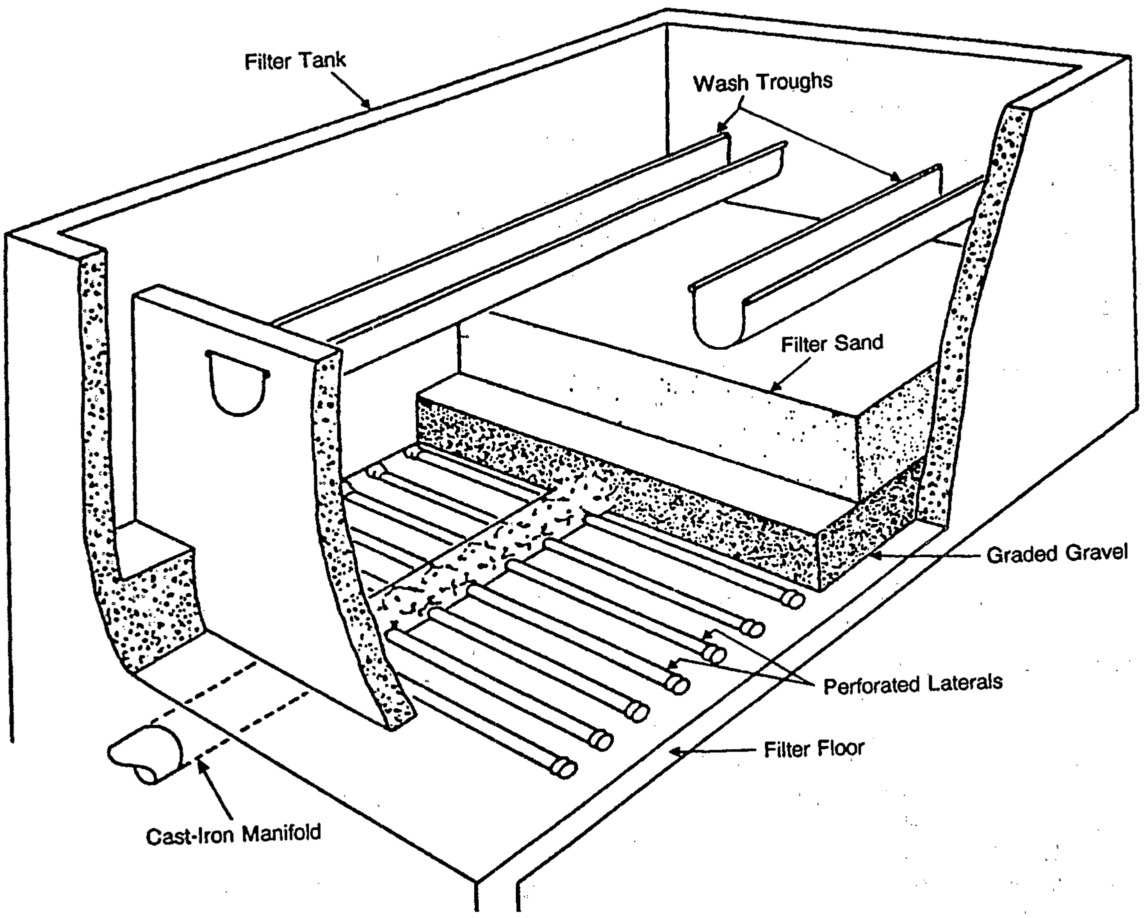

Figure 2. Cutaway view of typical rapid sand filter.

Rapid sand filters (see Figure 2) can treat raw water with high or variable turbidities at flowrates up to 2 gpm/ft^2. The filter components (except for the control panel) are contained in a watertight tank. The effective size of the sand ranges from 0.4 to 0.6 mm; this results in larger void spaces which do not fill and clog as quickly as in slow sand filters. The filter bed is usually 24 to 36 inches deep and rests on top of a layer of gravel 6 to 18 inches deep. The sand and gravel bed is supported by an underdrain system which collects the filtered water and also evenly distributes the backwash water.

Rapid sand filters are highly automated and are usually included in "package plant" systems (see Figure 3). Package plants generally consist of coagulation/flocculation, sedimentation, and filtration components. Occasionally, the sedimentation step is omitted. The filters are backwashed automatically at a predetermined head loss or whenever the turbidity of the filtered water begins to increase. On-line turbidimeters are used to continuously monitor and record the turbidity of the filtered water. Head loss indicators are provided to continuously measure the filter head loss.

High rate filters operate at application rates ranging from 3 to 10 gpm/ft^2. They are well-suited to treating raw water supplies with high or variable turbidities. High rate filters may be divided into dual-media and multi-media filters. Dual-media filters consist of an upper layer of coarse coal (anthracite) and a lower layer of sand; both layers are supported by a gravel bed. Multi-media filters consist of three types of media installed from top to bottom in the following order: coarse coal, sand, garnet. High rate filters are highly automated and contain components similar to those found in rapid sand filters. High rate filters are also included in package plant systems.

Diatomaceous earth (DE) filters have been used extensively for filtering swimming pool water; they may also be applicable for some small-community systems. DE filters are compact, pressure filters capable of removing Giardia cysts and algae from water supplies. However, they are most suited for water supplies with low turbidities (less than 10 NTU) and low bacteria counts.

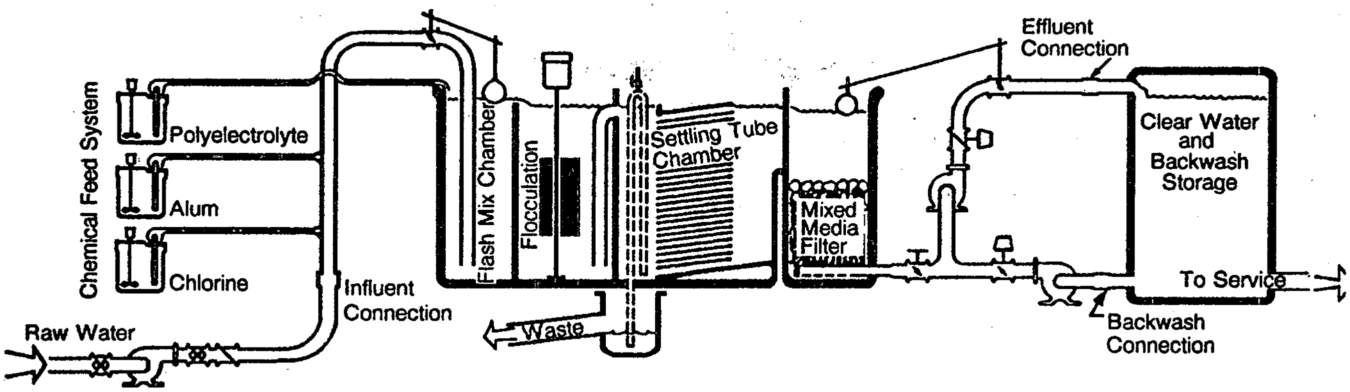

Figure 3. Flow diagram of a package plant.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

Slow sand filters are the simplest to operate. Daily operation consists of checking the raw water temperature and turbidity, filter effluent turbidity, and filter head loss. The filter effluent control valve should be periodically adjusted to maintain a constant discharge rate. Slow sand filter runs may last from 20 to 90 days depending on raw water quality before cleaning is necessary. Slow sand filters are not backwashed. Instead, the top 1 to 2 inches of sand are manually removed from the surface of the filter bed. The removed sand may be either washed and then stored for future use, or simply discarded. After cleaning, a "ripening period" of one to two days is required to allow the schmutzdecke or surface biological mat to redevelop. The filtered water is typically wasted during this period due to the poor water quality. New or washed sand should be added to the filter when the bed depth reaches 24 inches.

Other filter types, such as rapid sand filters, high rate filters, and package plants, can be highly automated. However, these systems require the presence of a properly trained operator on a daily basis to ensure continuous, reliable operation. The temperature and turbidity of the raw water should be checked daily. In addition, the filter flowrate, filter run time, filter head loss, backwash cycles, and filtered water turbidity should also be monitored on a daily basis. The duration of the typical filter run will range from 12 to 36 hours depending on the filter influent quality. The operator should periodically observe the filter backwash cycles to verify adequate cleaning of the filter and to check for excessive loss of filter media during the backwash cycle.

The effluent turbidity from diatomaceous earth filters should be checked periodically. The application of additional diatomaceous earth should be adjusted according to the measured turbidity levels. When the filter headloss reaches a predetermined level, the filter must be backwashed. After backwashing, a new precoat layer of diatomaceous earth must be formed on the filter elements before filtration can resume.

TECHNOLOGY LIMITATIONS

Slow sand filters are not recommended for waters with high or variable turbidities; water with high turbidity or algae levels will result in short filter runs. Generally, the raw water turbidity should be less than 20 NTU and the color should be less than 30 units. Slow sand filters do not remove synthetic organic compounds, disinfection by-product precursors, or inorganic chemicals. These filters also have relatively extensive land area requirements. Filters installed in cold climates must be housed.

Other filter types, such as rapid sand filters, high rate filters, and package plants, require daily attendance by properly trained operators. Inadequate coagulant dosages and/or poor sedimentation can result in filter blinding and reduced filter runs. Excessive levels of turbidity and color in the raw water may exceed package plant design specifications; in these cases, the flow capacity of the plant must be downrated to produce an acceptable water quality.

Diatomaceous earth (DE) filters do not effectively remove viruses unless the water is pretreated with coagulants and filter aids. Also, DE filters do not remove dissolved substances, such as color-causing materials.

FINANCIAL CONSIDERATIONS

Slow Sand Filters

Estimated construction and annual O&M costs are presented in Table 1 for uncovered slow sand filters. Construction costs include clay-liner, earthen berms, PVC piping, steel tank reservoir, effluent flow control structure, effluent flow meter, and pump for filter backfilling. The filter loading rate is 70 gpd/sq. ft. The costs for two filter sizes are as follows:

Table 1. Slow Sand Filter Construction and O&M Costs (1992$)

Capacity (gpd)	Construction Costs ($)	Annual O&M Costs ($)
50,000	207,900	6,800
100,000	271,100	8,100

Mixed Media Filters

Estimated construction costs for prefabricated, steel package filter units designed for flows less than 100,000 gpd and filtration rates between 2 to 5 gpm/sq. ft. range from $55,000 to $95,000. O&M costs are dependent on the filter size and the number of backwashes performed per day. Annual O&M costs are estimated to range between $4,200 to $9,500 for flows less than 100,000 gpd.

Complete Package Treatment Plants

Estimated construction costs for package plants consisting of coagulation/flocculation, sedimentation, and multimedia gravity filtration range from $98,000 to $160,000 for flows between 14,000 gpd and 144,000 gpd. The filtration rate is assumed to be 5 gpm/sq. ft. Annual O&M costs are projected to range from $10,100 to $14,400 for the same range of flows.

Diatomaceous Earth Filters

Estimated construction costs for package pressure DE filters with design capacities of 28,000 gpd and 86,000 gpd are $71,000 and $80,000, respectively. O&M costs for both systems (exclusive of DE filter aid cost) are estimated to be $10,000 per year. The annual DE filter aid cost for each filter is estimated to range from $225 (28,000 gpd) to $700 (86,000 gpd) at an application rate of 15 mg/L.

Costs presented in 1992$.

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DISINFECTION

TECHNOLOGY APPLICATIONS

The destruction or inactivation of disease-causing organisms (pathogens) in a community's drinking water supply is accomplished by disinfection processes. This is achieved in most small-community systems by using chlorine, ozone (O_3), or ultraviolet (UV) light to disinfect the water supply (sometimes referred to as primary disinfection) and by maintaining a chlorine residual in the community's water distribution system to prevent the regrowth of microorganisms (sometimes referred to as secondary disinfection). Community drinking water systems using a surface water supply or a ground-water supply under the direct influence of surface water must disinfect the water supply to comply with the requirements of the Surface Water Treatment Rule (SWTR).

PROCESS DESCRIPTION

The most commonly used disinfectant for small-community systems is chlorine. Ozone and UV may also be used as primary disinfectants; chlorine must still be added to the water to prevent regrowth of microorganisms in the distribution pipelines. A comparison of these three disinfectants is presented in Table 1; a brief description follows.

Chlorination may be accomplished by the application of chlorine gas or hypochlorite to the drinking water supply. Chlorine gas systems for small communities utilize 150 pound chlorine cylinders with cylinder mounted chlorinators. The chlorinator is used to regulate the chlorine gas feed rate. Chlorine gas is withdrawn from the cylinder by a vacuum which is created by circulating some of the water supply through an ejector. A small centrifugal pump is used to circulate water through the ejector to create the vacuum. The chlorine gas mixes with the water flowing through the ejector. A chlorine solution is then ejected into the water supply at the entrance to a contact tank or directly into a water main leading to the distribution system.

Chlorination may also be accomplished by using sodium hypochlorite, a liquid, or calcium hypochlorite, a solid. Sodium hypochlorite is available in concentrations ranging from 5 to 15 percent chlorine. A dilute solution is generally prepared and fed into the water by a chemical feed pump which has a variable output. Calcium hypochlorite is available in tablet, granular, or powdered form and contains about 70 percent available chlorine. Chlorine solutions are prepared by dissolving the calcium hypochlorite in a 30 to 50 gallon solution tank. The solution is applied to the water by a chemical feed pump with a variable output. Hypochlorite systems are generally safer and simpler to use than gaseous chlorine systems.

The pathogen reduction efficiency of chlorination systems is affected by contact time, concentration of free available chlorine, pH, temperature, and turbidity. Generally, the efficiency of chlorine disinfection is reduced under the following conditions: short contact times, insufficient dosages of chlorine, water pH above 7.2, low water temperature, and high turbidity levels. The SWTR specifies minimum chlorine concentrations (expressed as C) and contact times (expressed as T) which must be achieved at various pH concentrations and temperatures to provide inactivation of Giardia and viruses. These values are presented in the SWTR as CT values which are the product of the chlorine concentration (C) and the contact time (T). The contact time is defined as the time required for the water to travel from the point of chlorine application to the first customer during peak flow periods.

Ozone may be used as a primary disinfectant, especially in communities where chlorination may result in significant levels of trihalomethanes (THMs) or other disinfection byproducts. Ozone cannot be stored and must therefore be generated onsite. Efficient contact with the water supply is critical, since ozone is not highly soluble in water. A two-stage contactor is normally provided to satisfy any ozone demand and to ensure adequate contact. Chlorine must be added to the ozonated water prior to entry into the distribution system.

Ultraviolet light may be used as a primary disinfectant for small ground water supply systems. UV is not recommended for surface water systems as it is unable to inactivate Giardia cysts. UV systems consist of one or more UV lamps enclosed by quartz tubes. Water flows past the lamps and is exposed to the UV radiation; the UV radiation penetrates the microorganisms it comes in contact with and destroys the genetic material inside the bacteria or virus cells. UV systems cannot be used on turbid water supplies and are usually installed downstream of coagulation/flocculation, sedimentation, or filtration processes. Chlorine must be added prior to the distribution system entry point.

Table 1. Comparison of Three Disinfectants for Small-Commmunity Drinking Water Systems

Disinfectant	Advantages	Disadvantages	Application Point
Chlorine	• Effective for viruses, bacteria, and Giardia cysts • Can be used as either a primary or secondary disinfectant • Chlorine residual can be easily monitored • Available as a gas, liquid, or solid • Minimal O&M requirements, especially for liquid and solid forms	• May result in potentially harmful byproducts (THMs) • Significant safety concerns, especially for gas systems • May result in precipitation of iron and manganese	• Variety of application points • To minimize THM formation, generally added at the end of treatment steps
Ozone	• Effective against viruses, bacteria, and Giardia cysts • Enhances removal of biodegradable organics in slow sand filters	• Must be generated onsite • Does not produce a stable, long-lasting residual • May result in harmful byproducts • Low solubility in water • Complex O&M requirements • Exhaust gas must be treated to remove ozone • Difficult to measure residual	• Prior to rapid mixing step • Should provide adequate time for biodegradation of oxidation products prior to chlorination
Ultraviolet Light	• Effective against viruses and bacteria • Minimal O&M requirements • Very short contact times	• Not effective against Giardia cysts • Limited to groundwater systems not directly influenced by surface water supply • Not suitable for water containing significant levels of turbidity, color, or organic compounds	• Downstream of sedimentation or filtration processes

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

Chlorination systems must be monitored on a daily basis. Gaseous chlorine systems are more labor intensive and complex than hypochlorite systems. Regardless of the form of chlorine used, all chlorination systems should be checked daily for the following items:

- Chlorine residual concentration at the well house or treatment plant and at the farthest points of the distribution system
- Chlorine feed rate
- Amount of chlorine remaining in the cylinder or amount of solution remaining in mix tank; an adequate supply is essential to provide continuous chlorination of the water supply

All chlorine containers should be stored in a safe, secure room. Petroleum-based products, such as paints, thinners, and pesticides, should never be stored in the same room. All chlorine gas cylinders should be stored upright and securely chained to a wall. Containers of calcium hypochlorite should be kept sealed and stored in a dry location. Sodium hypochlorite solutions must be stored in cool, dark locations. These solutions lose their disinfecting power rapidly when exposed to sunlight. A maximum of about 1 month's supply of sodium hypochlorite should be stored onsite.

Ozone and UV systems must also be monitored on a daily basis. Most small systems consist of package units with built-in process monitors. Typical parameters monitored for ozonation systems include gas pressure, temperature, flowrate, electric power consumption, and ozone residual. For UV systems, when the light intensity decreases, the surface of the quartz tubes should be cleaned. UV lamps may require periodic replacement. In general, the lamps should be replaced annually; the actual frequency should be established based on site-specific experience. The turbidity of the water feed to the UV unit should also be monitored; excessive turbidity will impair UV disinfection.

TECHNOLOGY LIMITATIONS

The presence of organic compounds in a community's water supply could limit the use of chlorine as a primary disinfectant. Chlorine reacts with organic compounds and forms THMs; several THMs are suspected carcinogens. To prevent THM formation in systems using chlorine, treatment processes to remove organic material are necessary; these processes include coagulation, filtration, and activated carbon. Gaseous chlorine is very toxic; systems using chlorine gas require the following safety features:

- A separate room or building should be provided for chlorination equipment and cylinder storage.
- All chlorine handling areas should be well-ventilated and heated. A chlorine gas leak detector should also be installed.
- Emergency repair kits to stop leaks and self-contained breathing apparatus should also be provided.

Ozonation systems are complex to operate and have high O&M costs. Ozonation may also result in the formation of harmful byproducts. Ozone is highly unstable and has a low solubility in water; it does not produce a long-lasting residual. Chlorine must still be used as a secondary disinfectant.

UV light does not inactivate Giardia cysts; therefore, UV should not be used as a primary disinfectant for surface water systems and for groundwater systems directly influenced by surface water. UV systems are not suitable for water containing significant concentrations of turbidity, color, or organic compounds.

FINANCIAL CONSIDERATIONS

Chlorine

Estimated construction costs for small community gas chlorination systems do not vary significantly for capacities ranging from 10 to 80 lbs/day. A typical gas chlorination system consists of the chlorinator, scale, booster pump, and injector housed in a 10 ft. by 10 ft. building. The estimated construction cost for such a system is $25,350. The O&M cost is estimated to be approximately $3,500 per year. Hypochlorite systems (including housing) are estimated to have construction costs of $20,700 and annual O&M costs of $3,100. The actual equipment costs for hypochlorite systems are usually half of that for gas chlorination systems.

Ozone

Most small community ozonation systems have design capacities ranging from 5 to 20 lbs/day of ozone. Estimated construction costs (including housing) range from $153,500 to $189,000. O&M costs are estimated to range from $12,000 to $16,000 per year.

Ultraviolet Light

The costs reported in the literature for UV light systems vary widely. Generally, for systems treating less than 100,000 gpd, the estimated construction cost is less than $40,000. The annual O&M cost is estimated to be approximately $2,400.

Costs presented in 1992$.

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ACTIVATED CARBON

TECHNOLOGY APPLICATION

Drinking water supplies, especially from surface water sources, may contain significant concentrations of naturally occurring organic materials. These dissolved organics may cause taste, odor, or color problems in a community's drinking water and result in excessive consumer complaints. Some small-community drinking water supplies may become contaminated by synthetic organic chemicals (SOCs) originating from leaking underground storage tanks; agricultural runoff containing pesticides or herbicides; solid waste or hazardous waste landfills; or chemical wastes which have been disposed of improperly. These SOCs are toxic substances and may threaten the public health if significant concentrations develop in the drinking water. Activated carbon has been successfully used to remove synthetic organic chemicals such as benzene, carbon tetrachloride, trichloroethylene, and pesticides like DDT. Activated carbon effectively removes naturally occurring organic compounds which cause taste, odor, and color problems.

PROCESS DESCRIPTION

Two types of activated carbon, granular and powdered, have been used in drinking water treatment. Powdered activated carbon (PAC) is added directly to the raw water and removed by settling in sedimentation basins. PAC is most often used for taste and odor control. Granular activated carbon (GAC) units are more reliable for SOC removal than PAC units and are also very effective in controlling taste and odor.

Activated carbon processes remove dissolved contaminants by a process called adsorption. Particles of activated carbon contain an immense network of pores. These pores provide a very large surface area to which contaminants can adhere or stick. It has been estimated that one gram of activated carbon has a surface area equivalent to a football field. Significant bacterial populations may develop within the pores of the carbon particles. This may result in a reduction of the adsorptive efficiency of the carbon or an increase in the bacterial counts of the treated water.

All organic contaminants are not adsorbed by activated carbon at the same rate. The presence of dissolved solids or other competing contaminants may reduce the removal efficiency of activated carbon for a particular organic contaminant. It is therefore essential that adequate pilot testing be conducted on a community's water supply during the design phase. Typically, the addition of coagulation/flocculation, sedimentation, or filtration processes is necessary to pretreat surface water prior to GAC treatment.

The typical GAC unit can be similar in design to either a gravity or pressure filter. In some communities, the sand in existing filters has been either partially or completely replaced with GAC. A minimum GAC bed depth of 24 inches is recommended for taste and odor control. However, greater media depths (up to 10 feet) are needed to ensure adequate removal of potentially harmful organic contaminants and to extend the operating life of the unit. Activated carbon filters can be designed to treat hydraulic loadings of 2 to 10 gpm/ft^2. Sufficient detention time in the filter must be provided to achieve the desired level of removal of the organic contaminants. The detention time is determined by the volume of the GAC filter divided by the flow rate. This is referred to as the empty bed contact time (EBCT) since the volume of carbon in the bed is not considered. For adequate removal of most organic contaminants to occur, the EBCT should be about 10 minutes. EBCTs less than 7.5 minutes are generally ineffective.

OPERATION AND MAINTENANCE (O&M) REQUIREMENTS

GAC filters have O&M requirements which are very similar to rapid sand or multimedia filters. The major process control requirements include:

- Frequent monitoring of the head loss across the filter to determine backwashing requirements
- Monitoring of the feed water turbidity or suspended solids
- Monitoring of the filter effluent turbidity, bacterial counts, and organic contaminant concentrations

- Monitoring of the filter flowrate, backwash flowrate, and carbon loss due to backwashing

Since the pores of the carbon particles will over time eventually become saturated with the organic contaminant, the activated carbon must be periodically removed and replaced. The GAC vendor will be able to provide guidance concerning when to replace the GAC.

TECHNOLOGY LIMITATIONS

GAC filters require periodic testing of the finished water to determine the remaining bed life. The performance of GAC systems is affected by the contaminants present in the water, the variability of the raw water quality, rapid fluctuations in the filter flowrate, the types and operation of upstream treatment processes, and significant bacterial growth within the carbon particle pores which may result in filter plugging or a reduction in the surface area available for adsorption.

FINANCIAL CONSIDERATIONS

Construction costs for skid-mounted, pressure activated carbon filters having an empty bed contact time of 10 minutes and capable of treating flows ranging from 1,880 gpd to 67,900 gpd are estimated to range from $26,700 to $112,000. Costs may vary significantly depending on actual site specific pollutants and concentrations present.

Annual O&M costs without carbon replacement are estimated to range from $2,750 to $5,000 for the flow ranges noted above. The frequency of carbon replacement will be dependent on site specific design criteria. For the flow ranges noted above and an annual replacement frequency, estimated annual carbon replacement costs range from $100 to $2,400.

Costs presented in 1992$.

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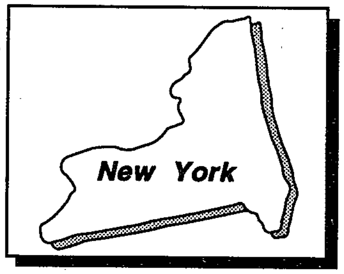
CASE STUDIES – WASTEWATER

WEST MONROE, NEW YORK CASE STUDY #1



BACKGROUND

The Town of West Monroe is located in Oswego County along the northern shores of Oneida Lake in upstate New York. This small community has a population of approximately 5,000 people. Prior to 1989, the community had no municipal wastewater collection or central treatment system. Up until that time, all wastewater was treated exclusively by individual onsite septic tank systems. The majority of these systems had functioned properly, but many of the septic systems in the Big Bay area of the Town were unreliable. This case study discusses the problems faced by the homeowners in the Big Bay area of West Monroe and the wastewater collection and treatment systems implemented to resolve these problems.

The Big Bay area consists of 220 homes on a small parcel of land adjacent to Oneida Lake. These homes, originally constructed as summer cottages, had gradually been converted into year-round residences as the population increased. The change in residency, combined with poorly draining soils and a high water table, caused the majority of the septic systems serving this area to fail. This situation persisted for almost 20 years due to the limited financial resources of the community and the perception that this was an individual homeowner problem rather than a community-wide problem. Eventually, the homeowners in the community were faced with the need to implement a solution to the failed septic systems or risk condemnation proceedings.

COMMUNITY RESPONSE

In response to this problem, several residents volunteered to form a committee (the Sewer Committee) to investigate alternative wastewater collection and treatment systems and to initiate the process of securing necessary government approvals and applying for available funds. The Sewer Committee received assistance from several state and county agencies during the planning stages of the project.

One of the first agencies to be contacted by the committee was the New York Department of Environmental Conservation (NYDEC). The NYDEC has a Self-Help Program to assist small communities with environmental problems. Personnel from the Self-Help Program helped the Town form a sewer district (a prerequisite to obtaining approval to construct municipal collection or treatment facilities in New York) and prepare a Request for Proposals (RFP) to hire a consulting engineering firm. Self-Help Program personnel also helped process the paperwork through the multilayered regulatory approval system, resulting in minimal delays and lower costs. In addition, the Oswego County Health Department assisted the Sewer Committee by helping prepare the RFP and by reviewing the collection system plans and specifications.

As the project proceeded, the Sewer Committee conducted several public information meetings which were attended by homeowners, Town Board members, and the Oswego County Health Department. Periodically, the Sewer Committee distributed flyers to the homeowners to keep them informed about the progress of the project.

EVALUATION OF ALTERNATIVES

A consulting engineering firm was selected to help the Town evaluate various alternatives and select a reasonable solution. The Sewer Committee had an active role in this process, visiting several municipalities to evaluate the performance of various systems. Several technologies were evaluated for the collection and treatment systems. A secondary treatment system was considered necessary to meet the NYDEC permit limitations of 30 mg/L Biochemical Oxygen Demand (BOD_5), 30 mg/L Total Suspended Solids (TSS), and 85-percent removal for both BOD_5 and TSS. In evaluating the various technologies, system reliability and overall cost were considered equally important. The alternative technologies evaluated and their estimated costs are shown in Table 1. The "life-cycle" costs presented include the initial construction costs, operation and maintenance (O&M) costs, and depreciation or replacement costs.

As can be seen in Table 1, the gravity sewer system and aerated lagoon alternatives had the lowest life-cycle costs. However, other factors precluded the selection of these technologies. The lagoon system was rejected because of proximity to residences, odor potential, and reduced performance under cold weather conditions. It was also observed that, for each alternative considered, the initial construction cost accounted for most of the life-cycle cost. Therefore, alternative construction methods using existing municipal resources were evaluated in order to reduce the initial construction costs, the annual user fee, and, ultimately, the total life-cycle costs. The subsurface sand filter was rejected due to its rigorous installation requirements. The gravity sewer system was also considered to be inappropriate for installation by municipal staff because of complex installation requirements.

SELECTION OF COLLECTION AND TREATMENT SYSTEMS

As a result of this process of evaluating alternative construction methods, the pressure sewer system and the activated sludge package plant technologies were chosen. By using municipal resources and having the homeowner install the grinder pump, the initial construction cost estimate for the pressure system sewer was reduced to $400,000. The revised construction cost estimate for the package plant (using municipal resources) was $250,000.

DESIGN CRITERIA

The design criteria for the collection system and treatment plant are discussed below. The key features of the pressure sewer system and package plant are summarized in Table 2.

Collection System

A low pressure sewer system with grinder pumps was installed. The sewer mains consist of PVC pipe ranging from 1.25 to 4.0 inches in diameter. The house service laterals are all 1.25 inch diameter polyethylene water pipe. The grinder pumps are progressing cavity pumps. The pump motors are 1 horsepower and require a 240 volt electrical service.

Treatment Plant

A prefabricated, steel package plant was installed. The plant is a 50,000 gallon per day extended aeration activated sludge process with secondary clarification. The plant also includes alum addition for phosphorus reduction in anticipation of future permit conditions containing phosphorus limitations. Excess capacity was provided to meet the demands of future growth in the community. The treatment plant effluent is disinfected by the application of sodium hypochlorite prior to discharge.

REDUCTION OF COSTS

The total project costs were significantly reduced by the following steps taken by the Town.

Assumption of General Contractor Functions

The Town acted as the general contractor for the entire project. The Town Highway Department Superintendent was instrumental in this role, overseeing the subcontractors used in the project and communicating on a regular basis with the consultant when problems arose during construction.

Use of Municipal Resources for Construction

Highway Department employees and equipment were used to build an access road to the treatment plant site and to clear and excavate the site. Use of the highway crew was limited from May 1 to October 15 so that highway

Table 1. Collection and Treatment System Alternatives and Costs for West Monroe, NY

	Initial Construction Costs ($)	Annual O&M ($)	Life-Cycle Costs ($)
Collection System			
Conventional Gravity Sewers	860,000	5,000	924,100
Pressure Sewers	800,000	10,000	936,400
Treatment System			
Subsurface Sand Filter	450,000	5,000	530,200
Aerated Lagoon	350,000	10,000	471,300
Activated Sludge Package Plant	400,000	15,000	580,800

Table 2. Key Design Features of the Selected Collection System and Treatment Plant

Collection System
• Low Pressure Sewers
• PVC Pipe - 1.25" to 4.0" Diameter
• House Service Laterals - 1.25" Diameter Polyethylene Water Pipe
• Grinder Pumps
• Pump Motors - 1 Horsepower, 240 Volt
Treatment Plant
• Extended Aeration Activated Sludge Process
• 50,000 gpd Capacity
• Prefabricated Steel Package Plant

personnel would be available for maintaining municipal roads during the winter season. Therefore, adherence to a strict schedule was essential to ensure completion of the site work prior to the start of the winter season. An unexpected pocket of "soft" or unconsolidated material at the site prevented the construction of the thick concrete slab needed to support the package plant and could have resulted in a significant delay. The consultant proposed the use of a timber pile foundation as a solution to the problem. The Highway Department purchased and installed 65 used telephone poles, using a pile driver borrowed from the Oswego County Road Department.

Direct Purchase of Materials and Equipment

Additional cost savings were achieved by direct purchase of the majority of system components and equipment specified by the consultant. West Monroe purchased all pipe, fittings, and manholes required for the low-pressure sewer system as well as the sand and gravel needed for the pipe bedding and backfill. Bids on the prefabricated steel package plant were requested from several manufacturers; the contract was awarded to the lowest responsible bidder. Complete grinder pump (GP) packages consisting of pump, fiberglass reinforced polyester tank, and controls were also purchased directly by the Town.

Creative Allocation of Responsibilities

Additional project cost reductions were achieved by making the GP installation and hook-up to the low-pressure sewers the responsibility of the homeowners. The GP packages were sized to accept wastewater from two residences, which resulted in the purchase of fewer GP packages and significantly reduced installation costs. Minimum monthly service charges assessed by the electric power company were avoided by connecting the GP package to the existing electrical service of one of the two homes. (The home which incurs the additional electrical fees receives a credit on its sewer use charge.) As general contractor, West Monroe provided installation guidelines and inspected each installation. This creative arrangement was successfully achieved by having all parties consent to a three-party agreement or easement between the Town and the two homeowners sharing the GP service. GP packages were not distributed until both homeowners had signed the agreement. The arrangement also provided that the Town be responsible for maintenance of the GPs.

RESULTS AND SUMMARY

The collection and treatment systems have been in operation since spring 1989, and, according to the Highway Department Superintendent, both systems have performed well since startup. A summary of the treatment plant performance is provided in Table 3.

Since the Town could not afford to hire a full-time treatment plant operator, a member of the Highway Department staff was assigned to operate the treatment plant. This individual has attended State of New York Operator Training courses to prepare for the state-certified operator licensing exams; his time is divided equally between the treatment plant and the Highway Department.

Table 3. Performance Summary West Monroe, NY, Wastewater Treatment Plant January - December 1991

Parameter	Flow (MGD)	BOD_5 (mg/L)	TSS (mg/L)
Permit Limit*	0.056	30	30
Influent**	-	312	212
Effluent**	0.020	13	28
% Removal	-	96	87

* Permit requires 85-percent removal for both BOD_5 and TSS.
** Influent and effluent values shown are annual averages.

Long-term financing for the entire project was obtained through the New York State Revolving Loan Program. A 20-year loan was secured at 5-percent interest. Short-term funding for initial project costs was obtained by the issuance of bond anticipation notes (BANs).

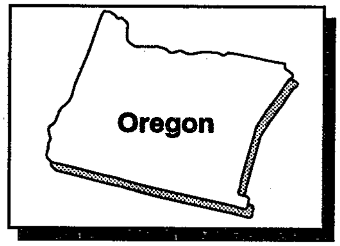
Overall, project costs were reduced by more than $500,000; the final project cost for 220 homes was $695,000 rather than $1,200,000 as was initially estimated. Annual sewer user fees are approximately $425 per home as compared to initial estimates of $625 using conventional construction methods.

In summary, the Town of West Monroe solved a longstanding public health problem at an affordable cost by incorporating the elements and strategies summarized in Table 4. This was achieved through the combined efforts of community, county, and state personnel, and careful cost reductions.

Table 4. Key Elements of West Monroe's Successful Project Resolution

- ✓ Community volunteers
- ✓ Assistance from the New York State Department of Environmental Conservation Self-Help Program and Oswego County Department of Health
- ✓ Employment of an engineering firm with small-community experience and knowledge of alternative collection and treatment technology
- ✓ Assumption of general contractor functions by the Town
- ✓ Project labor requirements supplemented by highly motivated municipal employees
- ✓ Direct purchase of materials and equipment
- ✓ Multiple hook-ups to properly sized single GP installations
- ✓ Use of existing municipal labor force to operate wastewater treatment plant

MAPLETON, OREGON
CASE STUDY #2



BACKGROUND

Mapleton is located on the west bank of the Siuslaw River in Lane County, approximately 10 miles inland from Oregon's Pacific coast. This small community, which has a population of approximately 800, is located in the Siuslaw National Forest.

Mapleton is an unincorporated village in a region that has been historically dependent on the timber and timber products industry. The commercial area of Mapleton encompasses approximately 30 acres and consists of three residences and approximately 16 commercial/industrial establishments. Due to its location on the bank of the Siuslaw River, local topography, and hydrogeology, this area experiences seasonal, very high groundwater tables. This condition has negatively affected the use of onsite disposal systems by the properties in this area. Mapleton had been aware of the wastewater disposal problem in this area for decades; a combination of factors in the early 1980s resulted in the active pursuit of a solution to the problem.

Prior to the construction of the Mapleton Sewerage Facility, all commercial establishments and residences either 1) utilized onsite septic systems, 2) discharged sewage untreated to the earth via cesspools, or 3) discharged directly to the Siuslaw River. Septic systems in general provided an acceptable means of handling wastewater in all but the riverside commercial area. Several buildings in this area had direct discharges to the river, and septic systems in this area were subject to seasonal surface and lateral leakage.

In the early 1980s, the severity of Mapleton's problem became apparent to the County Board of Health and to the Oregon Department of Environmental Quality (DEQ). Sanitary surveys of Mapleton's commercial area convinced DEQ and Lane County officials that the situation in Mapleton's commercial area warranted serious attention. To encourage action on Mapleton's part, a building permit moratorium was imposed on the commercial area until appropriate alternative sewage handling could be provided.

This case study examines the technical and financial problems Mapleton encountered and discusses how a joint effort by the state, local businesspersons, Lane County, and Mapleton resulted in the construction of an innovative and effective wastewater collection and treatment system.

COMMUNITY RESPONSE

In an effort to find a solution to this problem, the owners of the properties in the commercial area sought the assistance of a number of agencies. These included the Rural Communities Assistance Corporation (RCAC) , the Oregon DEQ, the Oregon Rural Communities Assistance Program (ORCAP), and Lane County. ORCAP and DEQ conducted sanitary surveys that indicated that onsite treatment was not feasible in the commercial area and that some form of centralized treatment and offsite disposal would be necessary for Mapleton.

Because Mapleton was unincorporated, it was recognized that a legal entity would need to be created to make decisions, pursue funding, and ultimately build and operate a centralized wastewater treatment system. To this end, the owners of the commercial properties formed the Mapleton Commercial Area Owners' Association (Association). An agreement was established between Lane County and the Association whereby the County would sponsor a request for an Oregon Community Block Grant. Once the treatment facility was built, all responsibility for its operation and maintenance would be passed to the Association.

Lane County and ORCAP remained actively involved in the project throughout both the pursuit of funding and the construction of the collection and treatment systems that were ultimately selected. The Siuslaw Port District (which is a state-established port management agency with authority over a substantial part of the Siuslaw River) provided legal assistance in the formation of the Association and ultimately acted as title holder to the property on which the treatment plant was constructed to provide the benefits of public rather than private ownership of the property.

The Association was actively involved in all phases of this project due to the willingness of both individuals and businesses to support the project. For example, the local bank made both staff time and the bank's physical resources available to the Association during the project. Members of the Association perceived the successful elimination of this problem as of major importance to the economic health of Mapleton.

A first application for block grant funding submitted by Lane County was not successful. With the continued assistance of Lane County, a second application was prepared; this ultimately resulted in the award of a grant in the amount of $319,000.

EVALUATION OF ALTERNATIVES

A consulting firm was hired by Lane County to assess the situation more completely and to recommend possible solutions. The consulting engineer determined that centralized treatment and offsite disposal would only be necessary for the commercial area, with the effluent to be disposed of by land application.

Early in the evaluation process, several factors were considered to be of great importance. First, the limited number of connections made the selection of a treatment system extremely sensitive to capital cost and the availability of state funds. Second, the isolated location of Mapleton in conjunction with the financial limitations made any system requiring the routine attention of experienced operations personnel unattractive. Finally, very small wastewater flows are difficult to treat effectively in conventional biological treatment systems.

As a result of these considerations, the engineer initially recommended the use of a recirculating gravel filter with filter effluent to be disposed of by application to public land. (Oregon requires that all treatment facilities use non-surface discharge options whenever possible.)

As the Association and the engineer further pursued the initial proposal, several major impediments to the use of land application became apparent. These included difficulties in obtaining right-of-way for installation of the necessary pipeline to the potential application sites and unacceptably high projected pumping costs due to the significant difference in elevation between the commercial area and the proposed application sites.

These considerations prompted the pursuit of a surface discharge permit from DEQ. In addition to Oregon's general bias against surface water discharges, DEQ had apparently not approved surface water discharges for many recirculating filter systems. As a result, Mapleton was required to evaluate the potential impact that the proposed system would likely have on the Siuslaw River. The results of this study in conjunction with review of NPDES monitoring data for other Oregon recirculating filters ultimately prompted DEQ to grant a surface water discharge permit to the Association.

SELECTION OF COLLECTION AND TREATMENT SYSTEMS

As a result of the process previously described, Lane County and the Association ultimately elected to construct a recirculating gravel filter on a site in the middle of the commercial area. In addition, due to the condition of the existing septic systems in the commercial area, new septic tanks and piping were installed for every connection served by the new system. The cost of the collection and treatment systems together was approximately $400,000.

DESIGN CRITERIA

The design criteria for the collection and treatment systems are examined below. Key features of the septic/gravity sewer collection system and filter are presented in Table 1.

Table 1. Key Design Features of the Selected Collection System and Treatment System

Collection System
• All New Services, Laterals, Mains; Laterals and Mains of 6-Inch PVC
• Installation of New Septic Tanks With Concrete Construction; Capacities From 1,000 to 3,000 Gallons
• Laterals and Mains Gravity Drain to Plant
Treatment Plant
• Recirculating Filter
• Pea Gravel Media (3 to 5 mm effective diameter, uniformity coeff. < 2.0)
• Two Cells, Each 35 ft. x 70 ft. Media Depth
• Two Recirculation Tanks; Total Capacity of 25,000 Gallons
• 25,000 gpd Design Capacity

Collection System

As noted, new building services, laterals, septic tanks, and mains were installed for the service area. PVC piping was used for branches and mains. The topography of the commercial area allowed the use of a gravity collection system, with no lift stations required. Septic tanks installed were of prefabricated concrete construction and ranged in size from 1,000 to 3,000 gallons.

Treatment System

A two-cell recirculating filter was constructed on a 1.5-acre site located in the commercial area. Each cell measures 35 by 70 feet and has a pea gravel media depth of 3 feet. Cell construction is bermed earth with concrete retaining walls and a PVC membrane liner. The distribution manifold is constructed of 2-inch perforated PVC pipe; the underdrains are perforated 4-inch PVC pipe. Two recirculation tanks are provided, each consisting of 60 feet of 72-inch diameter reinforced concrete pipe. Two recirculation pump stations are provided; each station consists of one 250 gpm (@ 22 ft. total dynamic head) submersible pump. Effluent is chlorinated using sodium hypochlorite and dechlorinated using sulfur dioxide. Effluent is pumped to the Siuslaw River by a discharge pump station which consists of two 100 gpm (@ 15 ft. total dynamic head) submersible pumps. A small chemical feed/control/lab building was also constructed. A schematic of the system is provided in Figure 1.

REDUCTION OF COSTS

The total project costs were reduced by the following steps.

Adoption of a Low Capital/Operating Cost Treatment System

The selection of a septic/recirculating filter treatment system has provided Mapleton with a cost-effective solution to its problem. Installation costs of the gravity sewers were somewhat higher than if Small Diameter Pressure Sewers (SDPS) had been used; however, the elimination of the grinder pumps associated with SDPS provided an offset savings. In addition, maintenance costs of the installed collection system are expected to be significantly less than if a SDPS system had been used.

As in the case of the collection system, the major cost savings associated with the treatment system are operations and maintenance (O&M) rather than capital. The recirculating filters do not require regular solids disposal, which eliminates a major conventional treatment operating cost. It should be noted that to a certain extent, this cost has been picked up directly by the customers in the form of septic tank cleanout costs. Another significant savings is realized due to the limited need for operator control. Mapleton has retained a local water distribution system operator on a part-time basis, thereby eliminating the need to hire an operator from outside the area.

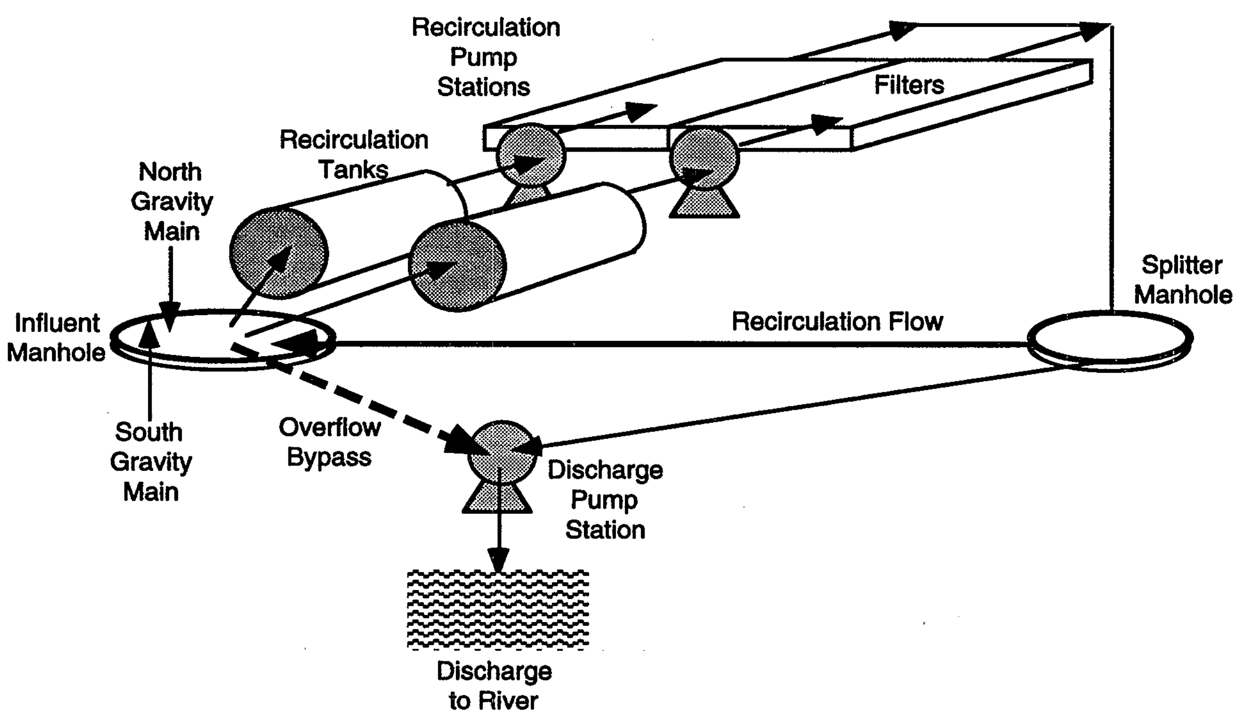


Figure 1. Plant schematic for Mapleton, Oregon.

Reliance on Lane County and Other Agencies

Mapleton made full use of the various types of assistance available to it from the State, County, and various community outreach programs. This reduced project costs in two ways. The most obvious was that these agencies provided technical and organizational guidance and services for which Mapleton might otherwise have obtained by contract. Examples of this include both Lane County's hiring of the consulting engineer and the provision of county resources to carry out improvements on the treatment plant access road.

Less obvious is the impact that the guidance had in keeping Mapleton "on the right track." Had Mapleton's citizens made decisions regarding this project with less guidance, they might well have made inappropriate and ultimately more costly choices.

Limitation of Funding Sources to Impacted Parties

In addition to the block grant, funding for this project was obtained directly from the affected parties; total local funding was approximately $100,000. As a result, Mapleton incurred no debt with this project.

The direct funding of this project was to a large extent made possible by the local bank's decision to provide loans at very attractive terms to businesses and residents affected by the project. In addition, the bank elected simply to pay the connection fees of one or more senior citizens. Monthly sewer fees for businesses are $64 per month; those for residences are $20 per month.

RESULTS AND SUMMARY

The collection and treatment systems have been in service since 1989 and both have generally performed up to or above expectations. A summary of recent plant performance data is provided in Table 2.

As noted above, the Village has retained an operator from the local Water District on a part-time basis; this individual has received the additional state training necessary to be type-certified for the Mapleton plant.

Relatively minor problems have surfaced since the plant went on line. Settlement of the plant site has required the addition of media to eliminate a low area on the level upper surface of the filter.

As a result of the successful start-up of the treatment system, DEQ lifted the building moratorium. Several new businesses have opened in the downtown area, and the Florence Regional Library recently opened a branch in the commercial area. Progress is also now being made to restore several commercial area buildings of historical significance.

In summary, Mapleton has overcome a number of technical and institutional barriers to solve a serious human health and environmental problem. This success was achieved in no small part through the cooperative effort of the Village and a number of agencies and through thoughtful consideration of Mapleton's resource limitations when evaluating alternative solutions. Table 3 summarizes these elements and strategies.

Table 2. Performance Summary
Mapleton, Oregon, Wastewater Treatment Plant
July 1991 to February 1992

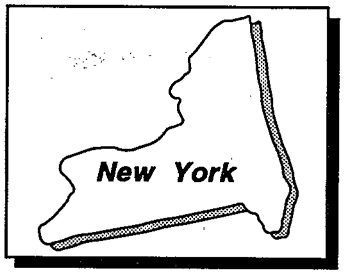
Parameter	BOD_5 (mg/L)	TSS (mg/L)
Permit Limits	10	10
June 1991	6	7
July 1991	10	6
August 1991	7	7
September 1991	-	-
October 1991	34*	7
November 1991	3	5
December 1991	4	10
January 1992	8	6
February 1992	14	8

* This datapoint was considered to be the result of laboratory error.

Table 3. Key Elements in Mapleton's Successful Project Resolution

- ✓ Community involvement
- ✓ Local businesses championed the project
- ✓ Employment of an engineering firm with experience in small-system design
- ✓ Selection of a low O&M cost treatment alternative
- ✓ Successful pursuit of state funding

PORTVILLE, NEW YORK CASE STUDY #3



BACKGROUND

The Village of Portville is located in Cattaraugus County in the southwestern portion of the State of New York, approximately 7 miles east of Olean on the north bank of the Allegheny River. The population of this bedroom community is approximately 1,100. Prior to 1988, the Village operated a primary treatment system consisting of an Imhoff tank and sand sludge drying beds.

As Portville's Imhoff tank provided only primary treatment, the New York State Department of Environmental Conservation (NYDEC) had been applying increasing pressure on Portville for more than a decade to upgrade the Village's wastewater treatment capabilities. This pressure ultimately took the form of a formal enforcement action against the Village. Finally, on New Year's Day in 1988, the Imhoff tank suffered a major structural failure.

As a result, the Village was faced with the need to replace the existing treatment system with one that would consistently meet secondary treatment standards or it would face further enforcement actions.

This case study examines the challenges encountered by the Village of Portville in upgrading its treatment plant, describes how the Village ultimately elected to construct a type of treatment system rarely used in the Northeast, and evaluates how well the Village has been served by that decision.

COMMUNITY RESPONSE

The Village government consists of four trustees and the Mayor. Together, these individuals made all the decisions concerning selection of both consulting engineers and a treatment system. Efforts to involve the general citizenry were somewhat limited; notification of Village residents was limited to those within 500 feet of the proposed plant site (which was the site of the former treatment plant). General community involvement was largely limited to the resolution of a post-startup odor problem.

In response to the NYDEC enforcement action, the Village retained a consulting engineer to design a new treatment plant, to handle the completion and submission of applications for the necessary permits, and to complete various grant and loan applications. As a result of the engineer's success in pursuing available funding, the Village received a grant of approximately $1.5 million, or 82.7% of the estimated cost of the treatment system first proposed. (Treatment technology selection is discussed subsequently in this case study.)

Following receipt of funding approval, the consulting engineer revised the original cost estimates for the proposed system; these revisions indicated higher than expected capital and operating costs. In conjunction with evaluation of the experiences of other area towns that had recently upgraded their treatment systems, these higher cost estimates convinced the Mayor and the trustees that the Village would not be financially capable of operating the proposed treatment system. This was perceived by the Village to be a "Catch-22" situation, in that failure to upgrade the treatment plant would place the Village at risk of further enforcement action by NYDEC, as would failure to operate the proposed treatment plant properly.

In an effort to resolve this conflict, a second consulting engineer was retained, who in turn retained a treatment process design engineer with experience in alternative small-scale treatment technologies. This individual's recommendation was ultimately accepted and constructed. Coincidentally, the capital cost of this second recommended technology was very close to that of the original more conventional recommendation.

NYDEC originally had concerns regarding the Village's revised proposal and did not immediately issue approval for the change. Following review of additional information regarding this technology, NYDEC did allow the Village and its engineers to proceed. The cost of the new facility and attendant equipment was approximately the same as originally estimated for the more conventional plant ($1.8 million). Funding for the remaining 17.5% of capital costs was provided by the Village through long-term loans.

SELECTION OF TREATMENT SYSTEM

In the early 1980s, the Village inspected portions of the collection system to evaluate the feasibility of reducing infiltration. At that time, rehabilitation was judged to be economically infeasible. Based on this earlier evaluation, the possibility of rehabilitating the collection system was not considered when the need to upgrade treatment capability was addressed in the late 1980s.

The Village's first consulting engineer recommended a Rotating Biological Contactor (RBC) treatment system. As noted previously, operating cost estimates for this original system convinced the Mayor and trustees that the Village could not carry the debt service necessary to fund the Village's share of the construction cost and at the same time cover the expected annual operations and maintenance (O&M) costs.

As a result of reaching this conclusion, the Mayor and trustees decided that a reevaluation of treatment alternatives was in order. The Village decided to consider a substantial change in project direction, knowing that this might entail the expenditure of more funds on system redesign.

A second consulting engineering team retained by the Village recommended a recirculating stone filter (RSF) treatment system. This recommendation was made on the basis of 1) this type of system's ability to handle wide ranges of hydraulic loading (important due to the high rate of infiltration and inflow experienced by Portville's collection system) and 2) its low O&M requirements and costs.

DESIGN CRITERIA

The design criteria for the Village of Portville's RSF system are discussed in the following paragraph. Key design features are presented in Table 1. A schematic of the system is shown in Figure 1.

The RSF design was chosen for its low O&M costs and its ability to treat widely fluctuating flows and waste strengths. The system ultimately constructed by the Village consists of three major components: a 700,000 gallon capacity "communal" septic tank, a 28,000 gallon recirculation dosing tank (with associated recirculation pumps and controls), and a 54,000 square foot pea gravel media filter. In order to handle septic tank solids, the Village also purchased an all-terrain sludge hauling/application vehicle, constructed a separate septage holding facility, and secured the use of a 30-acre application site.

REDUCTION OF COSTS

Capital cost reduction was not significant in Portville's wastewater treatment plant upgrade project. The constructed cost of $1.8 million was in fact somewhat high compared to the expected cost for a similarly sized, "package-type" extended aeration treatment system (source of comparison - MEANS Cost Data). This is due to several factors. First, RSF systems cannot effectively treat raw wastewaters. The solids content of typical raw sanitary wastewater will quickly plug the filter media and significantly reduce the life of the filter. It is therefore essential that the raw wastewater receive adequate pretreatment. This can be accomplished using gravity settling in either septic tanks, Imhoff tanks, or primary clarifiers. For the Portville treatment system, a communal septic tank was constructed to provide the necessary pretreatment and equalization. This 700,000-gallon unit increased capital costs substantially.

Table 1. Key Design Features of the Selected Portville, NY, Wastewater Treatment System

- Design capacity of 400,000 gpd
- Septic Tank Volume – 700,000 Gallons
- Filter Area of 54,000 sq.ft.
- Media Depth – 1.5 ft.
- Media – 1.5 - 2.5 mm Pea Gravel
- Recirculation – Four 970-gpm Pumps

The Village also made a large investment in the septage storage facility and transport/disposal truck. This capital expenditure was motivated by both concern regarding disposal of material from the "communal" septic system and an interest in developing septage handling capability as a revenue-generating operation.

The impetus for selecting the system ultimately constructed was its expected low operating costs. Actual annual operating costs through late 1990 were averaging about $1.93 per 1,000 gallons of sewage treated, includ-

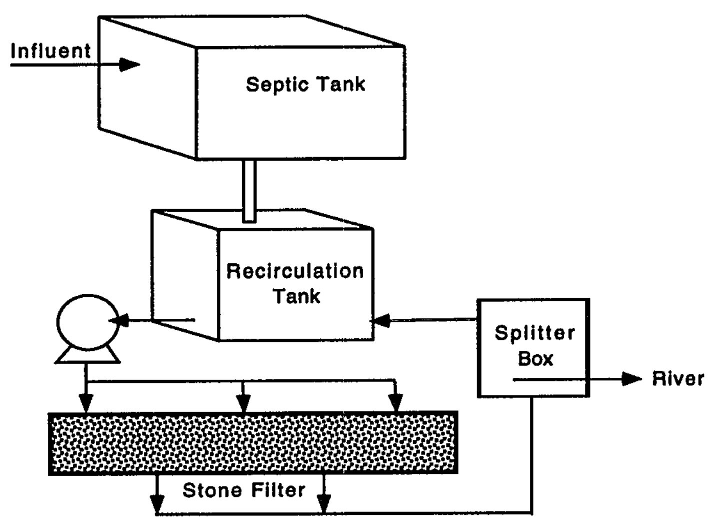


Figure 1. RSF system schematic.

ing debt service (which represents more than 50%). Labor costs are somewhat higher than normal for this type of system because the Village has provided a full-time operator. Non-labor, non-debt service operating costs have in fact been low, with pumping costs averaging about $.10/1,000 gallons. As a result of the operating costs mentioned earlier, the average resident pays approximately $12 per month, or $144 per year.

RESULTS AND SUMMARY

In general, the selected treatment system has provided a high quality effluent which has easily met the requirements of the SPDES permit. Table 2 summarizes plant performance from January 1989 through August 1990. During that time period, plant effluent BOD_5 averaged about 7 mg/L and effluent TSS averaged about 2 mg/L. The plant has also achieved complete nitrification, with certain effluent samples showing ammonia concentrations below 1 mg/L.

The plant has also experienced and had to overcome operational difficulties related to high flow, odors, and industrial discharges.

Higher than expected flow rates initially caused difficulties in meeting the filter bed's dosing and resting requirements. The pump supplier provided considerable assistance (at no cost) in modifying the recirculation system to allow it to function properly under the entire range of flows being experienced.

Odor control was one of the major community and regulatory concerns with the system ultimately constructed. Following startup, odor from the dosing tank became a problem. The addition of an aeration system to this tank did not provide the expected relief, and the Village was forced to install an odor control system.

In June 1991, a local pulp and paper mill significantly changed its production processes. This substantially changed the characteristics of the suspended solids in its discharge. This change in characteristics resulted in significant amounts of solids passing through the septic tank to the filter. This in turn resulted in the plugging of the dosing pumps and clogging of the pea gravel filter media. The effect of this was to render the filters effectively inoperable. The pulp and paper mill was ordered to cease its discharge to the plant. The mill has since achieved zero discharge. Rehabilitation of the system was carried out by the Village, which is currently attempting to recover the costs from the mill. Table 3 provides an overview of both positive and negative aspects of Portville's experience in upgrading its treatment system.

Table 2. Portville, NY, Performance Summary January 1989 Through August 1990

Parameter	BOD_5 (mg/L)	TSS (mg/L)
Influent	85	85
Effluent	6.8	1.7
% Removal	92	98

Table 3. Key Elements of Portville's Experience

Positive Elements

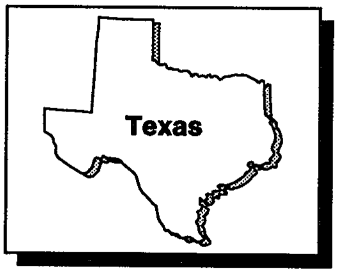
- ✓ Willingness to "step back" and change direction
- ✓ Selection of an engineering firm with small-system experience
- ✓ Successful pursuit of funding assistance
- ✓ Selection and construction of a treatment technology which under most conditions has easily met its permit

Negative Elements

- ✓ Overpurchase of O&M staffing and equipment
- ✓ Failure to consider effects of industrial waste source

CASE STUDIES – DRINKING WATER

LOS YBANEZ, TEXAS
CASE STUDY #4



BACKGROUND

The Town of Los Ybanez is located in Dawson County, Texas. The dwellings that make up the Town were originally constructed by the United States Department of Agriculture (USDA) as a migrant labor camp known as Dawson County Labor Camp. A total of 75 units were constructed on an 85-acre site.

In 1980, the entire site was purchased by the Ybanez family. The family began to renovate some of the 40-year-old dwelling units, since the site had fallen into general disrepair. Tenants began to move into units as they were rehabilitated. Most residents of the community are on low or moderate fixed incomes. Eighty percent of the units are subsidized by the Department of Housing and Urban Development.

In 1982, the 39 residents voted unanimously to incorporate the site into a town to be known as Los Ybanez. The Town was formally incorporated on April 2, 1983, and is governed by four councilmen and a Mayor.

By 1988, 22 of the wood frame structures had been rehabilitated, and were occupied by 40 households. Part of one structure was used as the Town Hall. The Town also had a church and general store. The total population of the Town was 150 persons. Drinking water for the residents was supplied by one well provided by the USDA when the original labor camp was built. A 50,000 gallon underground storage tank was provided, but the system did not include pressurized storage. Water distribution was accomplished through 4 inch and 6 inch asbestos cement mains. Due to the high dissolved solids content of the water, extensive mineral deposits had built up in the mains through the years, severely reducing available water pressure.

In 1988, the Texas Attorney General notified the Town of his intent on behalf of the Texas Department of Health to file suit against Los Ybanez. The suit would charge violations of rules and standards established by the Texas Department of Health governing the construction, operation, and maintenance of public drinking water systems. The following specific violations were to be charged:

1. That Los Ybanez failed to provide two water samples to the Department of Health for bacteriological analysis;
2. That minimum pressurized storage tank capacity of 50 gallons per connection was not provided;
3. That two or more pumps having a combined capacity of more than 2 gallons per minute per customer connection were not provided; and
4. That maximum allowable concentration limits for fluoride, chloride, and sulfate were exceeded.

The suit would allege a direct potential hazard to public health and sought civil penalities and injunctive relief to close the system until compliance was achieved.

COMMUNITY RESPONSE

Los Ybanez responded to the Attorney General in March 1988, prior to the suit being filed. The Town indicated that 16 samples for bacteriological analysis were submitted in 1987 and attributed the failure to submit two samples in 1988 to an operator who had subsequently been dismissed. The Town acknowledged that 9 of the 16 samples submitted in 1987 did not meet Health Department bacteriological standards.

Los Ybanez further advised the Attorney General of its intent to upgrade the water system. To this end it had contacted the Community Resources Group, Inc. (CRG) located in Lubbock, Texas. CRG works with the Texas Department of Agriculture to bring needed and affordable water and wastewater services to small, low-income communities in Texas.

CRG is a private, non-profit rural development organization established in 1975 to seek long-term solutions to problems faced by rural people and communities in the southern states of Alabama, Arkansas, Louisiana, Mississippi, Oklahoma, Tennessee, and Texas. CRG's work is supported financially by both public and private sources. In general, services are provided at no cost to rural communities.

The Town hoped to obtain a low-interest loan from CRG in order to meet the current Texas requirements and to provide sufficient capacity to allow the full occupancy of all available housing units. CRG advised the Attorney General that it was offering assistance, and requested that the suit be delayed for as long as the Town made good faith efforts to correct the problems.

CRG then contacted a consulting engineering firm. CRG requested that the firm investigate rapid means of providing safe water for Los Ybanez. The engineer responded in April 1988 that the quickest means would be to buy water from the neighboring City of Lamesa through an existing but abandoned 4 inch asbestos cement line which ended near the northwest part of Los Ybanez. Although parts of the line had been damaged or destroyed, the firm believed the line could be restored to service. The estimated cost of rehabilitation and repair was $35,000.

In May 1988, it was learned that the owner of the property on which the line was located objected to its repair and contended that the easement for its installation had been lost by non-use or abandonment.

With the assistance of CRG, Los Ybanez applied to the State of Texas for a grant to use in solving its drinking water problems. In December 1988, the Town was notified that it had been awarded $352,650 by the Texas Department of Commerce for water system improvements. The Town continued to work with the engineering firm to identify possible alternatives. In March 1991, the firm was appointed as the design engineer for the drinking water improvement project.

EVALUATION OF ALTERNATIVES

In order to improve the quality of water to comply with the standards established by the Texas Department of Health, four alternatives were evaluated:

1. Purchase water from the City of Lamesa;
2. Treat water from the existing well with activated alumina;
3. Treat water from the existing well with an electrodialysis system; and
4. Treat water from the existing well with a reverse osmosis system.

Purchase of water from the City of Lamesa was judged to be the most attractive option because of its low implementation cost. In addition, it was the only option which did not require Los Ybanez to operate a treatment system. This option was not selected because resolution of the easement issue was unlikely to be accomplished until after a protracted legal dispute.

The use of activated alumina would have been much less effective than either electrodialysis or reverse osmosis. Activated alumina has been shown to reduce fluoride levels, but does not effectively remove chloride, sulfate, or hardness. This option was therefore rejected.

Both electrodialysis and reverse osmosis are membrane separation technologies. Low total dissolved solids (TDS) water passes through the membrane, leaving a more concentrated stream on the feed side of the membrane. The low TDS side of the membrane is the product water stream. The higher TDS concentration side of the membrane produces a reject stream that must be disposed.

The two technologies differ in the force used to drive water across the membrane. In electrodialysis, electrical energy is used directly. In reverse osmosis, hydrostatic pressure produced by a pump is the driving force.

Reverse osmosis was the treatment alternative ultimately selected by Los Ybanez because of its lower capital cost. The reverse osmosis system was estimated to cost between $23,000-25,000, based on 15,000 gallons per day (gpd) of product water. The electrodialysis system was estimated to cost $54,000, based on production of 11,000 gpd.

Another factor considered in selecting the treatment system was operating costs. Vendor estimates showed an operating cost of $0.94/1,000 gallons product water for electrodialysis compared to $2.00/1,000 gallons for reverse osmosis. However, the Town obtained operating costs from Dell City, Texas which was already operating an electrodialysis system. Operating costs for 1983 in Dell City were $4.29/1,000 gallons.

PROJECT DESIGN

An approach for utilization of the reverse osmosis system was developed which took advantage of the fact that reverse osmosis would provide a product water quality which would significantly exceed the Health Department standards. In order to minimize capital investment, product water would be blended with untreated well water prior to disinfection. The resulting blend of 70 percent treated water and 30 percent untreated water was projected to meet the applicable standards.

In addition to the installation of the reverse osmosis treatment system, a series of other improvements to the system were accomplished. These included:

- Installation of a seal on the existing well to meet state requirements;
- Construction of two evaporation ponds to dispose of reject water from the reverse osmosis system;
- Installation of home water meters;

- Installation of a pressure tank to meet Texas requirements;
- Installation of a new distribution system from the pressure tank to each customer;
- Construction of a new fire protection system including lines, hydrants, and a 400 gallon per minute fire pump;
- Cleaning of the existing storage tank;
- Construction of a new pump station ; and
- Construction of a new 20,000 gallon ground storage tank.

The project was completed in late 1990. The total cost of the project was $336,060. User rates in Los Ybanez average $1.38/1,000 gallons for residential users.

RESULTS AND SUMMARY

The Town's drinking water was sampled by the Texas Department of Health in January 1991. The results of this sampling are shown in Table 1. Compliance with all parameters except fluoride, sulfate, and selenium has been achieved. Discussion with State personnel who have inspected the new system indicates they believe that by decreasing the percentage of raw water in the blend, compliance with all Texas Health Department standards can be achieved.

Los Ybanez is typical of small, rural communities which do not have either the financial or technical resources to comply with increasingly stringent drinking water requirements. By working with CRG, the Town was able both to obtain the necessary funding and to develop a suitable corrective action program. In addition to providing safe water, fire protection for the community was enhanced. This was achieved in less than two years, a short period given the extensive upgrade to the system and the need to obtain funding from outside the community. Prompt action by the Town avoided an enforcement action by the State of Texas which could have delayed compliance. Table 2 summarizes the key elements of Los Ybanez's reponse to solve its drinking water problems.

Table 1. Drinking Water Quality Los Ybanez, TX

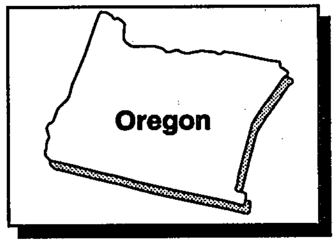
Parameter	Raw Well Water (mg/L)	Jan. 1991 Sample (mg/L)	SDWA Standard (mg/L)
Chloride	300	165	250
Flouride	4.8	2.7*	2
Nitrate (as N)	7.41	5.31	10
Sulfate	540	315*	250
pH	8.1	6.9	6.5-8.5
TDS	1458	823	1000
Arsenic	0.019	0.011	0.05
Barium	<0.05	<0.02	1
Cadmium	<0.005	<0.005	0.005
Chromium	<0.02	<0.02	0.05
Copper	<0.02	0.13	1.3
Iron	0.42	0.03	0.3
Lead	<0.02	<0.02	0.05
Manganese	<0.02	<0.02	0.05
Mercury	0.0005	0.0002	0.002
Selenium	0.029	0.016*	0.01
Silver	<0.01	<0.01	0.05
Zinc	0.03	0.08	5

*Concentration in violation of SDWA standards.

Table 2. Key Elements of Los Ybanez's Successful Project Resolution

- ✓ Assistance from CRG
- ✓ Successful pursuit of state funding
- ✓ Selection of an engineering firm with small-system experience
- ✓ Selection of a low operating cost treatment alternative by comparing vendor estimates with actual costs experienced by neighboring communities

WESTFIR, OREGON
CASE STUDY #5



BACKGROUND

Westfir is a small community located in Lane County about 40 miles southeast of Eugene, Oregon. The current population is approximately 310. The community was originally established to house workers from a nearby lumber mill. The City was incorporated in 1981 to improve its ability to provide municipal services to the community's residents.

In 1946, the lumber company constructed a drinking water system for the community. The system used a surface water supply and included chlorination, a storage reservoir, and a distribution system. Although the original water system supplied the community with a sufficient quantity of water, it became apparent during the late 1970s that the existing treatment system could not consistently comply with the turbidity maximum contaminant level (MCL) of the Safe Drinking Water Act (SDWA).

Violation notices issued at first by the Oregon Drinking Water Program (DWP) and later by the U.S. Environmental Protection Agency (EPA) culminated in the City agreeing to a compliance schedule to address the turbidity MCL violations. This case study discusses the actions the community of Westfir took to comply with the turbidity MCL of the SDWA.

COMMUNITY RESPONSE

Like most small communities, Westfir did not have the technical or financial resources to select an appropriate treatment technology and to obtain adequate funding to comply with the compliance schedule. Therefore, the community turned to the Lane County Housing Authority and Community Services Agency (HACSA) for help in identifying a treatment option and applying for financial assistance. The Lane County HACSA is a quasi-municipal agency that operates public housing at the local level and provides technical assistance to help rural communities solve problems that they do not have the resources to address on their own. The Rural Community Assistance Corporation (RCAC) was also instrumental in assisting Westfir. RCAC provided funding to Westfir through a small project development grant that enabled the community (with HACSA's assistance) to prepare an engineering report and an application for a Community Development Block Grant (CDBG). No local funds were available to prepare the application for the CDBG.

The City was successful in obtaining a $294,000 CDBG to upgrade the existing treatment system. Lane County HACSA contracted with Westfir to provide assistance for the duration of the project. These services included preparation of the Request for Proposals (RFP) for engineering services, development of bid documents and specifications, construction oversight, and grant closeout functions. The participation of the Lane County HACSA from project inception to completion was a key element in ensuring the successful completion of this project within the available funding allotment.

EVALUATION OF ALTERNATIVES

The engineering report submitted with the CDBG application had specified a package pressure filtration system to reduce turbidity levels. However, this design was ultimately rejected for the following reasons:

- For small community systems, the Oregon DWP favored a treatment system that could be more readily observable, in a physical sense, rather than the proposed closed pressure filters.
- The proposed filtration system would require regular attendance by a properly trained operator. Experience with similar types of systems in Oregon's rural communities indicated that this was not readily achievable due to limited financial resources and high rates of operator turnover.

An alternative water supply source (i.e., groundwater) was not feasible due to limited groundwater resources in the area and the potential for natural arsenic contamination due to volcanic rock formations.

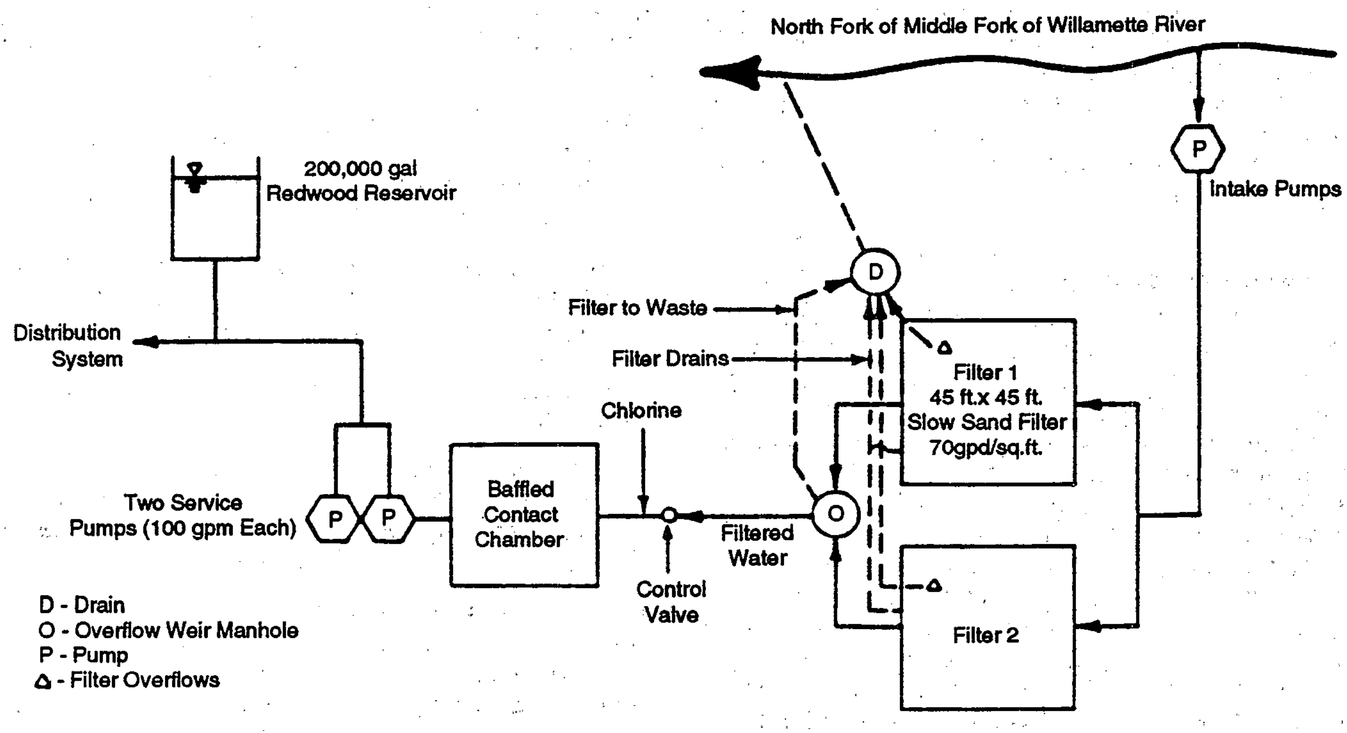
Unable to use a groundwater supply, the consulting engineer retained by the City proposed a slow sand filter to meet Westfir's requirements. The consultant had prior experience with both small communities and slow sand filter applications. Slow sand filters do not have complex operation and maintenance requirements; similarly, they do not have complicated design requirements. During the early 1980s, however, slow sand filters were still a relatively unproven technology. Therefore, the State approved the project contingent on the results of a pilot filter study. The compliance schedule was renegotiated with the EPA to permit adequate time for the pilot study. The Oregon DWP, which had decided in the early 1980s to encourage the use of alternative and simpler treatment technologies for small communities, assumed an active role in the study. The DWP supplied a pilot apparatus previously used in another study as well as technical assistance and analytical support. The pilot study was conducted during an 8-month period. The duration of the study ensured that the pilot system was subjected to any potential seasonal fluctuations in water quality, such as temperature, turbidity, color, and suspended solids. The pilot system was monitored daily for temperature, turbidity, filter head loss, and flow. The weather and river conditions were also monitored. Periodically, samples for total and fecal coliform analyses were collected and shipped to the state laboratory in Portland. The filter media utilized in the pilot study was slightly "out-of-spec." However, the engineer decided to use it because abundant supplies were available locally and at a reasonable cost. The results obtained at the end of the pilot study indicated that the proposed system would consistently achieve the turbidity MCL of 1 nephelometric turbidity unit (NTU), and construction approval was granted by the Oregon DWP.

DESIGN CRITERIA

Figures 1 and 2 present plan and cross-sectional diagrams of the installed treatment system. The total cost of the treatment system and distribution system improvements was $217,483 (1986 dollars), exclusive of engineering and grant administration fees.

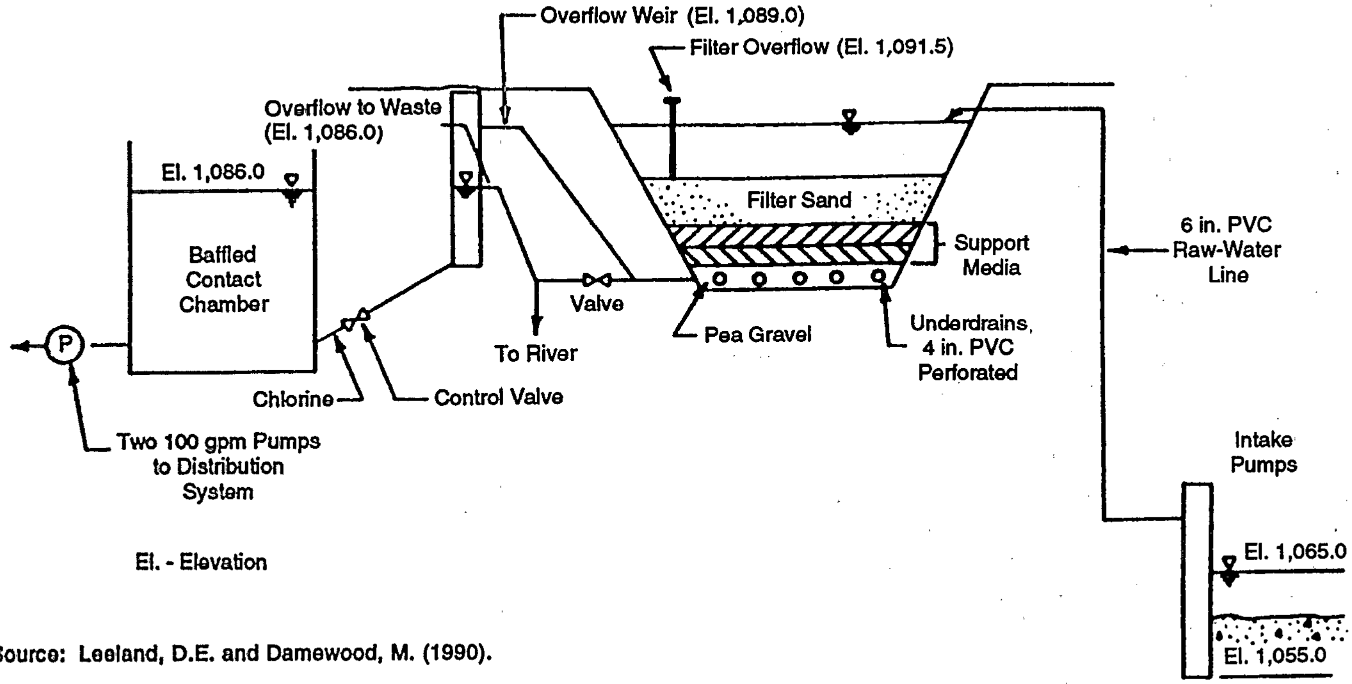
The main treatment system components, summarized in Table 1, consist of the following:

- The raw water intake consists of two, 2 horsepower (HP), submersible, 100 gallons per minute (gpm) pumps at a total dynamic head (TDH) of 50 feet. The pumps were installed in an infiltration-type wet well located immediately adjacent to the Willamette River.
- Filtration is provided by two 45 square foot (ft^2) slow sand filters designed to operate at 70 gallons per day per square foot (gpd/ft^2). To control costs, the filters were constructed with earth berms and elastomeric membrane liners rather than with reinforced concrete walls and bottoms. The filter underdrain system is comprised of 4-inch PVC pipe. The filter bed consists of a 30-inch graded gravel support layer and a 3-foot layer of filter sand. The filter sand media has a uniformity coefficient of 2.32, an effective size of 0.315 mm. A control manhole distributes filtered water either to the chlorine contact tank or, during periods of low



Source: Leeland, D.E. and Damewood, M. (1990).

Figure 1. Westfir slow sand filtration plant schematic plan.



Source: Leeland, D.E. and Damewood, M. (1990).

Figure 2. Westfir slow sand filtration plant schematic profile.

demand, to the river. An overflow weir inside the control manhole maintains a minimum water level just above the surface of the sand. This prevents accidental dewatering of the filter, which could adversely affect the microbiological population present at the surface of the filter.

- Filtered water is chlorinated by feeding sodium hypochlorite solution prior to the chlorine contact tank.
- Two 100 gpm centrifugal pumps were installed to pump treated water to the distribution system. Storage is provided by an existing 200,000 gallon redwood reservoir.

In addition to the treatment system project, a portion of the grant funds were used to replace some of the distribution system steel water mains to reduce water loss from leaking mains.

RESULTS AND SUMMARY

Operation of the treatment system commenced in November 1986. During the first year of operation, filtered water turbidity levels were slightly higher than the raw water levels due to the presence of excessive fine particles in the sand media. This problem demonstrates the need to thoroughly wash the sand media prior to installation.

As expected, operating requirements have been minimal. Daily equipment and process control checks (turbidity, chlorine residual) are performed by the operator in approximately 60 minutes. The turbidity of the river is also monitored daily. If the river turbidity levels become excessive, the raw water intake pumps are shut down until the turbidity levels subside or the reservoir level becomes too low. The reservoir contains about a 3-day supply of

Table 1. Key Design Features of the Westfir, Oregon, Slow Sand Filtration Installation

- Raw Water Intake Pumps - Two, 2 HP, 100 gpm Submersible Pumps
- Slow Sand Filters
  - — Two 45 ft^2 Filters
  - — Hydraulic Loading of 70 gpd/ft^2
  - — Sand Media (uniformity coefficient of 2.32 and effective size of 0.315 mm)
- Sodium Hypochlorite Chlorination System
- Treated Water Supply Pumps - Two 100 gpm Centrifugal Pumps

water for the community. Each filter is cleaned twice per year. One filter remains in service while the other one is cleaned. Experience has indicated that it takes about 36 hours to drain and dry out the filter. Two men use flat-bladed shovels to manually scrape the filter in about 7 hours. Approximately 1 inch of sand is removed to clean the filters. The removed sand is not cleaned for reuse; it is disposed of onsite.

The raw water submersible intake pumps were replaced by a single 200 gpm centrifugal pump. Apparently, the intake screens to the submersible pumps required frequent cleaning due to debris in the river. During periods of high river water, it was not possible to raise the pumps to remove the debris. Pump removal was also a problem due to limited manpower resources.

The requirements of the EPA compliance schedule were satisfied by the installation of a slow sand filter system. Although the capital cost for slow sand filtration is about the same as for package filtration plants, the operation and maintenance requirements are significantly less for slow sand filters. Westfir currently charges system users $20.00 per month to cover operation and maintenance expenses, utility charges, and water analyses. The rate was recently increased from $12.50 per month due to additional testing requirements recently implemented by the State. A summary of the key elements that contributed to the success of this project is presented in Table 2.

Table 2. Key Elements of Westfir's Successful Project Resolution

- ✓ Assistance from Lane County Housing Authority and Community Services Agency during entire project
- ✓ Funding provided by Rural Community Assistance Corporation to prepare CDBG application
- ✓ Project funded by CDBG
- ✓ Engineer experienced with small-community systems and alternative treatment technologies for small communities
- ✓ State regulatory agency supportive of alternative treatment technologies for small communities
- ✓ Performance of pilot study to adequately assess treatment technology capabilities under various water quality conditions
- ✓ Selection of a treatment system with low operation and maintenance requirements

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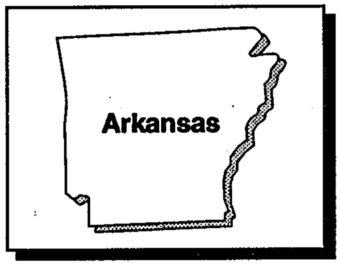
Leeland, D.E. and Damewood, M. "Slow Sand Filtration in Small Systems in Oregon." *Journal American Water Works Association* 82 (1990):50-59.

MOCKINGBIRD HILL, ARKANSAS CASE STUDY #6



BACKGROUND

Mockingbird Hill is a very small, unincorporated community in northwestern Arkansas. This group of residences and businesses is located in Newton County within the Ozark National Forest. Tourism is the dominant industry in this region.

By the early 1970s, the Mockingbird Hill area had grown to include approximately 90 residences and businesses. Historically, one of the impediments to development in the area had been limited drinking water sources. To foster further development, the residents of Mockingbird Hill decided to establish a centralized water supply. To accomplish this task, the Mockingbird Hill Water Association (MHWA) was formed, and Farmers Home Administration (FmHA) grant funding and loans were secured. A 3,200 foot deep well was drilled and a distribution system constructed. The deep well and rough terrain of the area contributed to the substantial $1.6 million cost of the project.

From the time service was established, there were problems with the quality of water produced by the Association's well. The water was high in hydrogen sulfide, dissolved and suspended solids, and color. To mask the unpleasant odors in the water, high chlorine dosages were routinely applied. Together, the raw water characteristics and the high chlorine dosage resulted in undrinkable water.

The residents of Mockingbird Hill generally used bottled water for drinking until the opportunity arose to tie into the nearby Deer-Wayton Water Association supply. This connection was made in 1982, and the MHWA was absorbed into the Deer-Wayton Water Association.

Joining the Deer-Wayton Water Association did not solve Mockingbird Hill's water supply problems, however. Water quality was substantially improved, but drought conditions in 1983, 1984, and 1985 resulted in water shortages for the Deer-Wayton system.

This case study examines steps taken by the residents of Mockingbird Hill to address their water supply problem and evaluates the performance of the technical solutions that were ultimately selected.

COMMUNITY RESPONSE

The drought-related shortages that occurred after the connection to the Deer-Wayton Water Association supply resulted from a series of shallow (less than 200 ft.) wells that supplied Deer-Wayton's system. These shortages caused a number of actual shutdowns of the supply from Deer-Wayton to Mockingbird Hill, which convinced numerous residents of Mockingbird Hill that Deer-Wayton was not the solution to their problem. As a result, several of the residents took the lead in re-establishing the MHWA.

The reborn Association retained a consulting engineer to test the water in the original deep well to determine if it could be economically treated to an acceptable quality. The Association and it's engineer also evaluated the condition of the well pumping system, which had been abandoned in place when the connection to Deer-Wayton occurred. The MHWA received significant assistance in both these tasks from a circuit rider with the Arkansas Rural Water Association (ARWA).

When the results of the engineering evaluation were received, the MHWA held a series of meetings that included the Association Board, the consulting engineer, and the County Health Department. As a result of these meetings, it was decided that the MHWA would rehabilitate the existing well and construct a treatment system to address the raw water quality problems. The president of MHWA, with the support of several local congressmen and senators, lobbied successfully to secure $600,000 in grant funding (FmHA). An additional $350,000 of long-term, low-interest loans were arranged. In addition, the MHWA Board, the consulting engineer, and the County Board of Health held discussions to determine the appropriate level of treatment.

EVALUATION OF ALTERNATIVES

Mockingbird Hill's engineer did not formally evaluate multiple alternative treatment systems. Instead, the engineer made recommendations based on recent experiences in dealing with low-quality drinking water sources.

Onsite testing at Mockingbird Hill revealed a hydrogen sulfide concentration of approximately 20 mg/L. Based on previous experience, the consulting engineer recommended that the hydrogen sulfide be removed using an air stripping tower.

In addition, excessive amounts of iron and manganese were present. The engineer recommended alum precipitation and filtration to remove these remaining contaminants. Given the very small flow to be treated and the desire that the selected treatment system require limited attention, the engineer recommended using a package-type water treatment system. The engineer had recently had a very positive experience with this proprietary water treatment system.

SELECTION OF DISTRIBUTION AND TREATMENT SYSTEMS

As noted above, Mockingbird Hill's engineer recommended installing an air stripping system, degassed water storage, and a package precipitation/filtration treatment system.

DESIGN CRITERIA

The design criteria for the treatment system are discussed below. Table 1 presents design characteristics of the selected treatment system.

The air stripping system was specified to provide 85-percent removal of hydrogen sulfide. The design capacity was specified as 100 gpm, with a design air throughput of 1,300 cfm. The resulting tower, which is approximately 3 feet in diameter and 10 feet tall, is situated on top of a stripped water storage tank.

Table 1. Key Design Features of the Selected Treatment System

- Air Stripping Tower: 100 gpm, 1,300 cfm Air Throughput
- Stripped Water Storage: 13,000 Gallons
- Package Precipitation System:
  - Automatic Chemical Feed System
  - Alum and Polymer
  - Upflow Adsorption Clarifier
  - Downflow Filter, 20 sq.ft.
  - Effluent Turbidimeter

The proprietary skid mounted treatment system incorporates chemical feed and mixing, an upflow adsorption clarifier, and gravity multi-media filtration. The nominal capacity rating by the manufacturer is 100 gpm; however, the State of Arkansas rates the unit at 60 gpm, based on a filter loading criterion of 3 gpm/ft^2.

Figure 1 illustrates the system installed to treat Mockingbird Hill's water supply.

COST REDUCTION

Cost-reduction efforts appear to have been limited in Mockingbird Hill's attempt to address its water supply problem. This can probably be attributed largely to the severity of the water shortage problem and to the technical difficulties posed by treating Mockingbird Hill's water supply.

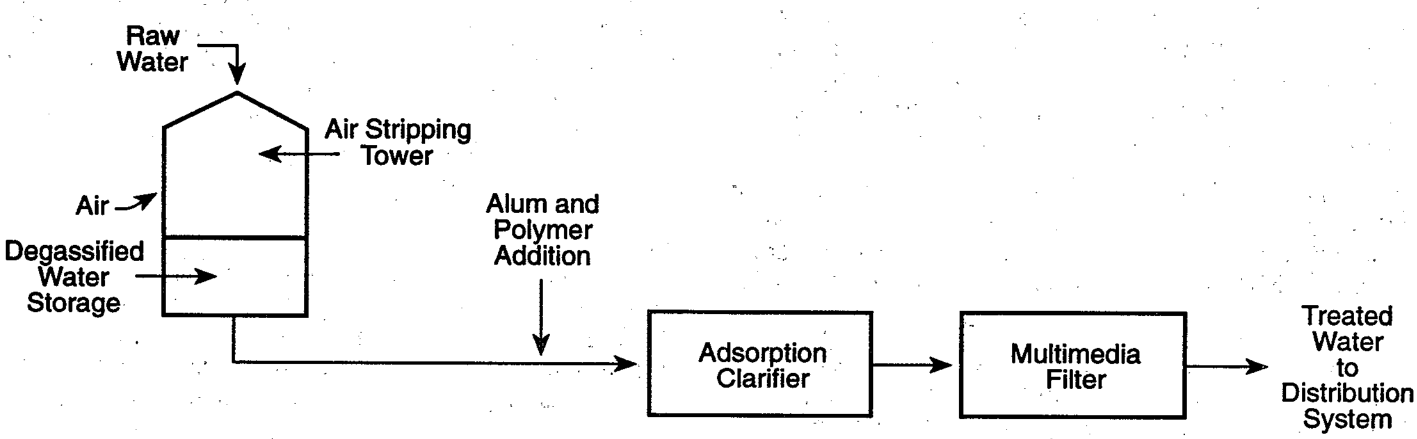


Figure 1. Plant schematic for Mockingbird Hill Water Association.

RESULTS AND SUMMARY

Table 2 summarizes the key elements of Mockingbird Hill's response to solve its drinking water problem. Since start-up, hydrogen sulfide removals have averaged 90 percent according to the system operator. The operator noted that at the present time, alum and polymer are not being added to the degassified water. Instead, chlorine is injected into the degassified water as it is pumped from the storage tank to the package treatment system. The chlorine is continuously applied at a dose of 4 mg/L. This dosage results in additional removal of hydrogen sulfide and the oxidation of iron and manganese. The insoluble precipitate resulting from the oxidation of iron and manganese is removed by settling and filtration in the package plant.

An online turbidimeter monitors the quality of the treated effluent. The turbidimeter regulates the application rate of coagulants when used. At preset turbidimeter levels, the package system will either automatically initiate a backwash cycle or shut-down (after a preselected time delay) if excessive turbidity levels are measured after the backwash cycle has been completed.

Since the installation of the treatment system, the service area has been expanded to 165 customers. Nonetheless, the debt service associated with the non-grant funded portion of the capital cost of the treatment system has been difficult for Mockingbird Hill to bear. The present monthly user charge is $16.25 for the first 1,000 gallons and $2.75 for each additional 1,000 gallons used. The average water use per customer is 3,000 gallons/month.

In addition to the financial difficulties, radium was detected in Mockingbird Hill's treated water after the treatment system was started up. While ongoing testing appears to indicate that concentrations of radium are below threshold levels, this issue has provoked significant concern on the part of the customers served.

Table 2. Key Elements of Mockingbird Hill's Experience

- ✓ Effort "championed" by a concerned citizen
- ✓ Assistance provided by Arkansas Rural Water Association
- ✓ All available financial assistance options successfully pursued
- ✓ Engineer knowledgeable of local water conditions
- ✓ Effective treatment technologies and equipment selected

RESOURCE DIRECTORY

U.S. ENVIRONMENTAL PROTECTION AGENCY HEADQUARTERS AND REGIONAL OFFICES

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Washington, DC 20460
202-260-2090

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212-264-2657

New Jersey, New York, Puerto Rico, Virgin Islands

Region 3
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Philadelphia, PA 19107
215-597-9800

Delaware, Maryland, Pennsylvania, Virginia, West Virginia, District of Columbia

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Atlanta, GA 30365
404-347-4727

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Dallas, TX 75202
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913-551-7000

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Denver, CO 80202
303-293-1603

Colorado, Montana, North Dakota, South Dakota, Utah, Wyoming

Region 9
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San Francisco, CA 94105
415-744-1305

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206-553-4973

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Contact: Mr. James Bailey
(501) 574-4550

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Hastings, NE 68902
Contact: Mr. Greg DuMonthier
(402) 461-2481

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Lakewood, CO 80401-5398
Contact: Mr. Thomas Feeley
(303) 980-9165

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Contact: Ms. Lorene Lindsay
(417) 451-3583

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Murfreesboro, TN 37129
Contact: Mr. James Coe
(615) 898-8090

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Institute
301 Old Hickory Trail North
Carrollton, GA 30117
Contact: Mr. John Henderson
(404) 836-6656

Kentucky Division of Waste Man-
agement
18 Reilly Road
Frankfort, KY 40601
Contact: Ms. Lisa Detherage
(502) 564-6716

Kirkwood Community College
6301 Kirkwood Blvd. S.W.
P.O. Box 2068
Cedar Rapids, IA 52406-2068
Contact: Mr. Douglas Fell
(319) 398-5678

Linn-Benton Community College
6500 S.W. Pacific Blvd.
Albany, OR 97321
Contact: Dr. John Carnegie
(503) 928-2361

Massachusetts DEP Training Center
Route 20
Milbury, MA 01527
Contact: Mr. Kimball Simpson
(508) 756-7281

New England Interstate ETC
2 Fort Road
South Portland, ME 04106
Contact: Mr. Kirk Laflin
(207) 767-2639

NREP Cabinet-Division of Water
18 Reilly Road
Frankfort, KY 40601
Contact: Ms. Nancy Fouser

Operator Training Committee of
Ohio
3972 Indianola Ave.
Columbus, OH 43214-3158
Contact: Mr. John McCraight
(614) 268-6826

Southern Illinois University
Environmental Training Center
Edwardsville, IL 62025
Contact: Mr. Donald Anderson

Sumter Area Technical College
Environmental Training Center
506 North Guignard Drive
Sumter, SC 29150
Contact: Dr. William Engel
(803) 778-1961

TREEO Center, Univ. of Florida
3900 S.W. 63rd. Blvd.
Gainesville, FL 32608-4830
Contact: Dr. James Bryant
(904) 392-9570

Utah Valley Community College
800 West 1200 South
Orem, UT 84058
Contact: Ms. Lasca Rose
(801) 226-5000

Washington Environmental Training
Center
12401 S.E. 320th St.
Auburn, WA 98002
Contact: Mr. Fred Delvecchio
(206) 833-9111

RURAL COMMUNITY ASSISTANCE PROGRAM

NATIONAL OFFICE

Rural Community Assistance Program (RCAP)
602 South King Street, Suite 402
Leesburg, VA 22075
Phone: (703) 771-8636
Fax: (703) 777-8753

NORTHEASTERN REGION

Rural Housing Improvement (RHI)
Earnest Beresh
218 Central Street
Winchendon, MA 01475
Phone: (508) 297-1376
Fax: (508) 297-2606

SOUTHEASTERN REGION

Virginia Water Project (VWP)
Mary Terry
1314 Peter's Creek Road #210
Roanoke, VA 24017
Phone: (703) 345-1184
Fax: (703) 342-2932

GREAT LAKES REGION

WSOS Community Action Commission (WSOS)
Orville Burch
109 S. Front Street
Fremont, OH 43420
Phone: (419) 334-8911
Fax: (419) 334-5125

MIDWESTERN REGION

Midwest Assistance Program (MAP)
Ken Bruzelius
P.O. Box 81
New Prague, MN 56071
Phone: (612) 758-4334
Fax: (612) 758-4336

SOUTHERN REGION

Community Resource Group (CRG)
Mark Rounsavall
2705 Chapman Road
Springdale, AR 72764
Phone: (501) 756-2900
Fax:(501) 756-2901

WESTERN REGION

Rural Community Assistance Corporation (RCAC)
Beth Ytell
2521 19th Street, #203
Sacramento, CA 95818
Phone: (916) 447-2854
Fax: (916) 447-2878

U.S. DEPARTMENT OF AGRICULTURE
STATE EXTENSION SERVICE DIRECTORS

Alabama Cooperative Extension
Service
109 Duncan Hall
Auburn University, AL 36849-5612
205/844-4444
Dr. Ann Thompson

University of Alaska
103 AHRB
Fairbanks, AK 99775-5200
907/474-7246
Dr. Irvin Skelton

College of Agriculture
University of Arizona
Forbes Bldg., Rm. 301
Tucson, AZ 85721
602/621-7209
Dr. James Christenson

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P.O. Box 391
Little Rock, AR 72203
501/671-2001
Dr. David Foster

University of California
Office of the Vice President
Agric. & Nat. Resources
300 Lakeside Dr., 6th Fl.
Oakland, CA 94612-3560
510/987-0060
Dr. Kenneth Farrell

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1 Administration Bldg.
Fort Collins, CO 80523
303/491-6208
Mr. Milan Rewerts

College of Agriculture and
Natural Resources
University of Connecticut
Box U-66, 1376 Storrs Road
Storrs, CT 06269-4066
203/486-4125
Dr. John Brand

University of Delaware
Townsend Hall, Rm 131
South College Avenue
Newark, DE 19717-1303
302/831-2504
Dr. Richard Fowler

University of the District of Columbia
901 Newton St., N.E.
Washington, DC 20017
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Mr. Reginald Taylor

University of Florida
1038 McCarthy Hall
Gainesville, FL 32611
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University of Georgia
Room 111, Conner Hall
Athens, GA 30602
404/542-3824
Dr. C. Wayne Jordan

University of Hawaii
3050 Maile Way
Gilmore 202
Honolulu, HI 96822
808/956-8234
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University of Idaho
Agricultural Science Building
Moscow, ID 83843
208/885-6639
Dr. LeRoy Luft

University of Illinois
122 Mumford Hall
1301 West Gregory Drive
Urbana, IL 61801
217/333-2660
Dr. Donald Uchtmann

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104 Agriculture Admin. Bldg.
West Lafayette, IN 47907
317/494-8489
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Iowa State University
315 Beardshear Hall
Ames, IA 50011
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Waters Hall, Room 113
Manhattan, KS 66506
913/532-7137
Dr. Walter Woods

University of Kentucky
Ag. Science Building N
Lexington, KY 40546-0091
606/257-4772
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Louisiana State University
Knapp Hall
Baton Rouge, LA 70803-1900
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Dr. Denver Loupe

University of Maine
102 Libby Hall
Orono, ME 04469
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3300 Metzerott Road
Adelphi, MD 20783
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Dr. Craig S. Oliver

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Amherst, MA 01003
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1420 Eckles Avenue
St. Paul, MN 55108
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Box 5446
201 Bost Extension Center
Mississippi St., MS 39762
601/325-3036
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University of Missouri
309 University Hall
Columbia, MO 65211
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210 Montana Hall
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214 Agriculture Hall
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Mail Stop 222
Reno, NV 89557-0004
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University of New Hampshire
Taylor Hall, Room 103
Durham, NH 03824
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New Brunswick, NJ 08903
908/932-9306
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New Mexico State University
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Las Cruces, NM 88003
505/646-3015
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276 Roberts Hall
Ithaca, NY 14853-5901
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919/515-2811
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311 Morrill Hall
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Dr. William H. Pietsch

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2120 Fyffe Road
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201 Ag. Admin. Bldg.
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113 Woodward Hall
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103 Barre Hall
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South Dakota State University
P.O. Box 2207D, Ag. Hall 154
Brookings, SD 57007-9988
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University of Tennessee
Box 1071
Knoxville, TN 37901-1071
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Texas A&M University
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Room 106
College Station, TX 77843-7101
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Logan, UT 84322-4900
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STATE EXTENSION SERVICE DIRECTORS (Continued)

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108 Morrill Hall
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Blacksburg, VA 24061-0402
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411 Hulbert Hall
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817 Knapp Hall
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University of Wisconsin Extension
432 N. Lake Street
Room 527, Extension Building
Madison, WI 53706
608/262-3786
Dr. Patrick G. Boyle

College of Agriculture
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P.O. Box 3354
Laramie, WY 82071
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Mr. Jim DeBree

National Association of State Universities & Land Grant Colleges
One Dupont Circle, Suite 710
Washington, DC 20036
202/778-0829
Dr. Robert L. Crom

NATIONAL RURAL WATER ASSOCIATION
STATE ASSOCIATIONS & STATE AFFILIATES

National Rural Water Association
P.O. Box 1428
Duncan, OK 73534
405/252-0629

Alabama Rural Water Assn.
4556 South Court St.
Montgomery, AL 36105
205/284-1489

Arizona Small Utilities Assn.
1955 W. Grant Road, Suite 125
Tucson, AZ 85745
602/620-0230

Arkansas Rural Water Assn.
P.O. Box 192118
Little Rock, AR 72219
501/568-5252

California Rural Water Assn.
216 W. Perkins St., Suite 204
Ukiah, CA 95482
707/462-1730

Colorado Rural Water Assn.
2648 Santa Fe Drive, #10
Pueblo, CO 81006
719/545-6748

Connecticut & Rhode Island Rural Water
11 Richmond Lane
Willimantic, CT 06226-3825
203/423-6737

Delaware Rural Water Assn.
P.O. Box 118
Harrington, DE 19952
302/398-9633

Florida Rural Water Assn.
1391 Timberlane Rd., Suite 104
Tallahasee, FL 32312
904/668-2746

Georgia Rural Water Assn.
P.O. Box 383
Barnesville, GA 30204
404/358-0221

Idaho Rural Water Assn.
P.O. Box 303
Lewiston, ID 83501
208/742-6142

Illinois Rural Water Assn.
401 South Vine
Mt. Pulaski, IL 62548
217/792-5011

Indiana Water Assn.
P.O. Box 103
Sellersburg, IN 47172
812/246-4148

Iowa Rural Water Assn.
1300 S.E. Cummins Rd., Suite 103
Des Moines, IA 50315
515/287-1765

Kansas Rural Water Assn.
P.O. Box 226
Seneca, KS 66538
913/336-3760

Kentucky Rural Water Assn.
P.O. Box 1424
Bowling Green, KY 42102-1424
502/843-2291

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318/738-2896

Maine Rural Water Assn.
14 Maine St., Suite 407
Brunswick, ME 04011
207/729-6569

Maryland Rural Water Assn.
P.O. Box 207
Delmar, MD 21875
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P.O. Box 17
Auburn, MI 48611
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RR 2, Box 29
Elbow Lake, MN 56531
218/685-5197

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P.O. Box 1995
Hattiesburg, MS 39403-1995
601/544-2735

Missouri Rural Water Assn.
P.O. Box 309
Grandview, MO 64030
816/966-1522

Montana Rural Water Systems Assn.
925 7th Avenue South
Great Falls, MT 59405
406/454-1151

Nebraska Rural Water Assn.
Route 3, Box 115
Humboldt, NE 68376
402/862-3140

NATIONAL RURAL WATER ASSOCIATION
STATE ASSOCIATIONS & STATE AFFILIATES (Continued)

Nevada Rural Water Assn.
P.O. Box 837
Overton, NV 89040
702/397-8985

New Jersey Assn. of Rural Water & Wastewater Ut.
RD 1 Route 72
Vincentown, NJ 08088
609/894-2750

New Mexico Rural Water Users Assn.
3218 Silver, S.E.
Albuquerque, NM 87106
505/255-2242

New York State Rural Water Assn.
P.O. Box 487
Claverack, NY 12513
518/851-7644

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P.O. Box 540
Welcome, NC 27374
704/731-6963

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Route 1, Box 34C
Bismarck, ND 58501
701/258-9249

Northeast Rural Water Assn.
512 St. George Road
Williston, VT 05495
802/878-3276

Ohio Assn. of Rural Water Systems
P.O. Box 397
Grove City, OH 43123
614/871-2725

Oklahoma Rural Water Assn.
1410 S.E. 15th
Oklahoma City, OK 73129
405/672-8925

Oregon Assn. of Water Utilities
1290 Capitol Street NE
Salem, OR 97303
503/364-8269

Pennsylvania Rural Water Assn.
P.O. Box 90
Saltsburg, PA 15681-0090
412/639-3246

South Carolina Rural Water Assn.
P.O. Box 479
Clinton, SC 29325
803/833-5566

South Dakota Assn. of Rural Water Systems
5009 W. 12th Street, Suite 5
Sioux Falls, SD 57106
605/336-7219

Tennessee Assn. of Utility Districts
P.O. Box 2529
Murfreesboro, TN 87133-2529
615/896-9022

Texas Rural Water Assn.
6300 Lacalma, Suite 120
Austin, TX 78752
512/458-8121

Rural Water Assn. of Utah
P.O. Box 661
Spanish Fork, Utah 84660
801/798-3518

Virginia Rural Water Assn.
133 West 21st Street
Buena Vista, VA 24416
703/261-7178

Washington Rural Water Assn.
P.O. Box 141588
Spokane, WA 99214-1588
509/924-5568

West Virginia Rural Water Assn.
P.O. Box 225
Teays, WV 25569
304/757-0985

Wisconsin Rural Water Assn.
2715 Post Road (Whiting)
Stevens Point, WI 54481
715/344-7778

Wyoming Assn. of Rural Water Systems
P.O. Box 1750
Glenrock, WY 82637
307/436-8636

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Environmental Protection Agency
Center for Environmental Research Information
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