United States
Environmental Protection
Agency

Office of Research and
Development
Cincinnati, OH 45268

EPA/625/R-95/008
April 1996

EPA

AMERICAN SOCIETY OF
CIVIL ENGINEERS

AMERICAN WATER WORKS ASSOCIATION
6666 W. QUINCY AVE., DENVER CO 80235

Technology Transfer Handbook

Management of Water Treatment Plant Residuals

EPA/625/R-95/008
April 1996

Technology Transfer Handbook

Management of Water Treatment Plant Residuals

U.S. Environmental Protection Agency
Office of Research and Development
National Risk Management Research Laboratory
Cincinnati, Ohio

American Society of Civil Engineers
New York, New York

American Water Works Association
Denver, Colorado

Notice

The U.S. Environmental Protection Agency (EPA) through its Office of Research and Development partially funded and collaborated in the research described here under EPA Contract 68-C3-0315 to Eastern Research Group, Inc. It has been subjected to the Agency's peer and administrative review and has been approved for publication as an EPA document.

The material presented in this publication has been prepared in accordance with generally recognized engineering principles and practices and is for general information only. This information should not be used without first securing competent advice with respect to its suitability for any general or specific application.

The contents of this publication are not intended to be a standard of the American Water Works Association (AWWA) or the American Society of Civil Engineers (ASCE) and are not intended for use as a reference in purchase specifications, contracts, regulations, statutes, or any other legal document.

No reference made in this publication to any specific method, product, process, or service constitutes or implies an endorsement, recommendation, or warranty thereof by AWWA, ASCE, or EPA. AWWA, ASCE, and EPA make no representation or warranty of any kind, whether expressed or implied, concerning the accuracy, product, or process discussed in this publication and assume no liability. Anyone using this information assumes all liability arising from such use, including but not limited to infringement of any patent or patents.

Contents

Contents (continued)

Page

Contents (continued)

Page

Contents (continued)

Page

Contents (continued)

Page

Contents (continued)

Page

Contents (continued)

Page

Contents (continued)

Contents (continued)

Figures

Figure **Page**

Figures (continued)

Figure **Page**

Figures (continued)

Figure **Page**

Figures (continued)

Figure **Page**

Tables

Table **Page**

Tables (continued)

Table **Page**

Tables (continued)

Tables (continued)

Tables (continued)

Conversion Factors

To convert . . .	**to . . .**	**multiply by . . .**
acres	hectares	0.4046944
cubic feet	cubic meters	0.02831685
degrees Fahrenheit	degrees Celsius	$t_{^\circ C} = (t_{^\circ F} - 32)/1.8$
feet	meters	0.3048
inches	centimeters	2.54
miles	kilometers	1.609344
ounces	grams	28.3495
pounds	kilograms	0.45354237
pounds per 1,000 gallons	grams per liter	0.1198322
pounds per square inch	kiloPascals	6.895
square inches	square centimeters	6.4516
tons	metric tons	0.90718474
U.S. gallons	liters	3.785

Acknowledgments

The organizations responsible for the development of this handbook are the U.S. Environmental Protection Agency's (EPA's) National Risk Management Research Laboratory (NRMRL), Technology Transfer and Support Division, the American Society of Civil Engineers (ASCE), and the American Water Works Association (AWWA). EPA's Office of Research and Development partially funded this effort under Contracts 68-C0-0068 and 68-C3-0315 with Eastern Research Group, Inc. (ERG). Funding was also provided by ASCE.

Many individuals participated in the preparation and review of this handbook. Below is a partial list of the major contributors.

Chapter 1: Introduction

Primary author: Jerry Russell, John Carollo Engineers, Phoenix, AZ. *Contributing authors:* Carl P. Houck, Camp Dresser & McKee Inc., Denver, CO; Janine B. Witko, Malcolm Pirnie, Inc., Mahwah, NJ; Brian Peck, John Carollo Engineers, Phoenix, AZ; and James E. Smith, Jr., EPA NRMRL, Cincinnati, OH.

Chapter 2: Regulatory Issues Concerning Management of Water Treatment Plant Residuals

Primary author: Scott Carr, Black & Veatch, Charlotte, NC. *Contributing authors:* James E. Smith, Jr., EPA NRMRL, Cincinnati, OH; Suzanna I. McMillan, Black & Veatch, Charlotte, NC; and Nancy E. McTigue, Environmental Engineering & Technology, Inc., Newport News, VA. *Key reviewers:* Alan Rubin and Robert Southworth, EPA Office of Water, Washington, DC.

Chapter 3: Characterization of Water Treatment Plant Residuals

Primary author: Timothy A. Wolfe, Montgomery Watson, Inc., Cleveland, OH. *Contributing authors:* Mike Mickley, Mickley and Associates, Boulder, CO; John Novak, Virginia Polytechnic Institute and State University, Blacksburg, VA; Leland Harms, Black & Veatch, Kansas City, MO; and Thomas J. Sorg, EPA NRMRL, Cincinnati, OH.

Chapter 4: Water Treatment Residuals Processing

Primary author: Jerry Russell, John Carollo Engineers, Phoenix, AZ. *Contributing authors:* Brian Peck, John Carollo Engineers, Phoenix, AZ; Tim Stephens, HDR Engineering, Inc., Phoenix, AZ; Duncan Browne, WRC Engineers and Scientists, Huntingdon Valley, PA; Jeannette Semon, City of Stamford WPCF, Stamford, CT; Betsy Shepherd, John Carollo Engineers, Phoenix, AZ; and Dale D. Newkirk, EBMUD-Water Treatment, Oakland, CA.

Chapter 5: Direct Discharge of Water Treatment Plant Residuals to Surface Waters

Primary author: Dennis George, Tennessee Technological University, Cookeville, TN. *Contributing author:* Christina Behr-Andres, University of Alaska, Fairbanks, AK.

Chapter 6: Discharge to Wastewater Treatment Plants

Primary author: Carl P. Houck, Camp Dresser & McKee Inc., Denver, CO. *Contributing authors:* Janine B. Witko, Malcolm Pirnie, Inc., Mahwah, NJ; Timothy A. Wolfe, Montgomery Watson, Inc., Cleveland, OH; Richard Tsang, Camp Dresser & McKee Inc., Raleigh, NC; and Paula Murphy, ERG, Lexington, MA.

Chapter 7: Landfill Options

Primary author: Nancy E. McTigue, Environmental Engineering & Technology, Inc., Newport News, VA. *Contributing authors:* Daniel Murray, EPA NRMRL, Cincinnati, OH; and Mark Wang, The Pennsylvania State University, University Park, PA.

Chapter 8: Land Application

Primary author: Bob Brobst, EPA Region 8, Denver, CO. *Contributing authors:* David R. Zenz and Thomas C. Granato, Metropolitan Water Reclamation District of Greater Chicago, Chicago, IL. *Key reviewers:* Robert Southworth and Alan Rubin, EPA Office of Water, Washington, DC; and Herschel A. Elliott, The Pennsylvania State University, University Park, PA.

Chapter 9: Brine Waste Disposal

Primary author: William J. Conlon, Rust Environment and Infrastructure, Sheboygan, WI. *Contributing authors:* Thomas D. Wolfe, The Palmyra Group, Rough & Ready, CA; and William Pitt, Camp Dresser & McKee Inc., Miami, FL.

Chapter 10: Radioactive Waste Disposal

Primary author: Thomas J. Sorg, EPA NRMRL, Cincinnati, OH. *Key reviewers:* Marc Parrotta, EPA Office of Water, Washington, DC; Nancy E. McTigue, Environmental Engineering & Technology, Inc., Newport News, VA; and Darren Lytle, EPA NRMRL, Cincinnati, OH.

Chapter 11: Economics

Primary author: Bruce Burris, HDR Engineering, Inc., Irvine, CA. *Contributing author:* Christopher Lough, DPRA, St. Paul, MN. *Key reviewer:* Janine B. Witko, Malcolm Pirnie, Inc., Mahwah, NJ.

Chapter 12: Case Studies

Primary author: Balu P. Bhayani, Colorado Springs Utilities, Colorado Springs, CO. *Contributing authors:* James M. Parsons, Cobb County-Marietta Water, Marietta, GA; Cecil Lue-Hing, Metropolitan Water Reclamation District of Greater Chicago, Chicago, IL; and Jerry Russell, John Carollo Engineers, Phoenix, AZ.

Chapter 13: Waste Minimization and Reuse

Primary author: Janine B. Witko, Malcolm Pirnie, Inc., Mahwah, NJ. *Contributing author:* James E. Smith, Jr., EPA NRMRL, Cincinnati, OH.

Peer Reviewers

James H. Borchardt, Montgomery Watson, Inc., Walnut Creek, CA; C. Michael Elliott, Stearns & Wheeler, Cazenovia, NY; Terry L. Gloriod, Continental Water Company, St. Louis, MO; Paul E. Malmrose, Montgomery Watson, Inc., Saddle Brook, NJ; Howard M. Neukrug, Philadelphia Water Department, Philadelphia, PA; S. James Ryckman, University of Dayton, Dayton, OH; James K. Schaefer, Metcalf & Eddy, Inc., Somerville, NJ; J. Edward Singley, Montgomery Watson, Inc., Gainesville, FL; Kevin L. Wattier, Metropolitan Water District, Los Angeles, CA; and Thomas L. Yohe, Philadelphia Suburban Water, Bryn Mawr, PA.

Handbook Development Team

Project directors: James E. Smith, Jr., EPA NRMRL, Cincinnati, OH; and Marla Berman, ASCE, Washington, DC. *Additional coordinators:* Jon DeBoer, AWWA, Denver, CO; Nancy E. McTigue, Environmental Engineering & Technology, Inc., Newport News, VA; and Janine B. Witko, Malcolm Pirnie Inc., Mahwah, NJ. Technical and copy editing and production support were provided by Eastern Research Group, Inc., of Lexington, Massachusetts.

Chapter 1
Introduction

Potable water treatment processes produce safe drinking water and generate a wide variety of waste products known as residuals, including organic and inorganic compounds in liquid, solid, and gaseous forms. In the current regulatory climate, a complete management program for a water treatment facility should include the development of a plan to remove and dispose of these residuals in a manner that meets the crucial goals of cost effectiveness and regulatory compliance. The development of a comprehensive water treatment residuals management plan typically involves each of the following steps:

- Characterize the form, quantity, and quality of the residuals.
- Determine the appropriate regulatory requirements.
- Identify feasible disposal options.
- Select appropriate residuals processing/treatment technologies.
- Develop a residuals management strategy that meets both the economic and noneconomic goals established for a water treatment facility.

This handbook provides general information and insight into each of the above-mentioned steps that a potable water treatment facility should follow in developing a residuals management plan. For additional information on some of the processes described in this handbook, see the *Handbook of Practice: Water Treatment Plant Waste Management* (Cornwell et al., 1987) or *Slib, Schlamm, Sludge* (Cornwell and Koppers, 1990).

1.1 Overview of Differences in Water Treatment Processes

The form taken by water treatment plant (WTP) residuals can vary greatly, depending on the source of untreated water and the type of unit processes incorporated in a water treatment facility. Although many different forms of residuals are generated in the potable water treatment industry, this handbook primarily addresses those residuals produced by the following general categories of WTPs, which are identified by the names of their most common unit processes:

- *Coagulation/Filtration plant:* This traditional form of WTP is typically used to remove turbidity and pathogenic organisms. These facilities may also be used to remove color, taste, and odor-causing compounds from the water supply. A variation of the process may use aeration and oxidation processes for the removal of iron and manganese. Unit processes may include screening, chemical pretreatment, presedimentation, microstraining, aeration, oxidation, coagulation/flocculation, sedimentation/precipitation, filtration, disinfection, and dissolved air flotation treatment. Other nonchemical variations include direct filtration, diatomaceous earth filtration, and slow sand filtration.

- *Precipitative softening plant:* This variation of a coagulation/filtration facility uses additional processes to reduce water hardness. Unit processes may include screening, chemical pretreatment, presedimentation, microstraining, aeration, oxidation, coagulation/flocculation, lime softening, sedimentation/precipitation, filtration, and disinfection.

- *Membrane separation:* This process is typically used to remove turbidity, total dissolved solids, hardness, nitrates, and radionuclides from a water supply. More recent applications address removal of microbiological contaminants. Membrane separation generally involves the use of microfiltration, ultrafiltration, nanofiltration, reverse osmosis, or electrodialysis, often times in combination with pretreatment practices.

- *Ion exchange (IX):* These facilities are used to remove inorganic constituents, including hardness, nitrates, arsenic, and radionuclides from water. The process involves the use of IX reactors in combination with pretreatment processes.

- *Granular activated carbon (GAC) adsorption:* GAC is used in many processes for the removal of naturally occurring and synthetic organic matter from water.

The fundamental differences between the unit processes of these five plant types characterize the type of residuals generated at a given facility. WTPs constructed to meet the ever-widening scope of future regulations may require a combination of these unit processes—a coagulation/filtration treatment plant using

GAC as a filter medium, for example. Consequently, this handbook's discussion of different water treatment methods is purposely broad to include existing types of residuals as well as those from future potable water treatment facilities. The treatment of off-gas from an air stripping process and the residuals from diatomaceous earth filtration are some of the residual streams not extensively addressed in this handbook.

1.2 Overview of Residuals Categories

WTP residuals are typically derived from suspended solids in the source water, chemicals (e.g., coagulants) added in the treatment processes, and associated process control chemicals (e.g., lime). Some potable water treatment processes produce residuals that are relatively straightforward to process and dispose of. For example, the leaves, limbs, logs, plastic bottles, and other large floating debris separated from water during the screening process are simply disposed of at conventional solid waste landfills. Most other treatment processes generate more complex residual waste streams that require more sophisticated processing methods and final disposal methods to protect human health and the environment. The four major categories of residuals produced from water treatment processes are:

- Sludges (i.e., water that contains suspended solids from the source water and the reaction products of chemicals added in the treatment process). Presedimentation, coagulation, filter backwashing operations, lime softening, iron and manganese removal, and slow sand and diatomaceous earth filtration all generate sludge.
- Concentrate (brines) from IX regeneration and salt water conversion, membrane reject water and spent backwash, and activated alumina waste regenerant.
- IX resins, spent GAC, and spent filter media (including sand, coal, or diatomaceous earth from filtration plants).
- Air emissions (off-gases from air stripping, odor control units, ozone destruction on units).

The chemical characteristics and contaminant concentration levels in these residual waste streams often dictate the ultimate disposal options. Furthermore, it is reasonable to expect that as drinking water quality is increasingly regulated, higher removal efficiencies of more contaminants will be required. To achieve these higher efficiencies, WTPs will need to use more advanced treatment technologies. Of potential concern is the case where the residuals are characterized as either hazardous or radioactive waste. Depending on the raw water quality and treatment process removal efficiency, hazardous or radioactive characteristics could be exhibited in potentially any residual waste stream mentioned above. Classification of WTP residuals as hazardous or radioactive material, however, is unlikely at this time or in the near future; no water treatment residuals have yet been classified as either.

Some other typical residual waste streams associated with a WTP are considered beyond the scope of this handbook. These waste streams include storm drainage, sanitary sewage flows, laboratory and building floor drainage, and waste from spill containment areas, all of which generate residuals processing and disposal concerns for the drinking water industry. Each of these residual waste streams has specific regulatory requirements typically associated with point and nonpoint source pollution control as defined under the Clean Water Act (CWA). As with all residual treatment and disposal issues, consult with the appropriate state and local regulators to determine specific requirements.

Table 1-1 lists some typical residuals generated from drinking water treatment processes, possible contaminants normally found in each waste stream, available residual disposal methods, and a quick reference to the applicable federal regulations. In all cases, specific state and local regulations must also be considered.

1.3 Overview of Residual Solids Treatment Processes

In many instances, regulatory requirements or the need for cost effectiveness dictate that a residual receive further treatment to make it acceptable for disposal. The three classic treatment processes for residuals solids are thickening, dewatering, and drying. Application of a particular process depends on the solids concentration of the residual. This handbook describes applications of several types of solids concentrating processes.

Table 1-2 indicates which common residual treatment processes are usually applied to residuals with low, medium, or high solids concentrations, respectively. Within the water treatment industry, the definitions of low, medium, and high solids concentrations vary, depending on whether a sludge is produced by a coagulation/filtration plant or a precipitative softening plant. The processes presented in Table 1-2 are often configured in series to create combination systems that can provide a low level of operational complexity with a high degree of operational flexibility.

This handbook gives background information about each of the most common treatment processes. This information can then be used in the preliminary selection and sizing of appropriate unit processes during the development of a residuals management plan. This handbook also briefly discusses available treatment technologies for gaseous residuals. These technologies include stripping, odor control, gaseous chemical leak treatment, and ozonation.

Table 1-1. Treatment Processes and Waste Streams (Robinson and Witko, 1991)

Major Treatment Process Type	Typical Residual Waste Streams Generated	Typical Contaminant Categories	Typical Disposal Methods	Relevant Chapter in Handbook	Regulation Covering Disposal Method
Coagulation/ Filtration	Aluminum hydroxide, ferric hydroxide, or polyaluminum chloride, sludge with raw water suspended solids, polymer and natural organic matter (sedimentation basin residuals)	Metals, suspended solids, organics, biological, radionuclides, inorganics	Landfilling	Chapter 7	RCRA/CERCLA
			Disposal to sanitary sewer/WWTP	Chapter 6	State and local regulations
			Land application	Chapter 8	RCRA, DOT
			Surface discharge	Chapter 5	NPDES (CWA), state and local DOH
	Spent backwash filter-to-waste	Metals, organics, suspended solids, biological, radionuclides, inorganics	Recycle		State and local DOH
			Surface discharge (pumping, disinfection, dechlorination)	Chapter 5	NPDES (CWA), state and local regulations
			Disposal to sanitary sewer/WWTP	Chapter 6	State and local regulations
Precipitative softening	Calcium carbonate and magnesium hydroxide sludge with raw water suspended solids and natural organic matter	Metals, suspended solids, organics, unreacted lime, radionuclides	Landfilling	Chapter 7	RCRA/CERCLA, state and local regulations
			Disposal to sanitary sewer/WWTP	Chapter 6	State and local regulations
			Land application	Chapter 8	RCRA, state and local regulations, DOT
	Spent backwash filter-to-waste	Metals, organics, suspended solids, biological, radionuclides, inorganics	Recycle	Chapter 5	State and local DOH
			Surface discharge (pumping, disinfection, dechlorination)	Chapter 6	NPDES (CWA), state and local regulations
			Disposal to sanitary sewer/WWTP		State and local regulations
Membrane separation	Reject streams containing raw water suspended solids (microfiltration), raw water natural organics (nanofiltration), and brine (hyperfiltration, RO)	Metals, radionuclides, TDS, high molecular weight contaminants, nitrates	Surface discharge (pumping, etc.)	Chapter 9	RCRA, NPDES, state and local regulations
			Deep well injection (pumping)	Chapter 9	RCRA, NPDES, state and local regulations
			Discharge to sanitary sewer/WWTP	Chapter 9	State and local regulations
			Radioactive storage	Chapter 10	RCRA, DOT, DOE
Ion exchange	Brine stream	Metals, TDS, hardness, nitrates	Surface discharge	Chapter 9	RCRA, NPDES, state and local regulations
			Evaporation ponds	Chapter 9	RCRA, NPDES, state and local regulations
			Discharge to sanitary sewer/WWTP	Chapter 9	State and local regulations
Granular activated carbon[a]	Spent GAC requiring disposal and/or reactivation, spent backwash, and gas-phase emissions in reactivation systems	VOCs, SOCs (nonvolatile pesticides), radionuclides, heavy metals	Landfill	Chapter 7	RCRA, CERCLA, DOT
			Regeneration—on/off site	Chapter 14	State and local air quality regulations (CAA)
			Incineration		State and local air quality regulations (CAA)
			Radioactive storage Return spent GAC to supplier	Chapter 10	DOT, DOE

Table 1-1. Treatment Processes and Waste Streams (Robinson and Witko, 1991) (Continued)

Major Treatment Process Type	Typical Residual Waste Streams Generated	Typical Contaminant Categories	Typical Disposal Methods	Relevant Chapter in Handbook	Regulation Covering Disposal Method
Stripping process (mechanical or packed tower)	Gas phase emissions	VOCs, SOCs, radon	Discharge to atmosphere GAC adsorption of off-gas (contaminant type and concentration dependent)	Not addressed	State and local air quality regulations (CAA)
	Spent GAC if used for gas-phase control	VOCs, SOCs, radionuclides	GAC adsorption of off-gas (contaminant type and concentration dependent) Return spent GAC to supplier	Not addressed	State and local air quality regulations (CAA)

[a] The discussion on disposal methods for GAC residuals is generic in nature. For more specific information on disposal options for GAC, see McTigue and Cornwell (1994).

Key

CAA = Clean Air Act
CERCLA = Comprehensive Environmental Response, Compensation and Liability Act
DOE = Department of Energy
DOH = Department of Health
DOT = Department of Transportation
NPDES = National Pollutant Discharge Elimination System
RCRA = Resource Conservation and Recovery Act
RO = Reverse osmosis
SOC = Synthetic organic chemical
TDS = Total dissolved solids
VOC = Volatile organic compound

Table 1-2. Representative Solids Concentration Treatment Processes[a]

Process	Solids Concentration	Equalization	Gravity Settling	Dissolved Air Flotation	Lagoon	Mechanical	Open Air	Thermal Drying
Thickening	Low	X	X	X	X	X		
Dewatering	Medium				X	X	X	
Drying	High				X	X	X	X

[a] Chapter 4 of this handbook provides a more complete discussion of processing alternatives.

1.4 Selection of Residuals Management Plan Options

Before selecting the treatment and disposal actions necessary to develop a residuals management plan, the manager of a WTP may start with a large array of residuals processing and disposal options. These options are narrowed through consideration of specific residuals characteristics and associated regulatory requirements. A focus on the available disposal options further narrows the array to a finite set of residuals management alternatives.

Figure 1-1 illustrates that the primary target during the development of a residuals management plan should be practical disposal options and treatment processes that will take into account economic and noneconomic factors of concern to the community. The technical criteria used in the selection of the final management plan differ from user to user; economic, cultural, social, and environmental factors are also site-specific, and are typically included in any final selection. The technical information

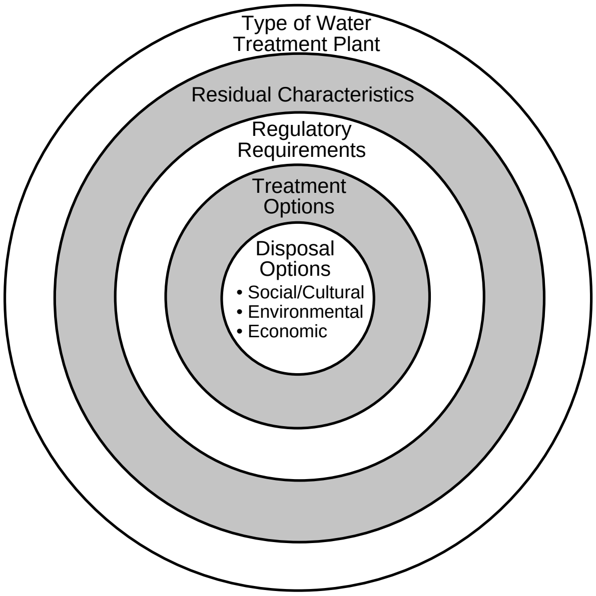


Figure 1-1. The primary target of a residuals management plan.

in this handbook can be used to screen out inappropriate residuals processing and disposal options. Cost curves for various treatment processes and a matrix of commonly encountered social, cultural, and environmental factors are included.

1.5 Handbook User's Guide

This handbook offers background information about the components of, and the development process for, a comprehensive residuals management plan. Developing a successful residuals management plan requires an understanding of the value of residuals characterization and the regulatory requirements, tailoring the treatment options to the requirements of the available disposal alternatives, and then developing rational evaluation criteria. This handbook is organized to logically guide the user through each of these steps.

Figure 1-2 illustrates a generic decision process for developing a residuals management plan. Each step in the process is keyed to a chapter in this handbook. Figure 1-2 can also be used as a quick reference to find specific information throughout the book.

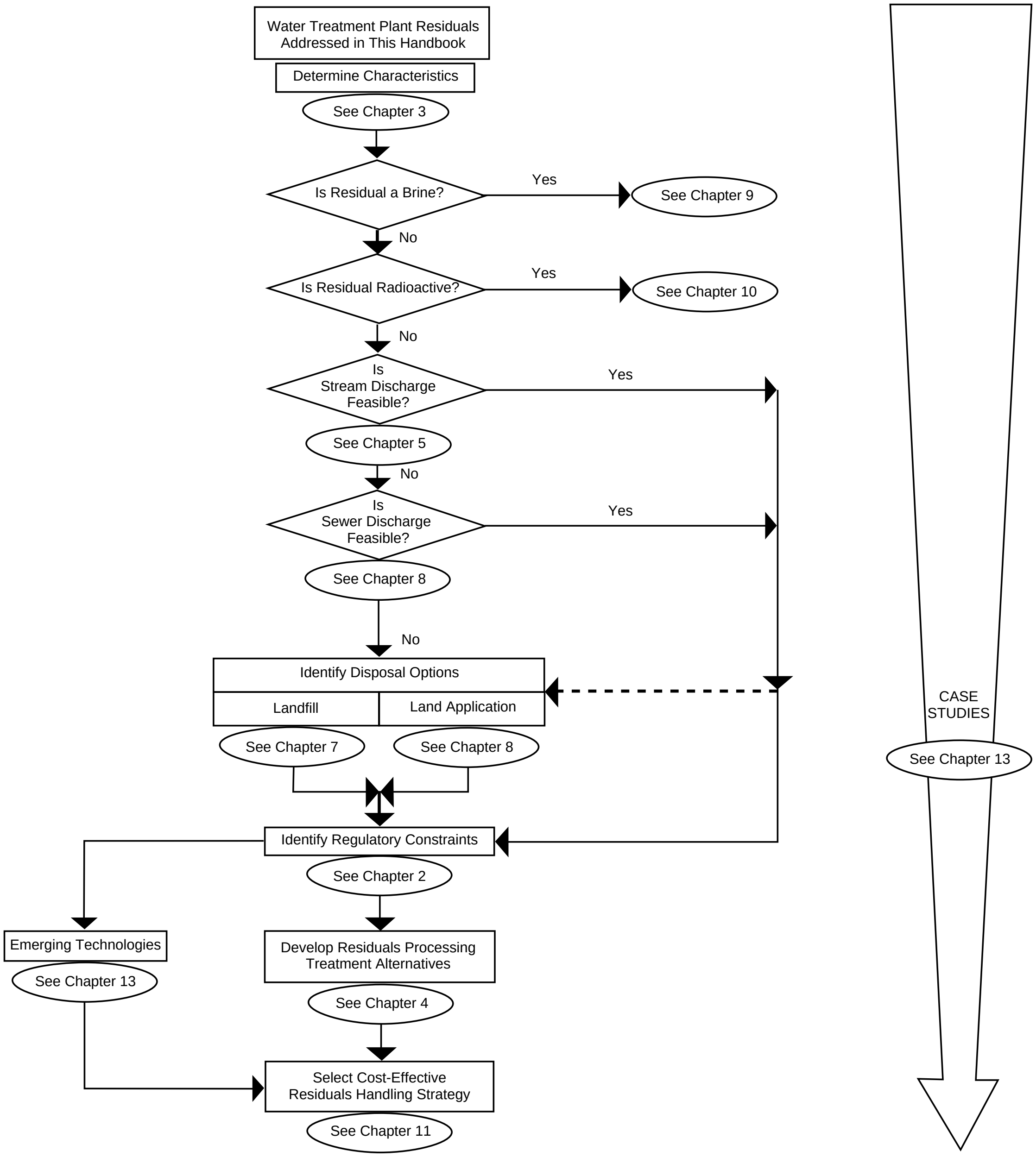


Figure 1-2. Handbook user's guide.

Chapter 2
Regulatory Issues Concerning Management of Water Treatment Plant Residuals

Water treatment utility managers who are beginning to explore alternative methods for disposal of water plant wastes may encounter difficulty identifying the regulations that affect the various management practices. The difficulty is compounded by the many different types of wastes produced by water treatment plants (WTPs).

This chapter provides an overview of the regulatory requirements governing the following disposal methods for WTP residuals: direct discharge, discharge to wastewater treatment plants, disposal in landfills, land application, underground injection, disposal of radioactive waste, and treatment of air emissions. Applicable federal regulations and typical state requirements are both discussed. In addition, the results of two surveys of state regulatory requirements conducted by the American Water Works Association (AWWA) Water Treatment Residuals Management Committee and the American Society of Civil Engineers (ASCE), are summarized in Appendix A. The surveys also identify commonly used residuals management practices in each state.

At the federal level, EPA has not established any regulations that are specifically directed at WTP residuals. Applicable regulations are those associated with the Clean Water Act (CWA); Criteria for Classification of Solid Waste Disposal Facilities and Practices (40 CFR Part 257); the Resource Conservation and Recovery Act (RCRA); the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA); and the Clean Air Act (CAA). The CWA limits direct discharges into a water course, while the other legislation governs other methods of use and/or disposal of wastes. Most states are responsible for establishing and administering regulations that will meet the requirements of these acts. The regulation of wastes, therefore, is primarily the responsibility of the states.

2.1 Discharge to Waters of the United States

The federal program to protect the quality of the nation's water bodies is authorized under the Federal Water Pollution Control Act (FWPCA) of 1972. Since its passage, the statute has been amended in 1977, 1978, 1980, 1981, and 1987, and renamed the Clean Water Act (CWA). The act and associated regulations attempt to ensure that water bodies maintain the appropriate quality for their intended uses, such as swimming, fishing, navigation, agriculture, and public water supplies.

The National Pollutant Discharge Elimination System (NPDES) applies to WTPs that discharge wastes directly to a receiving water. Under Section 402 of the CWA, any direct discharge to waters of the United States must have an NPDES permit. The permit specifies the permissible concentration or level of contaminants in a facility's effluent. EPA authorizes states to act as the primary agent for the NPDES program, provided that the state program meets all EPA requirements. State regulations and guidelines controlling the discharge of residuals, however, vary throughout the United States. Certain states permit direct discharge of residuals with or without pretreatment requirements. In other states, direct discharge has been restricted through limitations on suspended solids and pH (W.E. Gates and Associates, 1981; Cornwell and Koppers, 1990). Generally, direct discharge into streams has been permitted for clarified water such as settled backwash water or overflow from solids separation processes.

For states not granted primacy, EPA regional offices issue NPDES permits. An NPDES permit is issued to a discharger based on technology-based effluent limitations, water quality standards, or both.

2.1.1 Technology-Based Effluent Limitations

Under Sections 301 and 304 of the CWA, EPA is required to establish national effluent limitations for the major categories of industrial dischargers. These limitations reflect the capabilities of the best available technology that is economically feasible for use in treating industrial discharges to surface waters. Federal effluent limitations have not yet been issued for WTP residuals; therefore, the delegated states or the regional EPA offices are responsible for establishing the limits for WTP discharges. Federal guidelines for controlling WTP discharges were drafted but never fully implemented. The draft guidance document divides WTPs into three categories (W.E. Gates and Associates, 1981; Cornwell and Koppers, 1990):

- Plants that use coagulation, oxidation for iron and manganese removal, or direct filtration.
- Plants that use chemical softening procedures.
- Plants that use a combination of the procedures in the above categories.

The draft document defines the best practical control technology for each category and establishes discharge limits on pH and total suspended solids (TSS). Based on plant capacity, WTP discharge limits varied in the document from 0.6 to 1.3 g TSS per cubic meter of water treated, which corresponds to a TSS of approximately 30 to 60 mg/L in the discharge, assuming a 2 percent waste discharge of the main stream. Because secondary treatment or the equivalent is required for treating drinking water, 85 percent removal and/or a TSS level of 30 mg/L are typically required for WTP discharges.

2.1.2 Water Quality Standards

Under Section 303 of the CWA, each state is required to establish ambient water quality standards for its water bodies. These standards define the type of use and the maximum permissible concentrations of pollutants for specific types of water bodies. The states use water quality criteria documents published by EPA, as well as other advisory information, as guidance in setting maximum pollutant limits. EPA reviews and approves the state standards.

According to EPA, certain in-stream water quality standards at the edge of the mixing zone must be met to allow direct discharge of WTP residuals. A controlled release of water clarifier residuals and filter backwash that meets water quality standards may be considered technology-based controls in appropriate circumstances (Cornwell and Lee, 1993).

Some states have established maximum allowable concentrations for pollutants in discharges to water bodies. These limits generally apply if they are more stringent than the allowable discharge that will meet the in-stream water quality criteria. For example, Illinois does not allow a discharge of greater than 15 mg/L fluoride, and barium discharge must be lower than 2 mg/L, even if the 1 mg/L in-stream standard could be met through dilution in the mixing zone (Cornwell et al., 1987). An overflow from a solids separation process such as a lagoon, thickener, or, sometimes, backwash water, requires a discharge permit.

2.1.3 Special Concerns Regarding Aluminum

EPA has developed an ambient water quality standard for aluminum, requiring that the in-stream soluble aluminum level not exceed 87 µg/L on a 4-day average. The 1-hour average must not exceed 750 µg/L. This standard may be of concern for WTPs that use aluminum sulfate or other aluminum coagulants and rely on an NPDES-permitted discharge to dispose of settled backwash wastewater, supernatant from dewatering process units, or coagulation/flocculation residuals. The standard is focused on dissolved aluminum; most aluminum in WTP discharges is in solid form.

Because aluminum is abundant in some geologic formations, it is not unusual to find it in concentrations higher than 1 mg/L in receiving streams for NPDES discharges. Consequently, some states may mandate complete elimination of any NPDES discharges that contribute to the aluminum load of the receiving stream.

2.2 Discharge to Wastewater Treatment Plants

Three general categories of regulations affect the decision of whether to discharge WTP residuals to a sanitary sewer or directly to a wastewater treatment plant (WWTP): 1) federal and state hazardous waste regulations; 2) federal and state radioactive waste regulations; and 3) local receiving wastewater utility regulations, driven generally by the need for a utility to comply with the provisions of the CWA.

The first two categories of regulation usually are not a problem. WTP residuals are rarely, if ever, classified as hazardous waste. While water treatment facilities often generate a radioactive component in residuals, it is at a very low level. The radioactive component results from removing normally occurring radioactive material (NORM) from the process water by the addition of a coagulant. The resulting low-level radioactive WTP residuals are generally acceptable for disposal to a wastewater utility. The third category of regulations, local receiving wastewater utility discharge permit regulations, incorporates any provisions concerning hazardous or radioactive wastes that may be germane. Provisions are sometimes included that reduce the risk of operational problems at the receiving WWTP or of violation to its discharge permit. In small utility situations, often no formal local receiving wastewater utility regulations exist, and compliance with any regulatory guidance is of little concern.

All WWTPs must comply with EPA's general pretreatment requirements, under which WTP residuals are clearly identified as industrial wastes. In addition, EPA requires all WWTPs with wastewater inflow greater than 5 million gallons per day (mgd) to establish formal pretreatment enforcement programs. Under these programs, the WWTPs regulate discharges into municipal sewers by industries, including water treatment facilities, and other entities. The WWTPs must, at a minimum, enforce national pretreatment standards. They also may implement additional controls or limits based on local conditions for any of the following reasons:

- A WWTP's NPDES permit may require removal of pollutants that the plant itself cannot remove.
- Pollutants in residuals discharged into the sewer system may adversely affect the performance of the collection system or removal systems in the WWTP.
- Pollutants in residuals discharged into the sewer system may contaminate WWTP wastes, making dewatering and disposal more difficult and expensive.

EPA authorizes the states to review and approve individual WWTP pretreatment programs. In addition to the individual plant's pretreatment requirements, state agencies may provide specific guidelines.

2.3 Landfilling

Under RCRA Subtitle D regulations (40 CFR Parts 257 and 258), criteria have been established for the design and operation of nonhazardous, solid waste landfills. Landfills that receive only drinking water treatment residuals are subject to the requirements of 40 CFR Part 257. These requirements are also applicable to landfills that accept solid waste other than household waste, such as industrial waste. These criteria apply to all nonhazardous, nonhousehold solid waste disposal facilities and practices. They address seven areas of environmental concern pertaining to landfill design and operation (see Table 2-1). It should be noted that these criteria are performance based and do not include any specific design criteria.

2.3.1 Municipal Nonhazardous Solid Wastes

Municipal solid waste landfills (MSWLFs) are subject to the criteria of 40 CFR Part 258. Unlike Part 257, Part 258 includes specific design criteria in addition to performance-based criteria. If a utility disposes of its WTP residuals in a monofill, then, under federal regulations, Part 258 does not apply; Part 257 (discussed above) does instead. If, however, the WTP residuals are co-disposed of with municipal solid waste, the requirements established for MSWLFs apply. These landfill criteria (40 CFR Part 258) address six major areas as listed in Table 7-2.

Six location restrictions apply to MSWLFs. Landfills cannot be located in floodplains, wetlands, seismic impact zones, and unstable areas. Additionally, specific setbacks are required for landfills near airports and fault areas.

The minimum design criteria for new landfills are intended to give owners/operators two basic design options—a composite liner design, or a site-specific design

Table 2-1. Required Management Practices for Nonhazardous Industrial Waste Only and Co-Disposal Landfills, and Land Application Sites (40 CFR Part 257)

Environmental Concern	Management Practice
Floodplains	Facilities in floodplains shall not restrict the flow of the 100-year flood, reduce the temporary water storage capacity of the floodplain, or result in washout of solid waste, so as to pose a hazard to human life, wildlife, or land or water resources.
Endangered species	Facilities shall not cause or contribute to the taking of any endangered or threatened species of plants, fish, or wildlife, and shall not result in the destruction or adverse modification of the critical habitat of endangered or threatened species. Questions about the potential for adversely affecting endangered or threatened species at a particular site should be directed to the nearest Regional office of the U.S. Fish and Wildlife Service. (Note: Notices of draft NPDES permits also are routinely sent to the U.S. Fish and Wildlife Services. See 40 CFR 124.10(c).)
Surface water	Facilities shall not cause a discharge of pollutants into waters of the United States in violation of Section 402 of the Clean Water Act, shall not cause a discharge of dredged or fill material in violation of Section 404 of the Act, and shall not cause nonpoint source pollution that violates an approved Section 208 water quality management plan.
Ground water	Facilities shall not contaminate an underground drinking water source beyond the solid waster boundary, or beyond an alternative boundary. Consult the regulation for procedures necessary to set alternative boundaries (see Appendix D). To determine contamination, Appendix I of 40 CFR Part 257 provides a list of contaminant concentrations. Release of a contaminant to ground water which would cause the concentrations of that substance to exceed the level listed in Appendix I constitutes contamination.
Disease	Disease vectors shall be minimized through the periodic application of cover material or other techniques as appropriate to protect public health.
Air	Facilities shall not engage in open burning of residential, commercial, institutional, or industrial sold waste. Infrequent burning of agricultural wastes in the field, silvicultural wastes for forest management purposes, land-clearing debris, diseased trees, debris from emergency cleanup operations and ordinance is allowed.
Safety	1. Explosive gases generated by the facility shall not exceed 25 percent of the lower explosive limit for the gases in facility structures and 100 percent of the lower explosive limit at the property line. 2. Facilities shall not pose a hazard to the safety of persons or property from fires. 3. Facilities within 10,000 feet of any airport runway used by turbojet aircraft, or within 5,000 feet of any airport runway used by piston-type aircraft shall not pose a bird hazard to aircraft. 4. Facilities shall not allow uncontrolled public access so as to expose the public to potential health and safety hazards at the disposal site.

that meets the performance standard of RCRA Subtitle D and has been approved by the permit writer.

Subtitle D establishes specific criteria for the following landfill operational factors: regulated hazardous waste detection program, cover material, disease vector control, explosive gases control, air criteria, access requirements, storm water run-on and runoff management, surface water, liquids restrictions, recordkeeping.

The most extensive portion of Subtitle D pertains to ground-water monitoring and corrective action. The rule mandates that all landfills must have a monitoring system that yields sufficient ground-water samples from around the site. Both upgradient and downgradient wells and sampling are required. Tests to detect 15 heavy metals and 47 volatile organic compounds must be conducted, but approved states can modify the list to reflect site-specific conditions.

The financial assurance requirements of RCRA are designed to guarantee that funds will be available for closure and postclosure care of landfills and, if needed, corrective action. All landfills are required to install a final cover system with at least two components, an infiltration layer and an erosion layer. The minimum final cover requirements mandate an infiltration layer at least 18 inches thick with a very low permeability limit, and an erosion layer at least 6 inches thick that can sustain native plant growth. A written closure plan must be submitted to the state solid waste director. Postclosure care must be provided for 30 years; it includes maintaining the final cover, the leachate collection and disposal system, the ground-water monitoring program, and the landfill gas monitoring system.

WTP residuals that are co-disposed of with municipal solid waste in a sanitary landfill do not have to meet numerical pollutant limits. The residuals, however, cannot be hazardous and must not contain free liquids. Mechanical and natural dewatering of residuals is often adequate for removing the free liquid from the water treatment solids. Because EPA requires landfill operators to make random inspections of incoming waste or take other steps to ensure that incoming loads do not contain regulated hazardous wastes, landfill operators may require residuals disposers to prove their solids are not hazardous. Hazardous wastes (as defined in 40 CFR Part 261) are waste materials that exhibit ignitability, corrosivity, reactivity, or toxicity (see Section 2.7).

EPA now prohibits the disposal of noncontainerized or bulk liquid wastes in landfills. Subtitle D requires that the landfill owner or operator determine if the wastes (including municipal water treatment solids) are liquid wastes according to the Paint Filter Liquids Method 9095. This simple test is performed by placing a representative sample of the solids in a mesh Number 60 paint filter (available at paint stores) and allowing it to drain for 5 minutes. The solids are considered a liquid waste if any liquid passes through the filter during the 5-minute period.

2.3.2 Summary

In summary, neither 40 CFR Part 257 nor Part 258 requires that WTP plant residuals be stabilized or have a certain percent solids concentration. 40 CFR Part 258 does require that the residuals pass a paint filter test prior to co-disposal with solid wastes in a landfill. Both regulations require that the filled areas be periodically covered to protect public health. Both regulations require attention to protection of ground and surface water sources and control of gas migration.

Some states may impose further restrictions on bulk or special wastes such as those produced during water treatment. Besides the paint filter test, states also require a minimum total solids content. Nebraska, one of the most restrictive states, requires a minimum total solids content of 70 percent. Some states also have a maximum allowable ratio of residuals-to-municipal solid waste.

2.4 Land Application

2.4.1 Federal Regulations

Land application of WTP residuals is typically regulated at the state level. The recently developed federal standards for the use or disposal of sewage residuals (40 CFR Part 503) specifically exclude WTP residuals. Criteria for classification of solid waste disposal facilities and practices (40 CFR Part 257), however, could affect land application of WTP residuals (see Table 2-1). This rule regulates the disposal of nonhazardous wastes, which include residuals generated from a WTP. The objective of this rule is to prevent construction or operation of a residuals processing or disposal facility from adversely affecting surface water, ground water, endangered or threatened wildlife, or public health. Criteria were established under 40 CFR Part 257 regulating application of residuals containing cadmium and polychlorinated biphenyls (PCBs). As discussed in Chapter 3 on residuals characterization, concentrations of cadmium and PCBs in WTP residuals are usually below detection levels, making these criteria inapplicable.

Specific requirements for controlling disease vectors are also included under Part 257 to prevent adverse public health impacts resulting from application of solid wastes/residuals. Criteria are also established to protect ground-water quality beyond the application site boundary. Land application activities must not cause specific organic and inorganic chemicals to exceed maximum contaminant levels in the area. These criteria generally should not affect the ability to land-apply WTP residuals.

2.4.2 *State Regulations*

Most state agencies are waiting for EPA to establish land application criteria for WTP residuals. Despite the differences in their characteristics, many states apply WWTP biosolids criteria, such as metals loading limits, to the land application of WTP residuals.

Some states have taken it on themselves to establish specific criteria for land application of WTP residuals. For example, the Colorado Department of Health regulates beneficial land application of WTP residuals under domestic sewage sludge regulations (Colorado, 1986). The department has established specific regulations pertaining to the beneficial use of WTP residuals on land. These regulations require development of an approved beneficial use plan that:

- Identifies where the material will be used.
- Contains approval from the land owner and the appropriate county health department.
- Contains residuals analyses.
- Identifies the types of crops to be grown and the application rates.

Parameters to be analyzed include aluminum, arsenic, cadmium, pH, total solids, and nutrients. If WTP residuals are used with biosolids, then the practice must comply with the biosolids regulations. Application to land where root crops or low-growing crops are to be grown is prohibited if the crops are intended for human consumption.

The Colorado Department of Health does not allow beneficial use of WTP residuals with radioactivity levels exceeding 40 picocuries total alpha activity per gram (pCi/g) of dry residuals.

The Missouri Department of Natural Resources has also established guidelines for land application of alum residuals. These guidelines recommend that soil pH be maintained near 7.0 for alum residuals application and that total aluminum loading to the soil not exceed 4,000 lb/acre without site-specific investigations. No restrictions apply to land application of lime softening residuals (Missouri, 1985).

2.5 Underground Injection

The federal government has promulgated regulations for injecting wastes into the ground. Most states have fully adopted these regulations and have been granted primacy for enforcing the program. These regulations alone, however, will probably not prevent underground injection of WTP residuals. Some states, such as Florida, have chosen to promulgate and/or issue stricter regulations and/or policies, making it very difficult to dispose of WTP residuals such as brines via underground injection.

2.5.1 *Underground Injection Control Program*

Underground injection may be a disposal option for concentrates and brines from drinking water treatment processes. This option is subject to regulatory approval under the underground injection control (UIC) program, authorized by the Safe Drinking Water Act (SDWA). The UIC program regulates the subsurface placement of fluid in wells or dug-holes with a depth greater than their width. The program covers the disposal of hazardous waste and various other substances in wells.

After the 1980 amendments to the SDWA, EPA developed a mechanism to grant individual states primary enforcement responsibility for underground injection. The UIC regulations are enforced through a permitting system. Each state is required to develop a UIC permit program that enforces EPA standards, prevents underground injection unless authorized by a permit or rule, authorizes underground injection only where the process will not endanger drinking water sources, and maintains thorough records and inspection reports. Currently, all 50 states have established UIC programs, some of which are managed by EPA regional offices.

2.5.1.1 Classification of Underground Injection Wells

To prevent contamination of drinking water sources, EPA established regulatory controls and a classification system based on the type of waste injected and the location of the injection well (40 CFR 146.5):

- Class I includes wells used to inject hazardous waste, or industrial and municipal waste beneath the lowermost formation containing an underground source of drinking water within 1/4 mile of the well bore.
- Class II includes wells used to inject fluids generated from oil and natural gas production and refining.
- Class III includes wells that are injected with liquid (such as water) for extraction of minerals including sulfur, uranium, and other metals in situ.
- Class IV includes wells that are used to inject hazardous or radioactive wastes into or above a formation containing an underground source of drinking water within 1/4 mile of the well. This class of wells also includes wells used to dispose of hazardous waste into or above a formation containing an aquifer that has been exempted pursuant to 40 CFR 146.04.
- Class V includes the injection wells that are not covered in Classes I through IV.

Wells used for underground injection of WTP residuals fall into Classes I, IV, and V.

2.5.1.2 Permit Application

Underground injection wells are authorized by rule under RCRA and permitted at the state level under the SDWA.

Authorization by Rule

Authorization of underground injection by rule gives owners and operators an opportunity to operate underground injection facilities before their permit applications are approved. To be authorized by rule, owners or operators must comply with the applicable requirements. For Class I and III wells, the owners or operators must meet the requirements listed in 40 CFR 144.28 and individual state requirements. The authorization is void if a permit application has not been filed in a timely manner as specified in 40 CFR 144.31(c)(1); otherwise, it can be extended until the permit is issued or revoked by a UIC program administrator. For Class IV wells, the operation period can be as long as 6 months on the condition that requirements specified in 40 CFR 144.13, 144.14(c) are met. Currently, the operation of Class V wells is not authorized by rule.

Authorization by Permit

Except for owners or operators who are authorized by rule to run underground injection facilities, all other facilities must be authorized by permits. Facilities authorized by rule are also required to apply for a permit to continue operating their facilities on a long-term basis.

All applicants for underground injection permits must complete the application forms provided by a state UIC program, and submit necessary supporting documents. This paperwork should include the following information:

- Facility name, address, and ownership.
- Activities that are conducted in the facility and which require a permit under RCRA, CWA, or CAA.
- A list of principal products or services provided by the facility.
- Lists of the relevant permits and construction approvals issued to the facility.
- Geographic and topographic characteristics of the facility.

2.5.1.3 Underground Injection Control Criteria and Standards

The UIC program administrator reviews the permit applications based on UIC criteria and standards (40 CFR 146). Different criteria and standards apply to different classes of injection wells. The major criteria and standards are:

- *Construction requirements:* Configuration of the injection wells (hole size, depth of injection zone), injection pressure, type of cement, and type of injected fluids must be specified and meet the requirements. Both new and existing wells must be cased and cemented to protect sources of drinking water.
- *Operating requirements:* Certain operating conditions must be met. For example, the injection pressure must not exceed a maximum, and injection between the outermost well casing and the well bore must be avoided to protect underground sources of drinking water.
- *Monitoring requirements:* The nature of the injected fluids, injection pressure, flow rate, cumulative volume, and mechanical integrity must be monitored. Monitoring should take place at regular intervals and is based on class of well and operation type.
- *Reporting requirements:* Quarterly or yearly reports should include the operating conditions for the period, the results of the monitoring, and the results of any other required tests.

2.5.1.4 Actions Against Violations

Under the SDWA, no operator or owner of an underground injection facility is allowed to construct, operate, maintain, convert, plug, or abandon that facility, or conduct any other injection activity in a way that might contaminate a ground-water source of drinking water.

For Class I, II, and III wells, if ground-water quality monitoring shows the intrusion of contaminants to a ground-water source, then corrective action may be required. Corrective actions may be taken in the areas of operation, monitoring, or reporting, or might include closing the well, if required. If the operation of the well is authorized by a permit, modification of the permit with additional requirements might occur (40 CFR 144.39). If the permit is violated, appropriate enforcement action might be taken (40 CFR 144.55).

Under the SDWA, new construction of most Class IV wells is strictly prohibited. Increasing the amount or changing the type of waste injected in these wells is also forbidden.

If a Class V well causes a violation of primary drinking water regulations under 40 CFR Part 142, the operator of the well must take any action (including closing the injection well) to prevent contaminating drinking water sources.

2.5.2 Underground Injection Requirements Under RCRA

Section 3004(f) of RCRA requires EPA to determine whether underground injection of hazardous wastes will endanger human health and the environment. In response, EPA has banned the underground injection of hazardous wastes that do not meet the applicable treatment standards of the land disposal restrictions (see 40 CFR Part 148). The 1986 Hazardous and Solid Waste Amendments (HSWA) to RCRA enhanced the restriction of underground injection of hazardous waste. These amendments prohibit the disposal of hazardous waste through underground injection into or above a formation within 1/4 mile of an underground source of drinking water.

2.6 Disposal of Radioactive Waste

When radium isotopes and other radioactive materials are removed from a drinking water supply, they are usually concentrated in residuals that must be disposed of in ways that are cost-effective, practical, and protective of human health and the environment. The U.S. Nuclear Regulatory Commission (NRC) has the authority to regulate the handling and disposal of all licensed synthetic radioactive material. Certain materials, such as water and WWTP wastes containing naturally occurring radioactive material (NORM), are not licensed or regulated by NRC because they are not included under its authority as source, special nuclear, or byproduct material. Regulation of NORM is left to individual states (Hahn, 1989). Limitations on radioactive residuals are established case-by-case, using best professional judgment (Koorse, 1993a). Recognizing that water treatment could concentrate radionuclides, EPA has issued guidelines for managing radionuclide waste from water treatment containing up to 2,000 pCi/g (dry weight) of NORM (U.S. EPA, 1994b). The guidelines addressing disposal of residuals, brines, and solid wastes containing radium or uranium are discussed in detail in Chapter 10.

Several states regulate disposal of WTP residuals containing radionuclides. For example, in certain states, water quality standards include specific criteria for radionuclides. Some states also limit the discharge of wastes containing naturally occurring radionuclides into sanitary sewers. Illinois limits the use of residuals for soil conditioning on agricultural lands. Illinois and Wisconsin have developed criteria for landfilling residuals containing radium. In Wisconsin, the discharge of WTP residuals to a sanitary sewer collection and treatment system that is otherwise acceptable is governed by radium discharge limits as follows:

$$\frac{C_{Ra-226}}{400} + \frac{C_{Ra-228}}{800} \leq 1$$

where

$C_{\text{Ra-226}}$ = the concentration in picocuries per liter (pCi/L) of soluble Ra-226 in the wastewater

$C_{\text{Ra-228}}$ = the concentration of soluble Ra-228 in pCi/L in the wastewater

The average monthly combined radium concentrations discharge, measured in pCi/L, must not exceed the limits of the preceding equation, and the total amount of radiation released to the sanitary sewer system in any 1-year period cannot exceed 1.0 Ci (Hahn, 1989).

In Colorado, if the total alpha activity of the WTP residuals exceeds 40 pCi/g (dry weight), the WTP is required to contact the Colorado Department of Health, Radiation Control Division, for further disposal guidance (Colorado, 1990). In addition, North Dakota has regulations requiring the water plant to be licensed as a generator of radioactive material (Cornwell et al., 1987). Other states have adopted similar regulations that may affect whether WTP residuals containing radioactivity may be discharged to sanitary sewers.

2.7 Hazardous Waste

WTP residuals are generally not considered hazardous wastes. Even spent granular activated carbon (GAC) residuals are usually not hazardous wastes (McTigue and Cornwell, 1994; Dixon, 1993). Some disposal or use measures for WTP residuals, however, require demonstration that the material is not hazardous according to governing hazardous waste regulations.

Subtitle C, Section 3001 of RCRA addresses treatment, storage, and disposal of hazardous waste. These regulations are designed to ensure proper management of hazardous waste from “cradle to grave”—i.e., from the moment the waste is generated until it is ultimately disposed of. This approach has three elements:

- A tracking system requiring that a uniform manifest document accompany any transported hazardous waste from the point of generation to the point of final disposal.
- An identification and permitting system that enables EPA and the states to ensure safe operation of all facilities involved in treatment, storage, and disposal of hazardous waste.
- A system of restrictions and controls on the placement of hazardous waste on or into the land.

EPA employs two separate mechanisms for identifying hazardous wastes (as defined in 40 CFR Part 261). Wastes may be defined as hazardous based on their characteristics (ignitability, corrosivity, reactivity, or toxicity), or they may be specifically designated as hazardous in lists published by the agency. Since WTP residuals are not specifically designated as hazardous wastes, they can be classified as hazardous only if they exhibit any of the four hazardous characteristics.

A waste is classified as ignitable if it is capable of causing a fire that burns vigorously and persistently under standard pressure and temperature, causing a hazard. An aqueous waste stream is corrosive if it has a pH lower than 2.0 or higher than 12.5. A waste is reactive if it has any of these characteristics:

- Is unstable.
- Reacts violently or forms potentially explosive mixtures with water.
- Generates toxic gases or fumes when mixed with water.
- Contains cyanides or sulfides that can generate toxic gases or fumes.
- Can detonate if exposed to heat.
- Is already defined as an explosive material.

Although domestic WTP residuals are rarely ignitable, corrosive, or reactive, the greatest concern for water treatment wastes is toxicity, as determined by the Toxicity Characteristic Leaching Procedure (TCLP) (U.S. EPA, 1992c). The TCLP applies to certain metals, herbicides, pesticides, and volatile organic compounds (VOCs). A water treatment residual failing the TCLP test can be classified as a hazardous material. Many landfills require the disposer to document TCLP analysis results as proof that the material is not toxic before solids disposal is allowed.

The TCLP uses a vacuum-sealed extraction vessel to capture VOCs in the sample. Table 2-2 lists the contaminants analyzed in the TCLP, along with maximum allowable pollutant concentrations at which the waste is considered nonhazardous. WTP solids generally do not fail the TCLP and are not classified as hazardous materials. If WTP wastes are classified as hazardous wastes, they are subject to the handling, treatment, and disposal requirements specified in RCRA Subtitle C.

CERCLA also affects landfilling of WTP wastes. Through CERCLA, cleanup costs at hazardous waste sites can be assessed against the user of the site on a volume-use basis; the waste itself need not have directly caused the problems. Co-disposal of water treatment residuals, then, with potentially hazardous wastes significantly increases the potential for a utility to be held responsible for landfill remediation costs.

2.8 Air Emissions

The federal government has promulgated regulations to control air emissions from industrial sources. Most states have fully adopted these regulations and have been granted primacy for enforcing them. Some states such as New Jersey, Michigan, and California, have promulgated stricter regulations and/or policies governing air emissions. These states may require treatment of air stripper emissions using activated carbon treatment or combustion.

2.8.1 Federal Regulations

Federal regulations do not specifically regulate air emissions from drinking water treatment plants. Instead, they

Table 2-2. TCLP Constituents and Regulatory Limits (40 CFR Part 261.24)

Constituents	Reg. Level (mg/L)
Arsenic	5.0
Barium	100.0
Benzene	0.5
Cadmium	1.0
Carbon tetrachloride	0.5
Chlordane	0.03
Chlorobenzene	100.0
Chloroform	6.0
Chromium	5.0
o-Cresol	200.0
m-Cresol	200.0
p-Cresol	200.0
Cresol[a]	200.0
2,4-D	10.0
1,4-Dichlorobenzene	7.5
1,2-Dichloroethylene	0.5
1,1-Dichloroethylene	0.7
2,4-Dinitrotoluene	0.13
Endrin	0.02
Heptachlor (and its hydroxide)	0.008
Hexachlorobenzene	0.13
Hexachloro-1,3-butadiene	0.5
Hexachloroethane	3.0
Lead	5.0
Lindane	0.4
Mercury	0.2
Methoxychlor	10.0
Methyl ethyl ketone	200.0
Nitrobenzene	2.0
Pentachlorophenol	100.0
Pyridine	5.0
Selenium	1.0
Silver	5.0
Tetrachloroethylene	0.7
Toxaphene	0.5
Trichloroethylene	0.5
2,4,5-Trichlorophenol	400.0
2,4,6-Trichlorophenol	2.0
2,4,5-TP (Silvex)	1.0
Vinyl chloride	0.2

[a] If o-, m-, and p-cresol concentrations cannot be differentiated, the total cresol concentration is used.

set national standards for the quality of ambient air and the states must ensure that these standards are met. States generally regulate sources of air contaminants on a case-by-case basis. WTPs that use aeration technologies, for example, might be subject to state air quality regulations because of concerns about human exposure to air emissions.

The CAA, initially passed in 1970 and amended in 1977, 1988, and 1990, gives EPA authority to set national standards for the quality of ambient air and to regulate sources of pollution that may affect air quality. The cornerstone of the CAA is a set of National Ambient Air Quality Standards (NAAQS) for six pollutants: ozone, total suspended particulates, sulfur oxides, lead, nitrogen dioxide, and carbon monoxide. The NAAQS establish the maximum allowable concentration for each pollutant in all areas of the United States (see Table 2-3).

Table 2-3. National Ambient Air Quality Standards (40 CFR Part 50)[a]

Carbon monoxide	Primary: 35.0 parts per million (ppm) averaged over 1 hour and 9.0 ppm averaged over 8 hours; neither level to be exceeded more than once per year.
	Secondary: Same as primary.
Fine particulate matter[b]	Primary: 150 µg/m^3 averaged over 24 hours, with no more than one exceedance per year averaged over a 3-year period; also, 50 µg/m^3 expected annual arithmetic mean.
	Secondary: Same as primary.
Lead	Primary: 1.5 µg/m^3 arithmetic average over a quarter of a calendar year.
	Secondary: Same as primary.
Nitrogen dioxide	Primary: 100 µg/m^3 (or 0.053 ppm) as annual arithmetic mean concentration.
	Secondary: Same as primary.
Ozone	Primary: 235 µg/m^3 (0.12 ppm) averaged over 1 hour, not to be exceeded more than once per year. (The standard is satisfied if the number of calendar days on which the standard is exceeded is 1 or less. Multiple violations in a day count as one violation).
	Secondary: Same as primary.
Sulfur oxides	Primary: 365 µg/m^3 (0.14 ppm) averaged over a 24-hour period, not to be exceeded on average more than once per year over a 3-year period; 80 µg/m^3 (0.03 ppm) annual arithmetic mean.
	Secondary: 1,300 µg/m^3 average over a 3-hour period, not to be exceeded more than once per year.

[a] National Primary and Secondary Ambient Air Quality Standards, July 1, 1987.

[b] Standard applies only to particulate matter that is $\leq 10\,\mu m$ in diameter.

The federal NAAQS do not specify the means by which the emission levels are to be met. States must ensure that NAAQS are achieved by enforcing all national emission standards and implementing any additional controls necessary for their particular region. States must publish, and EPA must approve, State Implementation Plans (SIPs) that describe the measures to be taken to ensure that NAAQS are achieved and maintained.

Parts of the U.S. that fail to meet one or more of the NAAQS are designated as nonattainment areas. Figure 2-1, for example, shows the location of nonattainment areas for ozone. NAAQS have been established for ozone, and VOCs are regulated by the states as ozone precursors on a source-by-source basis.

In addition to setting NAAQS and limiting emissions to achieve these standards, EPA has implemented other programs for controlling airborne contaminants. Under Section 111 of the CAA, EPA has the authority to establish New Source Performance Standards (NSPS) for restricting emissions from new industrial facilities or facilities undergoing major modifications. Under the CAA, EPA must set the NSPS control levels that reflect the "degree of emission reduction achievable" through use of the best available control technology (BACT) that has been "adequately demonstrated."

For toxic air pollutants not covered by the NAAQS or NSPS, EPA promulgates National Emission Standards for Hazardous Air Pollutants (NESHAPs). NESHAPs address pollutants with more limited exposure but more extreme health effects than pollutants controlled under the other standards. Most NESHAPs are defined in terms of the rate of emission from a source. Thus far, NESHAPs have been promulgated for eight compounds: arsenic, asbestos, benzene, beryllium, mercury, polycyclic organic matter (POM), radionuclides, and vinyl chloride. For example, NESHAPs governing radionuclide emissions require that the maximum radiation dose to an individual be no more than 10 mrem/yr. Facilities must monitor their emissions at doses of 1 percent of the limit or 0.1 mrem/yr.

2.8.2 State Regulations

As indicated earlier, NAAQS are established by EPA for specific pollutants and the states then set standards to attain and maintain them. Each state's approach and timetable for ensuring compliance with NAAQS are summarized in its SIP. The SIP can incorporate many regulatory measures that go beyond federal emissions limitations to achieve compliance with the NAAQS. Each SIP is reviewed and approved by EPA.

Individual states differ considerably in their approaches and timetables for regulating air emissions. In general, states do not have specific requirements for WTPs; instead, they evaluate emissions of air contaminants on

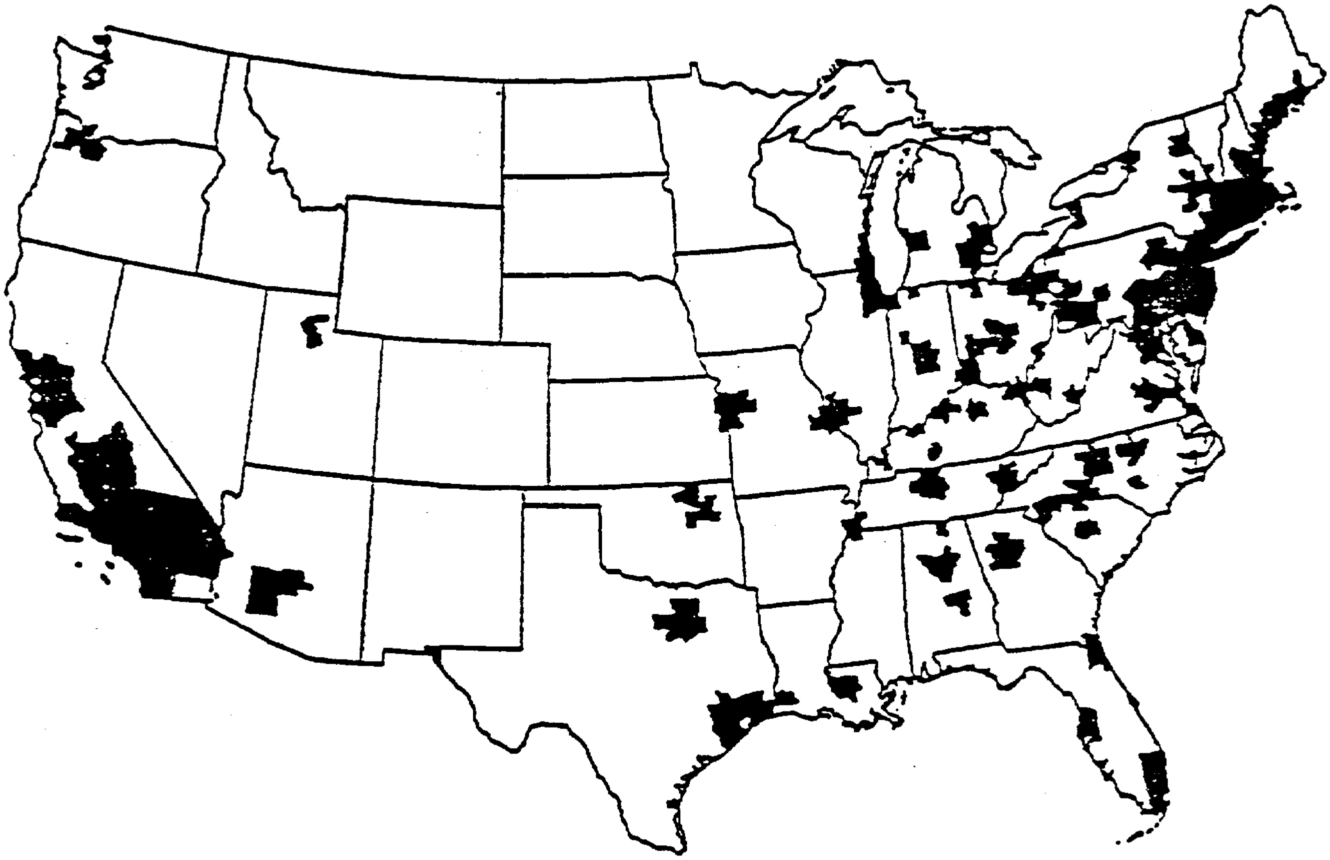


Figure 2-1. Areas exceeding the ozone NAAQs (U.S. EPA, 1992a).

a source-by-source basis. For example, some state requirements, along with local air quality regulations, limit gas phase emissions from stripping processes (mechanical or packed tower) and from reactivation systems using GAC because of the possibility of generating contaminants such as VOCs or radon. Some states do not permit radionuclide air emissions, thereby preventing the use of the packed tower aeration process. Other states require radon off-gas treatment. Some states only require permit applications from source operations where emissions exceed the maximum allowable rate set by the state.

Michigan has an extensive air quality program that encompasses emissions from WTPs. The Michigan Air Pollution Act authorizes the Michigan Air Pollution Control Commission (MAPCC) and the Michigan Department of Natural Resource's Air Quality Division to issue permits for the installation and operation of equipment or processes that may emit air contaminants. Based on this authority, MAPCC has promulgated a set of General Rules (amended April 17, 1992) concerning air use approval. These regulations require the issuance of an air use permit from MAPCC for the installation or modification of any process or equipment that may emit an air contaminant.

After installing the equipment, MAPCC must issue an operating permit before the equipment can go into operation full time. Rule 285 of MAPCC General Rules outlines permit system exemptions. Subsection (j) of this rule exempts lagoons and sewage treatment equipment from permitting requirements except for lagoons and equipment primarily designed to treat VOCs in wastewater or ground water, unless the emissions from these lagoons and equipment are only released into the general in-plant environment.

Air use permit applications submitted to MAPCC must include descriptions of the equipment; the site; the exhaust system configuration; data on the exhaust gas flow rate; an operating schedule; and information on any air pollutants to be discharged. Application review consists of a technical evaluation by the Permit Section engineers and a site evaluation by the Compliance Section district staff. After internal processing is completed, the Air Quality Division develops the necessary permit conditions and stipulations to ensure that the proposed process or plant operates in an environmentally safe and acceptable manner. Under Michigan law, failure to obtain or comply with a permit can result in fines of up to $10,000 and additional fines of up to $2,000 per day for as long as the violation continues.

In California, local districts develop their own regulations and permitting requirements for stationary sources of air emissions. These districts regulate operations that result in air emissions on a source-by-source basis. The Bay Area Air Quality Management District, for instance, has no specific regulation governing air emissions from WTPs but it does have permitting requirements for air stripping processes. California's South Coast Air Quality Management District has a nuisance regulation (Rule 402) that prohibits discharging, from any source, air contaminants that can cause injury, nuisance, or annoyance to the public. WTPs generating air emissions in this district would be evaluated on a case-by-case basis. Applications for air use must be submitted to the district.

The State of New Jersey has not promulgated regulations that specifically address air emissions from WTPs. Subchapter 16 of New Jersey's Air Pollution Control Act, however, outlines requirements for the control and prohibition of air pollution by VOCs. Section 16.6(a) of the law prohibits emitting VOCs into the atmosphere from any source, in excess of the maximum allowable emission rate as set in the regulation. In New Jersey, WTPs generating VOCs would be evaluated on a case-by-case basis.

Chapter 3
Characterization of Water Treatment Plant Residuals

The majority of residuals from water treatment plants (WTPs) fall into one of four categories:

- Naturally occurring, colloidal/particulate matter (e.g., clay, silt, algae) removed from raw water by sedimentation, filtration, membranes, or other processes; and, inert material in treatment chemicals (e.g., grit in lime).
- Naturally occurring, soluble substances (e.g., iron, manganese, calcium, and magnesium) converted to their insoluble precipitate forms by oxidation or pH adjustment.
- Precipitates formed (e.g., $Al(OH)_3$, $Fe(OH)_3$) when chemicals are added to water.
- Spent materials (e.g., granular activated carbon [GAC], powdered activated carbon [PAC], filter media, resins) that must periodically be removed from unit treatment processes after exceeding their useful lives.

These residuals are addressed in this chapter, which includes a discussion of sludges, liquid wastes, radioactive wastes, and physical and chemical characteristics.

3.1 Types and Quantities of Residuals

3.1.1 Sludges

Semi-solid residuals produced from mechanical water clarification processes (e.g., screenings, presedimentation), as well as those produced from the clarification of water that has been chemically preconditioned, are generally referred to as sludges. The three most common types of sludges are coagulant/polymeric, chemical softening, and oxidized iron/manganese.

If a raw water source has a high concentration of total suspended solids (TSS) the alum/iron coagulant sludges will contain a high percentage of gelatinous, hydroxide precipitates (e.g., $Al(OH)_3$, $Fe(OH)_3$), and will exhibit the overall characteristics indicated in Table 3-1. Iron/manganese sludges also tend to be composed of gelatinous hydroxide solids (e.g., $Fe(OH)_3$, $Mn(OH)_2$).

Chemical softening sludges primarily consist of crystalline calcium carbonate ($CaCO_3$), with the magnesium hydroxide ($Mg(OH)_2$) portion of the solids increasing as the magnesium content of the raw water increases. Because of the structured, crystalline nature of the $CaCO_3$ precipitate, the relationship between solids content and the overall sludge characteristics of chemical softening sludges is different from that for coagulant and iron/manganese sludges (see Table 3-2).

Table 3-1. Alum/Iron Coagulant Sludge Characteristics (ASCE/AWWA, 1990)

Solids Content	Sludge Characteristic
0–5%	Liquid
8–12%	Spongy, semi-solid
18–25%	Soft clay
40–50%	Stiff clay

Table 3-2. Chemical Softening Sludge Characteristics (ASCE/AWWA, 1990)

Solids Content	Sludge Characteristic
0–10%	Liquid
25–35%	Viscous liquid
40–50%	Semi-solid
60–70%	Crumbly cake

Several equations can be used to predict the quantity of alum/iron coagulant sludge to be generated, based on the raw water characteristics and amount of coagulant dose. The principal factors used in the estimation of coagulant sludge quantities are: 1) the suspended solids (SS)-to-turbidity ratio for the raw water (Cornwell et al., 1987); and 2) the waters of hydration assumed for the coagulant. Perhaps the most commonly used equations for predicting the quantity of alum or iron coagulant sludge are (Cornwell et al., 1987):

$$S = (8.34Q)(0.44Al + SS + A) \qquad \text{(Eq. 3-1)}$$

where
S = sludge produced (lbs/day)
Q = plant flow, million gallons per day (mgd)
Al = liquid alum dose (mg/L, as 17.1% Al_2O_3)
SS = raw water suspended solids (mg/L)
A = net solids from additional chemicals added such as polymer or PAC (mg/L)

and

$$S = (8.34Q)(2.9Fe + SS + A) \qquad \text{(Eq. 3-2)}$$

where

Fe = iron dose (mg/L, as Fe)

Similar equations can be used to predict the quantity of sludge produced when calcium and magnesium, carbonate and noncarbonate hardness is chemically precipitated. The quantity of sludge produced depends on whether lime/soda ash or caustic soda is used as the softening chemical(s) and on the total amount of hardness that is removed. Obviously, much more sludge is produced when lime, rather than caustic soda, is used to precipitate carbonate hardness, since the calcium associated with the lime must also be precipitated in the chemical softening process.

The quantity of sludge produced when soluble iron (II) and manganese (II) are oxidized to their insoluble precipitate forms (i.e., Fe(III) and Mn(IV)) depends on several factors. The factor that most affects the sludge quantities is the oxidant used (e.g., oxygen, permanganate, chlorine dioxide, ozone). Similar to using lime to precipitate calcium from hard water, using permanganate to oxidize iron or manganese results in more sludge. The manganese associated with the permanganate is reduced from a (VII) to (IV), and is precipitated along with the iron and/or manganese being oxidized.

3.1.2 Liquid Wastes

Perhaps the most common liquid waste generated at WTPs in the past has been spent filter backwash water. The spent filter water associated with filter-to-waste (rewash) has become more common as WTPs prepare for compliance with the Surface Water Treatment Rule. Slow sand filter wastes are also becoming more prevalent as some smaller communities return to the combined physical/biological benefits of slow sand filtration as described in Section 3.1.2.2. Regenerate wastes (i.e., brine and rinse water wastes) associated with ion exchange (IX) facilities continue to be produced by some softening plants. Reject waters from various membrane processes (e.g., reverse osmosis, nanofiltration, ultrafiltration, microfiltration) are gaining prominence as maximum contaminant levels for finished water are set at lower levels for more organic and inorganic contaminants.

3.1.2.1 Spent Filter Backwash Waters

Spent filter backwash water generally represents a volume of 2 to 5 percent of the total water processed at a WTP. The suspended solids concentration of spent filter backwash varies throughout the 10- to 15-minute duration of the backwash, with the water gradually becoming cleaner as the backwash proceeds. The average suspended solids concentration of spent backwash typically falls within the range of 50 to 400 mg/L.

Filter backwash water historically has been returned to the head of a WTP to be processed again. An equalization basin is usually used so that the spent backwash water can be returned to the head of the WTP at a rate less than 10 percent of the raw water flow into the WTP. Concerns over the recycling of microorganisms, aggravation of taste and odor problems, increase in disinfection byproducts, and other issues have drastically reduced the number of WTPs that directly recycle spent filter backwash. Thus, in recent years interest has been generated in better understanding the characteristics of backwash wastes, since these must be processed along with other WTP residual streams.

Another form of waste from filters that is becoming popular again is filter-to-waste, or rewash, which refers to the wasting of filtered water during the ripening stage of a clean filter. Concern over the passage of certain microorganisms (e.g., *Giardia, Cryptosporidium*, and viruses) through the filter media of a freshly backwashed filter has renewed interest in filter-to-waste.

The filter-to-waste period for ripening a freshly backwashed filter at most WTPs ranges from 15 minutes to an hour in length. Some WTPs are finding that the length of this filter ripening period can be shortened by introducing a coagulant aid or by allowing a filter to sit idle for a time between when it has been backwashed and is returned to service. The filtration rate used to ripen a filter varies from one WTP to another. While many WTPs filter-to-waste at the normal filtration rate, some plants filter at only a fraction of the normal rate. Often, this variation is due to the fact that a smaller pipe is available to convey the filter-to-waste flow. The filter-to-waste flow, while not considered to be of a quality that it can be sent directly into the distribution system, is generally a fairly clean waste stream. Therefore, at most WTPs this flow is equalized and returned to the head end of the plant. Other options being used to handle the filter-to-waste flow include discharging it to a local storm sewer with an appropriate National Pollutant Discharge Elimination System (NPDES) permit; discharging it to a sanitary sewer for processing at a local wastewater treatment plant (WWTP); introducing it to the solids handling stream of the WTP; or, treating it with a membrane process prior to returning the flow to the head end of the plant.

3.1.2.2 Slow Sand Filter Wastes

Slow sand filtration is a simple, economical, and generally reliable method of treating low turbidity waters for potable uses. Since application rates for slow sand filtration are low, on the order of 40 to 150 gallons per day per square feet (gpd/ft^2), the process is mostly used by smaller treatment facilities. Organics, silt, and other particles are trapped in the upper portions of the filter, which is periodically removed for cleaning. This process is called scraping and is normally performed by hand, but small mechanical equipment is sometimes used. The filter is resanded after several scrapings when the filter depth reaches a predetermined minimum design thickness. Material for resanding can be either new sand or old filter sand that has been removed and washed. A unique feature of the slow sand filter is the schmutzdecke, a biologically active layer in the top of the filter. Viruses, cysts, and other organisms are almost entirely removed during slow sand filtration. Thus, the sand removed during scraping can contain a fairly active biological population.

Sources of Liquid Residuals

- *Scraping:* Normal scraping removes the top 0.5 to 1 inch of sand (Tanner and Ongerth, 1990), although scraping to a depth of 4 inches has been reported (U.S. EPA, 1985a). Scraping generally is conducted at a given headloss or reduction in flow, but may be initiated on a regular cycle without regard for headloss. Consequently, scraping may occur from three to forty times annually. Some facilities dispose of the removed material by stockpiling it for other uses such as road sanding during the winter or as soil additives. More commonly, the material is washed and then stored for later addition back to the filter. In this case, the wash water constitutes a residuals stream that may require treatment. Current common disposal methods include discharging to a sewer or a receiving watercourse without treatment. Discharge to a receiving watercourse may require a state or EPA regional NPDES permit.
- *Raking:* Slow sand filters are sometimes raked, usually by hand with a garden rake, to loosen the top layer of material and improve the hydraulic rate without removing sand. This process normally does not produce any waste residuals. Wet-harrow cleaning is sometimes used; this procedure uses a flow of water to flush the raked deposits from the filter, and these deposits may require treatment (Logsdon, 1991). Facilities that practice raking normally need to remove more sand during scraping than facilities that do not rake.
- *Backwash water:* One facility reportedly reverses the flow through the slow sand filter without expanding the bed. The filter is raked prior to backwashing, which promotes removal of the schmutzdecke (Tanner and Ongerth, 1990). The backwash water is normally discharged directly to the receiving water, but may require treatment at some locations.
- *Filter-to-waste:* A fairly large volume of wastewater can be generated during filter-to-waste cycles as some WTPs waste the filtered water for 24 to 48 hours before slow sand filters are placed back on line after cleaning. A filter-to-waste procedure is specified under the current edition of “Ten State Standards” (Great Lakes, 1992). The high quality of this water normally allows disposal without treatment.

Quantities of Residuals

- *Scraping:* Scraping can remove sand to depths of 0.5 to 4 inches. Removing 1 inch of sand will generate 2 to 6 ft^3 of material per 1,000 gpd of filter design capacity, based on design rates of 45 to 150 gpd/ft^2.
- *Spent backwash water:* Because slow sand filters are seldom subjected to any type of backwashing, the majority of WTPs do not generate this particular residuals stream. A slow sand filter in northern Idaho was periodically cleaned by backwashing and scraping, and frequently exhibited higher-than-desired filtered water turbidities (Tanner and Ongerth, 1990). The extent, if any, to which backwashing contributed to this problem is not known, but backwashing is not normally a recommended practice.
- *Filter-to-waste:* Filtering to waste after cleaning is a recommended operating practice, and is required by some states. High quality filtered water is normally discharged to waste without treatment. At some locations, however, this residuals stream may be subject to provisions of an NPDES permit. Waste volumes are generally in the range of 200 to 600 gal/hr/100 ft^2 of slow sand filter area. Filtering-to-waste periods are normally of 24- to 48-hour duration but vary from site to site.

3.1.2.3 Ion Exchange Brine

IX offers the possibility of removing one or more ionic species from one liquid phase and transferring them to another liquid phase via an intermediate solid. In many cases the transfer can be made on a selective basis and with good chemical efficiency. The IX process is primarily used to remove hardness from water, particularly in small water systems and for residential use. IX also has been shown to be effective in removing nitrates and other contaminant ions, including barium, radium, arsenate, selenate, fluoride, lead, and chromate (Pontius, 1990).

In the application of IX to remove hardness, the hardness in the water (most often Ca^{2+} and Mg^{2+}) exchanges with an ion from the resin (generally sodium, Na^+) because the resin prefers the contaminant ions. The reactions are as follows (where X represents the solid IX material):

Carbonate Hardness

$$Ca(HCO_3)_2 + Na_2X \rightarrow CaX + 2NaHCO_3$$

$$Mg(HCO_3)_2 + Na_2X \rightarrow MgX + 2NaHCO_3$$

Noncarbonate Hardness

$CaSO_4 + Na_2X \rightarrow CaX + Na_2SO_4$

$CaCl_2 + Na_2X \rightarrow CaX + 2NaCl$

$MgSO_4 + Na_2X \rightarrow MgX + Na_2SO_4$

$MgCl_2 + Na_2X \rightarrow MgX + 2NaCl$

Regeneration

$CaX + 2NaCl \rightarrow CaCl_2 + Na_2X$

$MgX + 2NaCl \rightarrow MgCl_2 + Na_2X$

Through these reactions, calcium, magnesium, and other materials are removed from the water and replaced by an equivalent amount of sodium (i.e., two sodium ions for each divalent cation removed).

The exchange results in nearly 100 percent removal of hardness from the water; in the process, the resin becomes saturated and reaches its exchange capacity. At this point, breakthrough occurs and hardness can no longer be completely removed from the water. In practice, there is competition for the IX sites by elements other than calcium or magnesium; these other constituents may also limit the effectiveness of the resin at removing hardness. When this situation occurs or the resin is saturated, the IX material is regenerated. Regenerant water containing a large excess of Na^+ (such as a concentrated NaCl solution) is passed through the column to remove the hardness. The mass action of having a large excess of Na^+ in the water causes ions on the resin to be replaced by Na^+ and then enter the water phase. This constitutes a reversal of the initial IX reactions. The regenerated IX material can then be used to remove more hardness (Cornwell et al., 1987).

In addition to this regenerant waste—consisting of sodium, chloride, and hardness ions—wastes produced by IX processes include the backwash water and rinse water used, respectively, before and after the formal regeneration of the resin. The term *regenerant waste*, or brine, is frequently applied to the combination of the used regenerant and the slow rinse, the initial portion of the rinse. When comparing data, it is important to know whether concentrations are reported as only concentration in the regenerant waste itself, or as diluted with rinse water and/or backwash water.

Solids Content

Spent brine often has a very high concentration of total solids and total dissolved solids (TDS). Brine wastes usually contain very few suspended solids. Gradually, resins lose their capacity to be regenerated and upon being replaced, become a solid waste.

pH

IX brine from the water softening process is generally of neutral range pH. More generally, the pH of the brine depends on the nature of the regenerant. Other cation exchange resins may be regenerated using concentrated sulfuric or hydrochloric acid. The resulting regenerant waste will need to be neutralized from its low pH, which is primarily determined by how much excess acid was present in the regenerant solution. Anion exchangers are usually regenerated with a basic material or sodium chloride. Weak basic resins (which will remove strong anions such as chloride, sulfate, and nitrate) are typically regenerated using sodium carbonate. Strong basic resins (which will remove most anions such as chlorides, sulfate, nitrate, bicarbonate, and silica) are regenerated using sodium hydroxide. In this case, the high pH of the resultant regenerant waste will need to be neutralized before being discharged.

Ion Content

Regenerant waste contains the hardness removed from the resin, chloride from the regenerant solution, sodium present as excess regenerant, and smaller amounts of other ions removed from the IX resins. Table 3-3 shows the typical ranges of ion concentrations in the wastewater.

Table 3-3. Typical Chemical Constituents of Ion Exchange Wastewater (AWWARF, 1969)

Constituents	Range of Averages (mg/L)
TDS	15,000–35,000
Ca^{++}	3,000–6,000
Mg^{++}	1,000–2,000
Hardness (as $CaCO_3$)	11,600–23,000
Na^+	2,000–5,000
Cl^-	9,000–22,000

The chemical concentration of brines varies widely from plant to plant, depending on raw water hardness (concentration of the cations to be removed), regenerant dose and concentration, rinsing procedures, and cation exchange capacity of the resin.

Because the exchange of hardness for sodium ions is stoichiometric—that is, one equivalent for one equivalent—the total equivalents of ions in the wastewater will equal the original number of equivalents in the regenerant. The distribution of these equivalents between the various chemical species depends on the IX capacity of the resin and its regeneration efficiency, or taken together, the operating capacity of the resins. The operating capacity is always less than advertised exchange capacity because of incomplete regeneration and contaminant leakage (breakthrough of the least binding counter-ion prior to saturation of the resin).

Toxicity

IX brine typically has a high TDS content, with the predominant ion being the co-ion in the regenerant (chloride, sulfate, carbonate, or sodium, depending on the type of resin and, consequently, the type of regenerant used). The other ions present are those removed by the resin and reclaimed by the regenerant. With chlorides present, brine is corrosive to materials it contacts. Brine possesses varying levels of toxicity to the environment, depending on its TDS level and specific chemical makeup.

Quantity

The total amount of wastewater (spent brine) usually ranges from 1.5 to 10 percent of the amount of water softened, depending on the raw water hardness and the operation of the IX unit (AWWARF, 1969; O'Connor and Novak, 1978).

Figure 3-1 shows the expected wastewater volume as a function of raw water hardness for the case where all other variables are held constant. Table 3-4 reflects how the parameters mentioned above influence the quantity of brine wastewater produced.

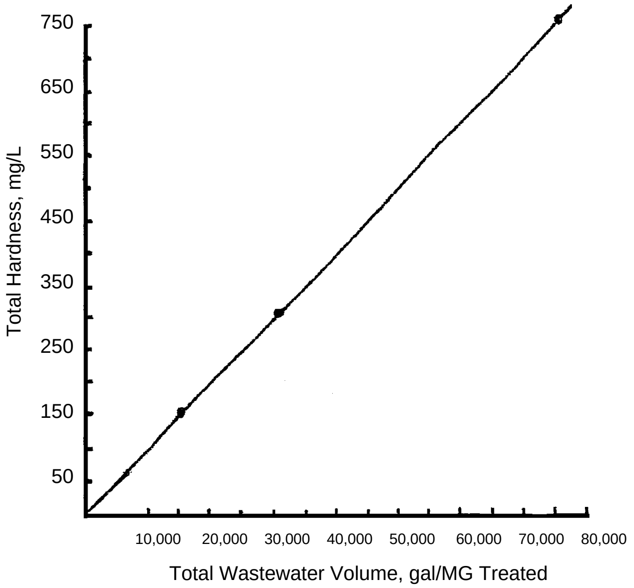


Figure 3-1. Generation of wastewater volumes with ion exchange (U.S. EPA, 1984a).

3.1.2.4 Reverse Osmosis, Nanofiltration, and Electrodialysis-Electrodialysis Reversal

Process Description

Reverse osmosis (RO), nanofiltration (NF) (or membrane softening), ultrafiltration (UF), and microfiltration (MF) membrane processes use semipermeable membranes to remove contaminants from a feedwater. Typically pretreated to minimize scaling and fouling of the membrane, feedwater flows across the membrane surface. Increase in pressure on the feedside of the membrane transports some of the water from the feedside, through the membrane, to the permeate side. Other constituents may be rejected by the membrane, to an extent that depends on the properties of the particular membrane used and the operating conditions. Water and other constituents not permeating the membrane flow out of the membrane system as the concentrate stream. Table 3-5 provides a system operation summary for these membrane processes. Each process has different operating conditions and ranges of feedwater TDS.

At one extreme of high rejection RO membranes, typically used for seawater or brackish water applications, ions may be almost completely rejected (rejections greater than 99 percent). The larger, more effective pore-sized NF membranes reject ions to a lesser degree. Here, the monovalent ions may be rejected to perhaps 50 to 70 percent and multivalent ions (hardness) to perhaps 90 percent levels. Other membrane processes have lower rejection rates of dissolved ions. Species other than dissolved ions, such as dissolved organics, dissolved gases, biological contaminants, and suspended solids, not removed in pretreatment steps,

Table 3-4. Regeneration of Cation Exchange Resins (AWWARF, 1969)

Plant	Gal Wastewater/ 1,000 Gal Water Processed	Raw Water Total Hardness (mg/L as $CaCO_3$)	Gal Regenerant/ft^3 Resin	Gal Rinse/ft^3 Resin	Concentration of Brine (lb/gal)	Gal Processed/ft^3 Resin-Cycle	Dosage (lb salt/ft^3 resin)	Reference
Crystal Lake Plant #6	21.9	233	7.3	19.4	0.90	1,220	6.6	U.S. EPA, 1984a
Crystal Lake Plant #8	17.2	244	5.1	19.0	1.26	1,400	6.5	U.S. EPA, 1984a
Eldon	71.9	375	3.9	61.7	1.43	750	5.6	U.S. EPA, 1976a
Grinnell	49.5	388	14.5	35.0	0.50	1,000	7.2	U.S. EPA, 1976b
Holstein	53.5	885	5.7	19.7	1.16	475	6.6	U.S. EPA, 1976b
Estherville	82.8	915	4.4	24.7	1.25	350	5.5	U.S. EPA, 1976b

Note: The chemical characteristics of the backwash water did not show a large variation from the raw water; therefore, they were not included in this analysis.

Table 3-5. Membrane Process Operations Summary (Mickley et al., 1993)

Membrane Process	Feedwater TDS (mg/L)	Typical Operating Pressure (psi)	Typical System Recovery (%)	System Rejection (%)
Seawater RO	10,000–45,000 (high P)	800–1,200	20–50	99+ (TDS)
Brackish RO	500–3,500 (low P) 3,500–10,000 (medium P)	100–600	60–85	85–96 (TDS) 95–98 (hardness)
Nanofiltration	Up to 500	50–150	75–90	80–90 (hardness)
Ultrafiltration, microfiltration	Not used to remove TDS	Below 100	90 and above	Zero rejection of TDS; rejection of other species dependent on specific membrane
Electrodialysis (ED)	Up to 7,500 (not economical at high TDS)	Not applicable	70–90+	(Effective monovalent removal can be >95 for ED and >99+ for EDR)

Key
P = phosphorus.
TDS = total dissolved solids.

are similarly rejected according to their size and interactions with the membrane.

In the electrodialysis (ED) process there is no pressure used and, thus, no significant bulk flow of water through the membrane. The process employs sets of anion- and cation-selective membranes, electrodes placed outside of the sets of membrane pairs, and an impressed electrical potential across the entire membrane stack. Several flow channels are created between the membrane pairs and between the outer membranes and the electrodes. Water is fed to all channels. The electrical force causes movement of the cations and anions in different directions and out of certain channels into other channels. Some channels become relatively depleted in ions and others become more concentrated. All product streams are combined and all concentrate streams are combined upon exit from the membrane stack. Electrodialysis reversal (EDR) is an ED process in which the polarity of the electrodes is reversed on a prescribed time cycle, thus reversing the direction of ion movement in a membrane stack. The effective rejection of dissolved ions in the ED-EDR process can be quite high (above 90 percent). The EDR process has several advantages over the straight ED system, including lower membrane fouling, less need for pretreatment, and higher reliability. Virtually all new ED plants are of the EDR type. ED-EDR processes do not remove nonpolar contaminants.

UF and MF membrane systems are receiving increased attention because of their potential to remove particulates, microorganisms, and larger organics—constituents addressed in the Safe Drinking Water Act (SDWA) amendments. This discussion, however, is restricted to the RO, NF, and ED-EDR membrane processes.

The recovery rate of these membrane systems refers to the percent of feedwater that is converted into product water. Thus, a 60 percent recovery process would have a concentrate stream of 40 percent of the feedwater flow.

Table 3-6 shows the applications for the RO, NF, and ED-EDR processes and the removal characteristics of the membrane processes. For producing drinking water, NF is limited to freshwater (lower TDS) applications and ED-EDR to applications calling for removal of polar constituents in a nonselective manner.

As of September 1992, 137 drinking water membrane plants sized 25,000 gpd or greater, of the various types mentioned, existed in the continental United States (Mickley et al., 1993). Of these plants, 101 (74 percent) were brackish water RO plants, 16 (12 percent) were NF plants, 13 (9 percent) were ED-EDR plants, and the remaining 7 (5 percent) were seawater RO plants.

Nature of the Concentrate

Most residuals, or more generally, industrial wastes, are characterized by the chemicals added during the processing of the waste. Membrane concentrate, on the other hand, has very few process-added chemicals and thus reflects the character of the raw water used. Membrane processes do not produce more pollutant material or mass—they redistribute, or concentrate, those constituents present in the raw water that are rejected by the membrane.

Generally, posttreatment concentrate contains all the species in the raw water that are not removed in pretreatment. Pretreatment typically consists of acid addition, antiscalant addition, and a 5-µm filtration step. Species that are not rejected by the membrane are present in both the product and concentrate at the same concentration as in the original feedwater. Species that are rejected by the membrane are concentrated to an extent depending on the membrane rejection of the particular species and the membrane system recovery (defined as the percentage of the feed flow recovered as permeate product). The nature of the concentrate may be affected by posttreatment.

Table 3-6. Membrane Process Applications for RO, NF, and ED-EDR (Mickley et al., 1993)

Applications	Reverse Osmosis (RO)	Nanofiltration (NF)	Electrodialysis–Electrodialysis Reversal (ED-EDR)
TDS reduction			
Seawater desalting	Very effective	Not effective	No (not economical at high TDS)
Brackish water desalting	Very effective	Dependent on feedwater makeup	Very effective
Freshwater treatment			
Hardness ions removal (softening)	Very effective	Very effective	No
Dissolved organics removal	Very effective	Very effective	No
Color removal	Very effective	Very effective	No
THM precursor reduction	Very effective	Very effective	No
Specific inorganic and radionuclide removal	Very effective	Dependent on feedwater makeup	Dependent on feedwater makeup

Key
TDS = total dissolved solids.
THM = trihalomethane.

Because membrane concentrate is defined by the raw water characteristics and raw water characteristics are site specific, the specific nature of the concentrate is also site specific.

Solids

Due to the need to prevent plugging of membrane system flow channels, allowable feedwater levels of suspended solids are fairly low for the RO, NF, and ED-EDR systems. Depending on the specific flow channel dimensions, the feedwater limits may range from 1 to 5 SDI units. In general, therefore, the levels of suspended solids are low in the feedwater, and these levels are concentrated to a degree that depends on the system recovery.

The TDS level can vary over a wide range, depending on the TDS level of the feedwater and the membrane system rejection and recovery. Table 3-7 provides ranges for feedwater TDS for the various membrane processes. The amount of TDS in the concentrate depends on the rejection level of the particular membrane system and the process recovery.

pH

The pH of the concentrate depends on the pretreatment pH adjustments and possible posttreatment pH adjustments. The pretreatment adjustments are dictated by the need to control the scaling potential from solubility-limited species, such as calcium carbonate, and by the pH limits of the particular membrane. Typically, the feedwater is acidified to a range of about 5.5 to 7.0. The pH of the concentrate is generally higher than the feedwater pH, due to distribution of the carbonate species between the concentrate and the product streams. Posttreatment pH adjustment may be used to render the concentrate compatible with the receiving water, in the case of disposal to surface waters. Adjustment may be used to ensure the noncorrosive nature of the concentrate in cases where the discharge will be exposed to piping or confining vessels prior to, or as part of, the disposal method.

Table 3-7. Concentration Factors for Different Membrane System Recoveries (Mickley et al., 1993)

Recovery (%)	Concentration Factor
50	2.0
60	2.5
70	3.33
80	5.0
90	10.0

Quantity

The quantity of concentrate is directly related to the recovery, R, of the membrane system. Equation 3-3 can be used to calculate the quantity of concentrate that is generated by the membrane system.

$$Q_c = Q_f(1 - R) \qquad \text{(Eq. 3-3)}$$

where
Q_c = quantity of concentrate flow
Q_f = quantity of feedwater flow
R = recovery rate of the membrane system

Q_c may also be expressed in relation to the product volume flow (Equation 3-4).

$$Q_c = Q_p(1 - R)/R \qquad \text{(Eq. 3-4)}$$

where
Q_p = quantity of product volume flow
R = recovery rate of the membrane system

For example, if the feed flow is 2 million gallons per day (mgd) and the recovery is 70 percent (R = 0.70), then Q_p equals 1.4 mgd and Q_c equals 0.6 mgd from either of the above expressions.

The recovery of a membrane process is generally limited by the potential for sparingly soluble salts, as they become concentrated, to precipitate and scale membranes.

Toxicity

The potential toxicity concerns for concentrate have to do with aquatic species, in the case of surface discharge, and with vegetation, in the case of land applications such as spray irrigation. Potential sources of toxicity include individual components in the raw water that are concentrated in the membrane process to an extent that they are toxic to the life form in question. In general, drinking water membrane concentrate is not toxic as long as the affected life forms are matched to the general TDS level of the concentrate. In the incidences where concentrate is potentially toxic, dilution of the concentrate is generally a means of rendering the concentrate nontoxic.

Ionic Content and Prediction of Concentrate Concentrations

For species that are completely rejected by the membrane and thus totally retained in the concentrate, the degree of concentration, or the concentration factor (CF) may be defined as:

$$CF = 1/(1 - R) \qquad \text{(Eq. 3-5)}$$

where
R = the fractional system recovery

The CFs for different recoveries are shown in Table 3-7.

For example, if the feedwater has a TDS level of 10,000 parts per million (ppm), the rejection of the membrane is assumed to be complete, and the recovery is 60 percent, then the concentrate would have a TDS level of 25,000 ppm (from 10,000 multiplied by a concentration factor of 2.5). Each constituent of the TDS would similarly be present in the concentrate at 2.5 times the feedwater concentration.

As suggested by Table 3-5, complete rejection of constituents is not always the case. For species that are not completely rejected, concentration still takes place, but to a lesser extent. A theoretical expression for this situation (Saltonstall and Lawrence, 1982) is

$$CF = 1/(1 - R)^r \qquad \text{(Eq. 3-6)}$$

where
r = the fractional rejection for the species in question

Note that for the case where r = 1, Equation 3-6 reduces to Equation 3-5.

Table 3-8 shows concentration factors calculated from Equation 3-6.

Each constituent has its own characteristic rejection, which is a function of the particular membrane, the particular species, and the operating conditions. The data in Table 3-8 provide some insight into the complexity of estimating concentrate concentrations, but do not provide a simple means for doing this.

Estimates for concentrate concentrations require the specification of raw water characteristics, the particular membrane used, the membrane system configuration, and the operating conditions. Membrane manufacturers have computer programs that can provide design quality estimates for this. These programs, however, do not provide 1) a rapid and simple means of estimating, or 2) a means for estimating the concentrations of constituents present in minor amounts—constituents that may influence the permitting of concentrate disposal.

Analysis has shown that assumption of 100 percent rejection is quite accurate for seawater and brackish RO membranes (Mickley et al., 1993). Use of the 100 percent rejection assumption will be conservative in the sense of overestimating the concentration of the concentrate, a worst-case scenario. This approach may also be used for EDR systems. The errors should be in the range of 15 percent and under. For NF membrane systems, assumption of complete rejection of all species leads to more significant errors. Consequently, it is recommended that rejections of 70 and 90 percent be used, respectively, for monovalent and multivalent ions.

Other guidelines to be used in estimating concentrate concentrations are:

- Heavy metals should be considered to be rejected as multivalent ions.
- Organics cover a wide range of molecular weights, from less than 100 to greater than 100,000. Most are rejected to a high degree in RO systems, somewhat less so for NF systems, and to a low degree in EDR systems (unless the organics in question are polar in nature). For RO and NF systems, organics of molecular weight of approximately 1,000 may be considered to be completely rejected.
- Non-ionized gases have a rejection of zero in these systems. Thus, the product and concentrate concentrations are the same as for the feed concentration.
- In RO and NF systems, the pH of the permeate will be lower than that of the feedwater, and the pH of the concentrate will be greater than that of the feedwater. The changes are caused by the redistribution of carbonate species across the membrane. The amount of the pH change in permeate and concentrate depends on the feed pH, the recovery, and the amount of carbonate species present. Typical feed pH values are in the range of 5 to 7, depending on the type of membrane. Concentrate pH values may be up to 1 or 1.5 pH units higher than the feed pH,

Table 3-8. Tabulation of Concentration Factors[a] (Mickley et al., 1993)

	Rejection Values						
Recovery Values	**1.0**	**0.99**	**0.98**	**0.95**	**0.90**	**0.80**	**0.70**
0.90	10.0	9.77	9.55	8.91	7.94	6.31	5.61
0.88	8.33	8.16	7.99	7.50	6.74	5.45	4.41
0.86	7.14	7.00	6.87	6.47	5.87	4.82	3.96
0.84	6.25	6.13	6.03	5.70	5.20	4.33	3.61
0.82	5.56	5.46	5.37	5.10	4.68	3.94	3.32
0.80	5.00	4.92	4.84	4.61	4.26	3.62	3.09
0.78	4.55	4.48	4.41	4.21	3.91	3.36	2.89
0.76	4.17	4.11	4.05	3.88	3.61	3.13	2.72
0.74	3.85	3.79	3.74	3.60	3.36	2.94	2.57
0.72	3.57	3.53	3.48	3.35	3.14	2.77	2.44
0.70	3.33	3.29	3.25	3.14	2.96	2.62	2.32
0.68	3.13	3.09	3.05	2.95	2.79	2.49	2.22
0.66	2.94	2.91	2.88	2.79	2.64	2.37	3.13
0.64	2.78	2.75	2.72	2.64	2.51	2.26	2.04
0.62	2.63	2.61	2.58	2.51	2.39	2.17	1.97
0.60	2.50	2.48	2.45	2.39	2.28	2.08	1.90

[a] Calculated using the equation $CF = 1 \div (1 - R)^r$.

prior to any pH adjustment that may be done in post-treatment.

- Although dependent on pH, any chlorine present as essentially non-ionized HClO may be considered as not rejected by membranes.
- Radionuclides should be treated as multivalent ions.
- Fluorine and bicarbonate have pH-dependent rejections that range from values of zero at pH of 5 to typical monovalent rejections at pH of 7.
- Silica rejections are dependent on the type of membrane; the rejections can range from 75 to 98 percent for RO membranes to 20 to 70 percent for NF membranes.

3.1.3 Radioactive Wastes

The types and quantities of radionuclides in residuals depend on the ability of the WTP to remove specific radionuclides from the drinking water. For example, under normal operation, cation exchange treatment will remove radium, but not uranium or radon. Consequently, the regenerant brine waste from a cation exchange system will contain only radium, even though the raw water may contain uranium and radon. Table 3-9 lists drinking water treatment processes, the radioactive contaminant that they remove, and the types of residuals. This list is based upon the known ability of the water treatment processes to remove the specific radionuclides either currently regulated or proposed for regulation for drinking water by EPA (U.S. EPA, 1991b). The radionuclides radium-226, radium-228, uranium, and radon-222, are all naturally occurring.

Radon is a gas than can be removed from drinking water by air stripping and GAC, neither of which produces a residual for routine disposal. Furthermore, radon has a very short half-life of approximately 3.5 days and decays to essentially zero in roughly 28 days. Therefore, radon should not be found in any waste stream from a conventional water treatment process, except in the air from an air stripper. Lead-210, the next long-lived daughter product following radon-222 in the decay series, will be found on any material that adsorbs radon, however.

Some materials used in drinking water treatment processes, either for direct removal of a contaminant such as GAC and IX resins, or indirect removal of a contaminant such as filter sand in conventional treatment, will adsorb radionuclides. When the time arrives for these materials to be replaced, they will contain the radionuclides adsorbed but not removed from the material during the treatment process. A list of drinking water treatment process materials and the potential radionuclides contained on these materials is provided in Table 3-10.

The concentration of radionuclides in waste streams produced by any water treatment process depends on a number of factors: concentration of the radionuclide in the source water, the percent removal of the contaminant, the volume of the waste streams, and the mode of operation of the treatment process. These factors will

Table 3-9. Summary of Treatment Processes and the Types of Wastes Produced From the Removal of Radionuclides From Drinking Water

Treatment Process	Radionuclide Removed	Types of Residual/Waste
Coagulation/Filtration	Uranium	Sludge (alum/iron) Filter backwash water
Lime softening	Radium, uranium	Lime sludge Filter backwash water
Cation exchange	Radium	Brine waste Backwash water
Anion exchange	Uranium	Brine waste Backwash water
Iron removal processes • Oxidation/Filtration • Greensand adsorption	Radium	Filter backwash water
Reverse osmosis	Radium, uranium	Reject water
Electrodialysis	Radium, uranium	Reject water
Air stripping	Radon	Airborne radon

Table 3-10. Water Treatment Process Materials Containing Radionuclides

Treatment Process	Radionuclide Removed	Process Materials
Coagulation/Filtration	Radium, uranium	Filter medium (sand) Filter medium (coal)
Lime softening	Radium, uranium	Filter medium (sand) Filter medium (coal)
Cation exchange	Radium	Resin
Anion exchange	Uranium	Resin
Iron removal processes • Oxidation/Filtration • Greensand adsorption	Radium	Filter medium (sand) Filter medium (coal) Greensand
Reverse osmosis	Radium, uranium	Membrane
Electrodialysis	Radium, uranium	Membrane
GAC adsorption	Radon, uranium, radium	GAC
Selective sorbents	Radium, uranium	Selective sorbent media

Key
GAC = granular activated carbon.

not be identical for any two plants and, consequently, the concentration of radionuclides in the waste streams will be site specific.

A limited amount of information has been reported on the concentration of radionuclides in the waste streams of several water treatment processes. Although these data are site specific, these field measurements can be used for approximating ranges, or levels, of radionuclide concentrations that may be expected in these waste streams.

3.1.3.1 Coagulation/Filtration Wastes

Laboratory, pilot plant, and full-scale system studies have shown that uranium can be removed from source water by conventional coagulation/filtration treatment, and that removals are pH dependent (White and Bondietti, 1983; Lee and Bondietti, 1983; U.S. EPA, 1987). Removals can range from 50 to 85 percent. The technical literature does not reveal any information on the concentration of uranium in the waste streams of the full-scale plants reported upon.

3.1.3.2 Lime Softening Wastes

Lime softening has been found to be very effective in removing both radium and uranium, achieving removals of up to 99 percent of both radionuclides (Lee and Bondietti, 1983; Clifford, 1990; Bennett, 1978; Brink et al., 1978; U.S. EPA, 1976a,b; Meyers et al., 1985; Sorg and Logsdon, 1980; Jelinek and Sorg, 1988; Sorg, 1988). Removals are pH-dependent with highest removals achieved at pH levels of 10.0 to 10.5.

The lime softening process generates a lime sludge that is precipitated during the process and which will contain most of the uranium and radium removed during the treatment cycle. A liquid waste is also produced by the backwashing of the filter media. The backwash water may be recycled to the front of the treatment plant, or disposed of separately or with the lime sludge.

Field data showing specific measurements of radium-226 and 228 in grab samples from full-scale WTP waste streams are listed in Table 3-11. These data show a wide range of radium concentrations in the waste streams. For example, wet sludge from clarifier systems has radium concentrations ranging from 980 to 4,577 pCi/L. Dry weight concentrations vary from 2.8 to 21.6 pCi/g for the same source of sludge. As expected, concentrations of radium-226 in lagoon sludges are higher with volumes ranging from 5,159 to 11,686 pCi/L. Radium-226 in filter backwash water ranges from 6 to 92 pCi/L. No information was found in the literature on the concentration of uranium in lime softening treatment wastes.

3.1.3.3 Cation Exchange Wastes

The chemistry of radium is similar to that of calcium and magnesium (hardness ions). Thus, cation exchange resins in the sodium form used to soften water are very capable of removing radium-226 and radium-228 from drinking water (Clifford, 1990; Bennett, 1978; Brink et al., 1978; U.S. EPA 1976a,b; Meyers et al., 1985; Sorg and Logsdon, 1980). The cation IX regeneration process produces three waste streams: backwash water, regenerant brine, and final rinse water. Although the regenerant brine contains most of the radium released from the resin during the regeneration process, both the initial backwash water and final rinse water will contain some quantity of radium.

The concentration of radium in the waste streams is site specific and depends on the method of plant operation

Table 3-11. Summary of Radium Concentration in Lime Softening Sludges and Backwash Water

Location/Waste	Source Water[a] (pCi/L) Ra-226	Ra-228	Ra-226 pCi/L	Ra-228 pCi/L	Ra-226 pCi/g (dry)	Ra-228 pCi/g (dry)
W. Des Moines, IA	9.3					
Sludge (clarifier drawoff)			76.0	—	—	—
Backwash water			6.3[b]	—	—	—
Lagoon sludge			5,159	596	10.8[b]	1.3
Webster City, IA	6.1					
Sludge			980–1,114	—	—	—
Backwash water			50–92	—	—	—
Elgin, IL	3.5–7.5					
Sludge (clarifier)			—	—	2.8–10.7	—
Sludge (clarifier)			948	873	8.6	8.0
Sludge (blanket)			—	—	1.3–12.5	—
Sludge (lagoon-active)			9,642	9,939	11.3	11.7
Sludge (lagoon-inactive)			11,686	12,167	10.9	11.3
Sludge (lagoon-entrance)			—	—	6–30	—
Backwash water			11.5–21.9[c]	—	—	—
Sludge (filtrate)			0.5–0.48	—	—	—
Bushnell, IL	12.6					
Sludge (clarifier)			4,577[c]	—	21.6[c]	—
Peru, IL	3.1–6.1		—	—	—	—
Backwash water			9.6, 13.8, 87.7[b]	—	—	—
Sludge (pit)			—	—	9.2	—
Colchester, IL	12.1					
Sludge (clarifier)			2,038	236	15.0	1.7
Backwash water			<20	<39	—	—
Beaver Dam, WI	2.7–7.1	1.0–2.4				
Sludge (clarifier)			—	—	1.4	1.4
Wapum, WI	3.3–4.1	1.9–1.4		—	1.9	0.9
Sludge (clarifier)			—	—	1.9	0.9

[a] Cornwell et al., 1987; AWWARF, 1969; U.S. EPA, 1987; U.S. EPA, 1976a; Myers et al., 1985; Sorg and Logsdon, 1980.
[b] Composite samples.
[c] Grab samples.

and regeneration employed. Examples of radium-226 concentrations in waste streams from several IX plants are shown in Table 3-12. The average concentration of radium-226 in backwash water, brine, and rinse water together ranged from 22 to 82 pCi/L. The peak concentrations of radium-226 that occur during the brine cycle ranged from 158 to 3,500 pCi/L.

3.1.3.4 Anion Exchange Wastes

Laboratory, pilot plant, and full-scale system studies have shown that anion exchange treatment (chloride form) is very effective for removing uranium from drinking water (U.S. EPA, 1987; Jelinek and Sorg, 1988; Sorg, 1988). This treatment process is similar to cation exchange for radium and hardness removal. Anion resins are used in the chloride form and are regenerated with sodium chloride (salt). Similar to cation exchange, the anion exchange process produces three waste streams during regeneration: backwash water, regenerant brine, and rinse water. Although the waste is a continuous stream, each portion contains significantly different concentrations of uranium. The highest concentrations occur during brine regeneration; lower concentrations occur during the initial backwash and rinse cycles.

The quantity and concentration of uranium in the waste stream are site specific and depend upon the method of plant operation and regeneration used. Most conventional anion resins have such high uranium removal capacities that treatment runs could last as long as a year or more. Current practice shows, however, that regeneration generally is conducted once every month or two to produce waste streams with lower uranium concentrations. The concentration of uranium in the waste streams depends to a great extent on the raw water concentration and the length of the treatment run (i.e., the volume of water treated).

Field data from a uranium removal plant treating water with a uranium concentration of approximately 130 μg/L for two months show peak uranium concentrations during brine regeneration as high as 9.1 mg/L (see Table 3-13) (Sorg et al., 1991). When the backwash water, brine waste, and rinse water are combined, the average uranium concentration was in the range of 2 to 10 x 10^5 μg/L.

3.1.3.5 Iron Removal Processes Wastes

Oxidation/filtration and greensand filtration are basic water treatment methods for iron and manganese removal, but field data have shown that these processes

Table 3-12. Summary of Radium-226 Concentrations in Brine Waste From Ion Exchange Treatment (Clifford, 1990; Bennett, 1978; Brink et al., 1978; Clifford et al., 1988)

Location	Raw Water Concentration (pCi/L)	Backwash Water (pCi/L)	Brine (pCi/L)	Avg. of Backwash + Brine + Rinse (pCi/L)	Peak Concentration in Wastewater (pCi/L)
Eldon, IA	49	9–30[a]	400–3,500[a]	—	3,500
Estherville, IA	5.7	94[b]	5–320[a]	—	320
Grinnell, IA	6.7	6–19[a] 12[b]	210–320[a]	—	320
Holstein, IA	13	7.8[a]	70–1,100[a]	—	1,100
Dwight Correctional Center, IL (Greensand)	3.3	—	—	22, 27, 29[b]	—
Hersher, IL	14.3	—	—	65–94[b]	315
Lynwood, IL	14.7	—	—	64[b], 70[b], 82[b]	—
Elkhorn, WI	7.2	5.2–7.8[a]	27–158	23.1	158
Redhill Forest, CO	11–35	10–30	11–2,000	—	2,000

[a] Grab samples.
[b] Composite samples.

Table 3-13. Summary of Uranium Concentrations in Ion Exchange Treatment Plant Wastewater (Jelinek and Sorg, 1988; Sorg et al., 1991)

Location	Raw Water Concentration (μg/L)	Backwash[a] (μg/L)	Brine[a] (μg/L)	Rinse[a] (μg/L)	Avg. of[b] Backwash + Brine + Rinse (μg/L x 10^5)	Peak Concentration in Wastewater (g/L)
Arrowbear County Water Distribution, CA	135	2.4–1,040	53.2–6,780,000	36–9,140,000	2.0–9.9	9.1
Coal Creek, CO	39–110	—	—	—	16.5	—

[a] Grab samples during regeneration cycle.
[b] Samples collected from storage tank.

frequently remove some fraction of radium in the source water as well (Bennett, 1978; Brink et al., 1978; U.S. EPA, 1976a,b). These data show a wide range of radium removal efficiencies, from almost 0 percent to as high as 70 percent. These filtration processes produce a backwash water waste stream from the filters; the concentration of radium in this stream will vary widely because of the wide range of removal. Table 3-14 shows data from several filtration systems in Iowa, Illinois, and Wisconsin. The concentration of radium in grab samples of backwash water from these plants ranges from 120 to 1,980 pCi/L, while composite samples range from 34 to 165 pCi/L.

Table 3-14. Summary of Radium-226 Concentrations in Waste Stream From Iron Removal Filters (Clifford, 1990; Bennett, 1978; Brink et al., 1978; Clifford et al., 1988; U.S. EPA, 1992b; Wisconsin, 1984)

Location	Raw Water Ra-226 (pCi/L)	Backwash Water Ra-226 (pCi/L)
Eldon, IA (anthracite filter)	49	254–1,027[a]
Esterville, IA (anthracite filter)	5.7	165–1,980[a] 165[b]
Holstein, IA (anthracite filter)	13	80[b]
Adair, IA (greensand filter)	6.6	190[a]
Stuart (anthracite filter)	16	120–230[a]
Redhills Forest, CO (conventional treatment package plant)	31.5	60 pCi/L (filter backwash)
Kaukauna, WI (greensand filter)	—	52.5[b]
(anthracite filter)		33.5[b]
Elkhorn, WI (sand filter)	7.2	11.9 (avg.)
Herscher, IL (150 avg.) (anthracite filter)	14.3	144[b], 159[b], 149[b]

[a] Grab samples.
[b] Composite sample.

3.1.3.6 Reverse Osmosis and Electrodialysis Wastes

RO and ED are capable of removing anions and cations and, therefore, are effective for the removal of both radium and uranium from backwash waste (Clifford, 1990; U.S. EPA, 1976a; Sorg and Logsdon, 1980; Sorg et al., 1991). The quantity and concentrations of radioactive contaminants in reject water depend on the concentration of contaminants in the source water, the removal rate, and the fraction of water rejected. Assuming a high contaminant removal rate of 95 percent or greater, a good approximation of the concentration of radionuclides in reject water can be easily calculated by the following:

$$\text{Concentration in reject water} = \frac{\text{Concentration of radionuclide in raw water}}{\text{Fraction of water rejected}} \qquad \text{(Eq. 3-7)}$$

Table 3-15 presents radium-226 concentrations in reject waters from several RO plants in Iowa and Florida. The data show a radium-226 concentration range of 7.8 to 43 pCi/L in the reject water.

Uranium levels in reject waters from a pilot plant study conducted by Charlotte Harbour Water Association are shown in Table 3-16. Reject water concentrations range from 301 to 1,125 µg/L for three different RO membranes. This broad range is a result of the wide-ranging feedwater uranium concentrations, 154 to 682 µg/L, and the recovery rates of the individual membranes tested.

Although no field data on the concentration of radium or uranium were found in the literature for ED, the concentration of these contaminants in the reject waste stream from ED systems should be in the same range as RO wastes because of the similarity in the treatment processes used for each method.

Table 3-16. Summary of Uranium Concentration in Reject Water of Reverse Osmosis Treatment (U.S. EPA, 1988c)

Location	System Capacity (1,000 gpd)	System TDS (mg/L)	Raw Water Uranium (µg/L)	Percent Water Rejected (µg/L)	Reject[a] Water Uranium
Charlotte Harbour, FL[b]					
Membrane 1	5	450	682	40	1,125
Membrane 2	5	760	154	50	301
Membrane 3	5	575	277	90	304

[a] Calculated based on 99 percent rejection.
[b] Single membrane.
Key
TDS = total dissolved solids.

3.1.3.7 Waste Materials

Solid waste materials from drinking water treatment processes can be grouped into two categories: 1) selective/specific sorbants that are not regenerated or reused, and 2) materials used in treatment processes that will retain some level of contamination and are replaced periodically (e.g., filter sand). These materials are listed in Table 3-10. Of the two categories, selective/specific sorbants will contain the highest concentration of contaminant by a very large factor because, by design, they are used to remove specific contaminants from source water. Two of the more widely known sorbants are the radium selective complexes (RSC), used for radium removal, and GAC, used for radon removal. The amount of radioactive contaminant retained on

Table 3-15. Summary of Radium-226 Concentrations in Reject Water of Reverse Osmosis Treatment

Location	System Capacity (1,000 gpd)	Raw Water TDS (mg/L)	Raw Water Ra-226 (pCi/L)	Percent Water Rejected (%)	Reject Water Ra-226 (pCi/L)
Bay Lakes Estates MHP, FL[a]	40	2,532	3.2	—	—
Venice, FL[a]	1,000	2,412	3.4	46	7.8
Sorrento Shores, FL[a]	200	3,373	4.6	61	7.9
Spanish Lakes MHP, FL[a]	70	1,327	10.4	69	20.5
Nokomis School, FL[a]	0.8	1,442	11.1	—	11.9
Bayfront TP, FL[a]	1.6	895	12.1	72	19.4
Kings Gate TP, FL[a]	30	1,620	15.7	—	—
Sarasota Bay, FL[a]	5	2,430	20.5	50	37.9
Greenfield, IA[b]	150	2,200	7.6	31	43
Lamont, IL[c,d]					
• Pilot system-1	—	510	11.6	50	19.2
• Pilot system-2	—	681	13.9	85	14.2
• Pilot system-3	—	451	13.0	75	14.1

[a] Sorg et al., 1980.
[b] U.S. EPA, 1976b.
[c] Clifford et al., 1988.
[d] Single membranes.

these materials depends on the concentration of the contaminant in the source water, the percent removal of contaminants, and the length of operation (usually determined by the capacity of the sorbant).

RSC material has been used to remove radium directly from source water and from the brine waste of IX processes (Clifford et al., 1988; U.S. EPA, 1988b; U.S. EPA, 1985a). Samples of RSC material from several WTPs in Illinois, Wisconsin, and New Hampshire, show radium concentrations ranging from 3.6 to 9.1 pCi/g (Redhill Forest, CO) (see Table 3-17). The low value of 3.6 nCi/g was the average of three RSC samples from an exhausted bed treating a water source with a total radium concentration of 18.1 pCi/L. The higher concentration of 9.1 nCi/g was the calculated concentration of radium-226 on an RSC system at 1 year, removing radium (1,200 pCi/L) from a spent brine solution of an IX plant. This system was not run to exhaustion at that point in time.

GAC is used to remove radon from source water, but because of the short half-life of radon, the longer-lived decay product, lead-210, is actually contained on the GAC (U.S. EPA, 1990b; Lowry and Brandow, 1985; Watson and Crawford-Brown, 1991). The amount of lead-210 retained on GAC can be calculated from data on the source water radon concentration and the percent removal.

Core GAC samples from two GAC radon removal plants in New Hampshire were analyzed for lead-210 after 293 days of operation (Mont Vernon), and after approximately 4 months (Amherst). As expected, the concentration of lead-210 decreased with bed depth. The maximum level at the top for the Mont Vernon system was 757 pCi/g, and 297 pCi/g for Amherst. The Mont Vernon system had a higher influent radon concentration (210,500 pCi/L) as compared with the Amherst system (41,800 pCi/L), and had been operating for approximately 3 months longer. Thus, a higher level of lead-210 would be anticipated at the Mont Vernon site. Neither system was considered to be close to radon exhaustion, each having an expected bed life of 5 to 10 years.

3.2 Physical Characteristics of Residuals

The increasing need to process WTP residuals in preparation for their final disposal/beneficial reuse has intensified investigations into their physical characteristics. These characteristics significantly affect the ability to handle, thicken, dewater, and convey WTP residuals prior to the disposal/beneficial reuse. The following sections discuss solids content, specific resistance, com-

Table 3-17. Concentration of Radionuclides on Water Treatment Process Media and Materials (Bennett, 1978; Brink et al., 1978)

Location	Treatment Process	Process Media/Material	Radionuclide Ra-226	Concentration
Lamont, IL (Pilot Study)	Radium selective complexer	Resin	Ra-226 Ra-226	3.6 nCi/g (exhaustion) (total radium)
Redhill Forest, CO	Radium selective complexer (brine treatment)	Resin	Ra-226	$770x18^8$ pCi/ft^3
Herscher, IL	Iron removal	Filter media	Ra-226 Ra-228	111.6 pCi/g 38.9 pCi/g
	Cation exchange	Resin	Ra-226	43 pCi/g
Lynwood, IL	Cation exchange	Resin	Ra-226 Ra-228	9.6 pCi/g 6.6 pCi/g
Dwight Correctional Institute, IL	Natural greensand (cation exchange)	Greensand	Ra-226	29–46 pCi/g
Peru, IL	Lime softening	Filter media	Ra-226 Ra-228	4.6 pCi/g 3.6 pCi/g
Elgin, IL	Lime softening	Filter media	Ra-226 Ra-228	16.0 pCi/g 8.3 pCi/g
Elkhorn, WI	Iron removal	Filter media (sand)	Ra-226 Ra-228	1.47 pCi/g 0.48 pCi/g
	Cation exchange	Resin	Ra-226 Ra-228	6.04 pCi/g 2.7 pCi/g
Mt. Vernon, NH	GAC Radon removal	GAC	U-235/238 Pb 210	549–9,050 pCi/g (top of bed) 757 pCi/g (max)
Amherst, NH	GAC Radon removal	GAC	Pb-210	297 pCi/g (max) —

Key
GAC = granular activated carbon.

pressibility, shear stress, density, and particle size—physical characteristics that can affect the solids dewatering process.

3.2.1 Solids Content

The solids content of residuals varies widely, depending on factors such as the raw water characteristics, coagulant type and dose, and whether lime is used. The data shown in Table 3-18 indicate the range of solids found for a variety of water treatment sludges in Missouri. The alum sludges have the lowest solids content, while the lime sludges and sludges from Missouri River plants have the highest. In general, the greater the amount of incorporated suspended solids and calcium carbonate in the sludge, the higher the solids content. Therefore, when low-turbidity water is treated, the solids content of the residuals will be low, while the solids content of residuals from turbid waters will be much higher.

Table 3-18. Settled Solids Concentration of Residuals From Water Treatment Plants in Missouri (Calkins and Novak, 1973)

Location	Sludge	Settled Solids (%)
Higginsville	Alum	3.1
Macon	Alum	3.4
Boonville[a]	Lime and alum backwash	3.96
Jefferson City[a]	Lime and iron backwash	4.1
St. Louis County[a]	Iron backwash	4.62
Moberly	Alum	6.3
Kirksville	Alum	7.8
Kansas City[a]	High magnesium softening sludge	8.0
Boonville[a]	Lime and alum	8.17
St. Louis[a]	Iron backwash	8.95
Boonville	Lime and alum	10.1
St. Joseph[a]	Catfloc backwash	11.3
St. Louis County[a]	Iron (secondary basin)	12.2
Kansas City[a]	Lime and magnesium	15.2
Jefferson City[a]	Lime and iron	19.1
St. Louis[a]	Iron (secondary basin)	19.3
St. Louis County[a]	Iron (primary basin)	21.1
Kansas City[a]	Softening sludge	25.3
Jefferson City[a]	Lime and iron	26.8
Columbia	Lime	33.0
St. Louis[a]	Lime and iron (primary basin)	35.6
St. Joseph[a]	Catfloc	35.8
Mexico	Lime	54.0

In Table 3-19, the data show that the solids content following gravity settling and vacuum dewatering depends on the raw water turbidity, coagulant dose, and coagulation mechanisms. Because the pH at which coagulation occurs will determine the coagulation mechanism, pH will also influence the residuals solids content.

For example, in Table 3-19, the data show that the thickened and dewatered solids content are lower when coagulation is conducted at pH 8.1 than when it occurs at pH 6.5. At a coagulation pH of 6.5, the predominant means of incorporating the natural turbidity-causing particles into the sludge was adsorption-charge neutralization (i.e., the positive hydrolysis species formed by adding alum to the water were able to adsorb on the surface of the natural turbidity particle, neutralizing the negative charge of these particles and allowing them to be incorporated into the sludge). This process results in a lower alum dose and, consequently, in a sludge that is easier to thicken/dewater due to a higher percentage of natural turbidity-causing particles and a lower percentage of gelatinous aluminum hydroxide.

At a pH of 8.1, the predominant means of incorporating the natural turbidity into the sludge was enmeshment in a precipitate, or sweep-floc (i.e., the alum added reacted with the alkalinity of the water to form large amounts of gelatinous aluminum hydroxide precipitate, in which the natural turbidity particles were entrapped). This type of sludge is more difficult to thicken/dewater because of the high percentage of aluminum hydroxide present. Also, alum sludge has been shown to have a lower solids content than iron or lime sludge.

Table 3-19. Effect of Coagulation Mechanism on Alum Sludge Properties (Knocke et al., 1987)

Coagulation Conditions						
Influent Turbidity NTU	Coagulant Dose mg/L	Coagulation pH	Mechanism	Ultimate Thickened Solids Conc. Percent	Specific Resistance r m/kg x 10^{11}	Vacuum Dewatered Solids Conc. Percent
40	10	6.2	Adsorption-charge neutralization	6.0	55	42
40	15	6.3	Adsorption-charge neutralization	5.5	60	22
7	40	6.5	Mixed	1.0	95	15
7	75	7.1	Enmeshment	1.0	150	11
7	75	8.1	Enmeshment	0.5	310	9

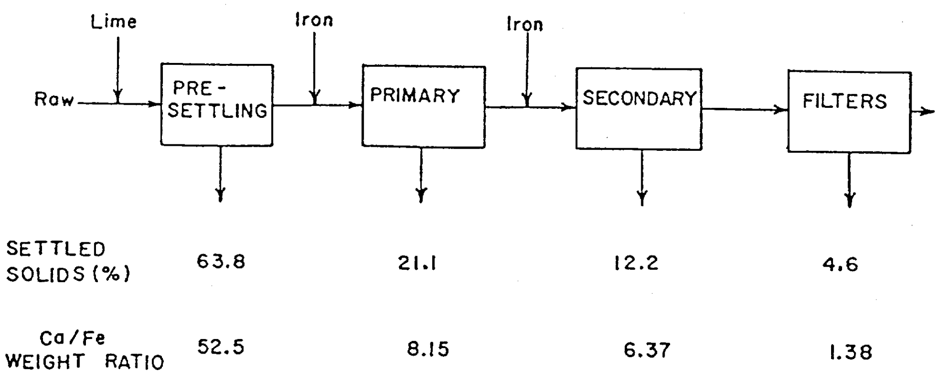


Figure 3-2. Change in sludge settled solids concentration throughout a treatment plant (Calkins and Novak, 1973).

The solids content of residuals varies through a plant and depends on the chemicals added and the addition sequence. In Figure 3-2, the change in solids throughout a Missouri River surface water treatment plant is shown. The settling sludge, consisting of settled clay and calcium carbonate residues, settles to greater than 60 percent solids. At the other end of the plant, the settled filter backwash is 4.6 percent solids. Filter backwash solids generally will consist of the lighter solids that fail to settle in sedimentation basins, so they are usually lower in solids than is material from the clarifiers. As shown in Figure 3-2, the decrease in settled solids corresponds to a decrease in calcium carbonate as reflected by the calcium-to-iron weight ratio of the sludge solids.

With regard to softening residues, calcium carbonate solids form dense slurries with a high solids content. Settled solids in excess of 60 percent have been found for relatively pure calcium carbonate. When $Mg(OH)_2$ is incorporated into softening residues, however, the solids content decreases, usually in direct proportion to the magnesium fraction. Figure 3-3 shows that the settled solids content of softening residues can vary from less than 10 percent to more than 60 percent as the magnesium fraction in the sludge decreases.

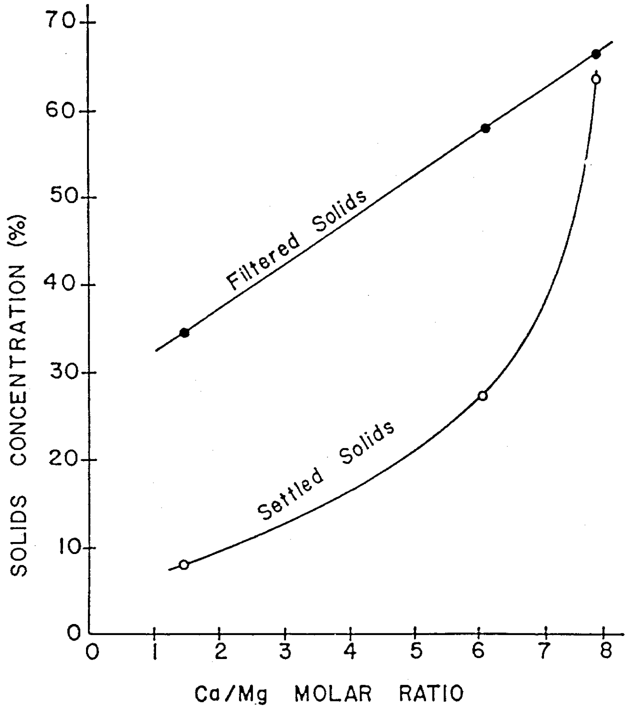


Figure 3-3. Effect of Ca-to-Mg ratio on the solids concentration of softening sludge (Calkins and Novak, 1973).

An operational definition for water in sludges divides the water into three simple fractions—nonremovable water, capillary water, and removable floc water. While there is considerable debate about the types and distribution of water in sludges, this operational definition is useful in estimating the potential solids content that can be achieved by dewatering. Nonremovable water is the water associated with surfaces and has been described as “vicinal” water. This water is thought to form a hydration shell around surfaces. The large crystalline surfaces of calcium carbonate residues will have relatively little nonremovable or visual water because the surface area of this material is relatively small.

Capillary water is that which is trapped between particles and retained by capillary action. Capillary water is removed by application of pressure, which results in flocs moving closer together and squeezing out water between flocs. Removable floc water is water retained within flocs in a manner similar to that of a sponge; it is removed as the flocs are squeezed or compressed by the application of pressure. In effect, both capillary water and removable floc water are expelled simultaneously by the application of pressure. As the pressure increases, more water is removed; at the highest pressure, any remaining water is primarily nonremovable water. This process is depicted in Figure 3-4.

Applied pressure is the major operational variable in dewatering. As the applied pressure of a dewatering process is increased, the cake solids concentration increases, but the lower-solids alum sludge is always well below lime sludge levels, no matter what process

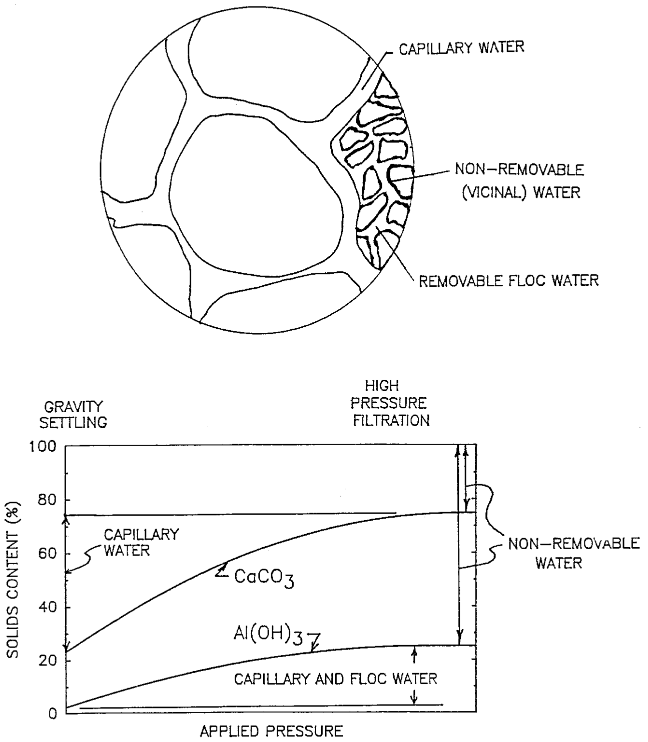


Figure 3-4. Water distribution and removal in a softening ($CaCO_3$) slurry and coagulant ($Al(OH)_3$) slurry (adapted from Novak, 1986).

is used (see Table 3-20). Lime sludge particles contain almost no floc water. Water retained after settling is fixed to surfaces and between particles (capillary water). Alum sludge, in addition to a having higher nonremovable water fraction, contains a large amount of internal floc water.

Because polymer conditioning agents interact only with the outer surface of flocs and do not alter their internal floc structure, polymer conditioning neither affects internal floc water or vicinal water, nor significantly affects dewatered cake solids. Therefore, the solids content of any water treatment residue is determined by the inherent composition of the solids as they form and by the applied pressure of the dewatering device. Once formed, only processes that affect the internal floc structure (such as freeze-thaw conditioning) will substantially influence the solids content that can be achieved by dewatering.

3.2.2 Specific Resistance

Specific resistance is a measure of the rate at which a sludge can be dewatered, and it reflects the size of particles in the filter cake as water passes through the sludge. While initially developed to assess vacuum filter performance, specific resistance also indicates the dewatering rate by a variety of filtration processes, including gravity settling and sand bed dewatering.

A residue's specific resistance to filtration is usually measured using a vacuum filter device. The test is described in detail in Cornwell et al. (1987). Resistance to filtration depends on the porosity or permeability of the sludge cake. Permeability is a function of particle size and particle deformation (compressibility) when pressure is applied. Specific resistance may be calculated from filtration data using the following formula:

$$r = \frac{2\,PA^2 b}{\mu c} \qquad \text{(Eq. 3-8)}$$

where
r = specific resistance to filtration
P = pressure drop across sludge cake
A = surface area of filter
μ = filtrate viscosity
c = weight of dry solids deposited per volume of filtration
b = slope of a plot of t/V versus V
t = time of filtrate
V = filtrate volume

Specific resistance data are useful for comparing sludges and for evaluating the effect of polymers on dewatering (See Figure 3-5). Because the specific resistance of a sludge often depends on the pressure applied during filtration (see Section 3.2.3) and on the mixing applied during chemical addition, resistance values are used primarily as an index of dewaterability rather than as well-defined sludge properties. Sludge

Table 3-20. Specific Gravity of Sludge Particles and Cake Solids Concentrations Obtainable From Various Laboratory Dewatering Methods (Novak, 1989)

			Cake Solids Concentration (%)		
Type of Slurry	Specific Gravity of Particles	Settled Solids Concentration (%)	Vacuum Filtration	Centrifugation	Pressure Filtration
Lime sludge (low Mg)	1.19	28.5	56.1	60.6	69.5
Iron sludge	1.16	26.0	50.1	55.6	64.6
Ferric hydroxide	1.07	7.2	22.7	28.2	36.2
Lime sludge (high Mg)	1.05	5.6	21.0	24.8	34.6
Aluminum hydroxide	1.03	3.6	17.2	19.0	23.2

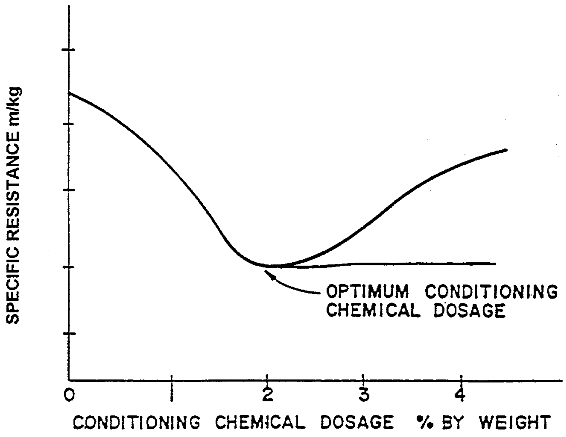


Figure 3-5. Use of specific resistance to determine optimum chemical dosage (Cornwell et al., 1987).

Table 3-21. Specific Resistance for Various Chemical Sludges (Calkins and Novak, 1973)

Location	Sludge	Specific Resistance 10^{10} (m/Kg)
Jefferson City	Lime and iron	2.11
Jefferson City	Lime and iron	4.3
Kansas City	High magnesium softening sludge	5.49
Boonville	Lime and alum	5.83
Boonville	Excess lime and alum backwash	5.98
Jefferson City	Lime and iron	6.12
Jefferson City	Lime and iron	6.79
Jefferson City	Lime and iron	7.0
Kansas City	Softening	11.57
Boonville	Excess lime and alum backwash	13.2
St. Joseph	Cationic flocculent	14.1
St. Louis	Lime and iron	21.2
Kansas City	High magnesium softening sludge	25.1
St. Louis	Iron	40.8
Boonville	Lime and alum	53.4
St. Louis County	Iron backwash	76.8
St. Louis County	Iron	77.6
St. Joseph	Cationic-flocculent backwash	80.1
St. Louis	Iron backwash	121.8
St. Louis County	Iron	148.5
Moberly	Alum	164.3

resistances change during storage as a result of chemical aging and biological activity. Specific resistance values are most useful in comparing treatment options, conditioning chemicals, or mixing conditions.

Specific resistance of sludges varies widely, as shown in Table 3-21. In general, residuals with specific resistance of 10 x 10 m/kg or less are considered to dewater readily, while those with a specific resistance of 100 x 10^{11} m/kg have poor dewaterability. As indicated by the data in Table 3-22, lime softening sludges dewater rapidly and alum sludges from relatively clean surface waters dewater slowly.

Specific resistance for coagulant sludges increases as the pH rises and as the raw water turbidity decreases. The factors in coagulation that lead to high moisture content in the sludge cakes also cause sludge to dewater slowly. Alum sludges from low turbidity raw waters have both a low rate of dewatering and a low solids content.

Chemical conditioners, usually anionic polymers, may be added to sludges to decrease the specific resistance. The selection of polymers and the appropriate dose for conditioning is usually determined by laboratory testing. Factors influencing the polymer dose include the solids content, pH, and mixing intensity.

3.2.3 Compressibility

Most coagulant residues are highly compressible. Compressibility is thought to occur as a result of floc deformation during dewatering. The major consequence of compression is that dewatering rates decrease as the pressure applied during the process increases.

The compressibility of residues is often represented by the following equation:

$$r_2 = r_1 \Delta P^S \qquad \text{(Eq. 3-9)}$$

where

r_1 = specific resistance at pressure, P_1
r_2 = specific resistance at pressure, P_2
$\Delta P = P_2 - P_1$
S = coefficient of compressibility

The coefficient of compressibility, S, is usually determined from a log-log plot of specific resistance (R) versus pressure (P). For coagulant residues, values of S vary from approximately 0.8 to 1.5, while values for softening residues may be as low as 0.4 for materials comprised primarily of calcium carbonate.

Similar relationships have been described, where compressibility relates to empirical constants that are determined experimentally. A somewhat different approach describes the formation of a skin at the surface of the filtering medium that resists the transport of water through a filter cake. These approaches all recognize

Table 3-22. Summary of Floc Density and Dewatered Solids Concentration Data for Several Chemical Sludges (Knocke et al., 1993)

			Maximum Dewatered Solids Concentration (% by wt)			
Sludge Type	**Floc Density (g/mL)**	**Dry Solids Density (g/mL)**	**Grav. Thick.**	**Vacuum Filter**	**Centrifuge**	**Pres. Filt.**
Polymer WTP	1.06–1.08	1.58	2.7	17	11	21
Polymer WTP[a]	>1.30	1.60	25	45	33	40
Iron I	1.26–1.28	2.86	8.9	27	31	30
Iron I	>1.30	2.86	16	54	44	53
Alum I	1.22–1.25	2.55	3.0	17	15	25
Alum III	1.14–1.16	2.45	3.0	14	10	—
Lime	>1.30	2.47	13	41	42	—

[a] Freeze-thaw conditioning provided.

that residuals tend to deform under pressure, resulting in an increased resistance to filtration.

Some of the changes in filtration resistance are attributable to particle shear, which results from the movement of water through the cake. The particles in sludge cakes undergoing filtration have been shown to desegregate, an effect that was especially troublesome for alum sludges. In contrast, calcium carbonate slurries were resistant to shear. Because the breakup or disaggregation of flocs led to increased polymer conditioning requirements, high pressure dewatering systems may be appropriate for alum sludges.

Generally, solids that consist primarily of coagulant materials from clean raw water sources dewater poorly when pressure is applied. In contrast, solids with a large fraction of rigid particles, such as softening sludges, will not deteriorate as dramatically when pressure is applied. Whether this effect is caused by particle deformation, skin formation, or particle breakup may be less important than the effect high compressibility has on dewatering and on the selection of dewatering equipment.

3.2.4 Shear Stress

Shear stress is an important characteristic in determining the handleability of a sludge. The undrained shear strength of various water treatment residues, shown in Figure 3-6, varies markedly with the solids content. Figure 3-6 also shows that the sludge settled solids concentration provides a reasonable estimate of the range of solids concentrations where a sludge makes the transition from a liquid to a handleable solid. This condition is clearly presented in Figure 3-7, where the solids concentration needed to produce a handleable sludge occurs in a range of 0.02 to 0.05 tons/ft^2. The data in Figures 3-6 and 3-7 also show that alum sludges

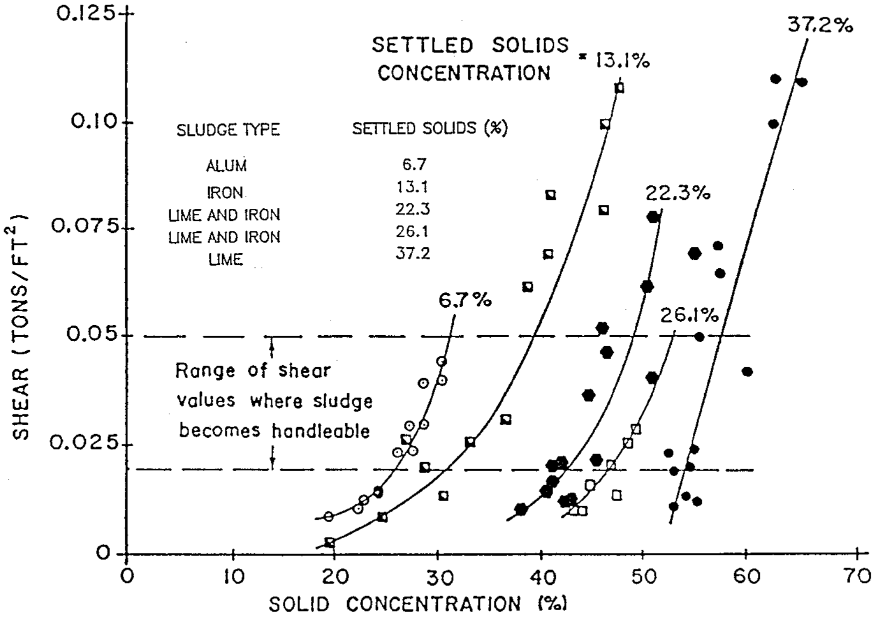


Figure 3-6. Variation in shear strength with sludge moisture content (Novak and Calkins, 1973).

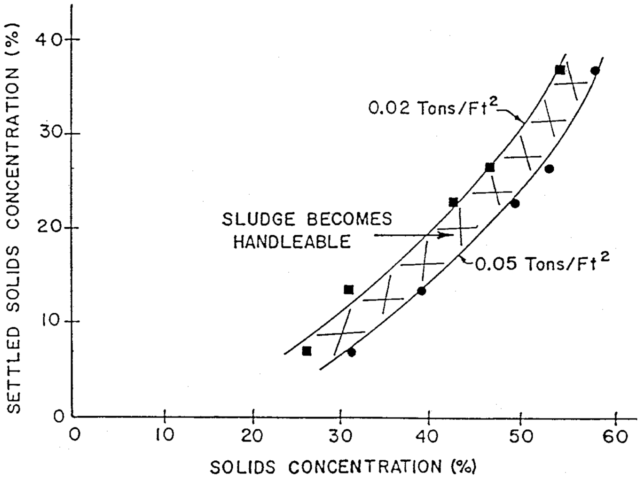


Figure 3-7. Comparison of sludge settled solids concentration with the solids concentration where a sludge becomes "handleable" (Novak and Calkins, 1973).

generally fall in the settled solids range of 7 percent and below. Therefore, solids concentrations of 15 to 20 percent may be sufficient to produce a handleable sludge. In contrast, some softening sludges may require concentrations above 50 percent before they can be handled.

In a study of the dry weight density-moisture relationship for an iron coagulant and two alum coagulant sludges, the iron sludge showed the typical humped curve, where maximum density occurs at an optimum moisture content. The alum sludge reached a maximum density at the lowest moisture content (Figure 3-8). The coagulant residuals were extremely plastic and compressible and greatly exceeded these values for high clay soils.

When landfilling these materials, the solids content should be as high as possible to minimize the amount of landfill space required. In addition, because the cost of landfilling is most often determined on a weight basis, increasing the solids content and reducing the water content of residuals prevents having to pay for the cost of landfilling water.

3.2.5 Density

Floc density varies with floc size, with density decreasing as floc size increases (see Figure 3-9). The major impact of mixing shear is to make flocs smaller. For similar floc sizes, mixing has no effect on floc density, a finding supported by others.

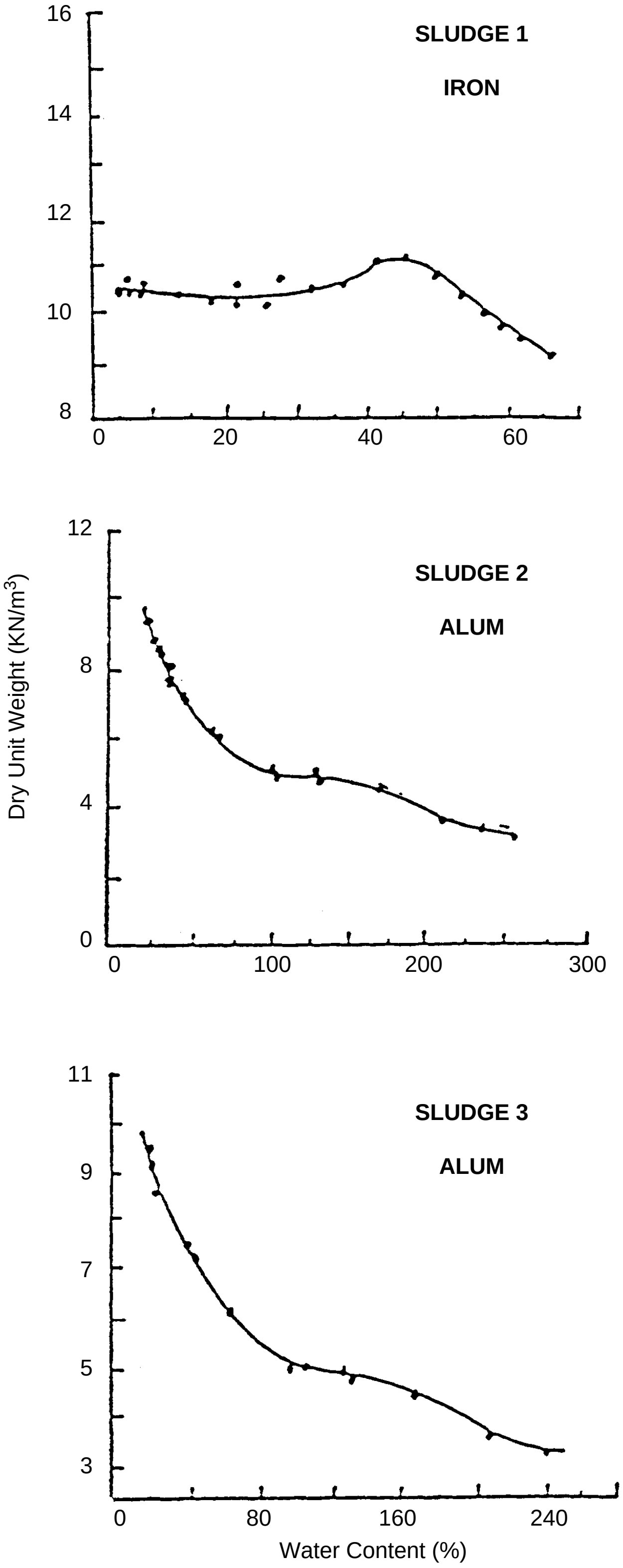


Figure 3-8. Compaction curves of test sludges (Cornwell et al., 1992).

As the volume of suspended solids (Kaolinite clay) in the floc increases, the floc density also increases. Settled and dewatered cake solids increase as the suspended solids in the cake increase. The effect of the solids-to-coagulant ratio on floc density and cake solids suggests that these two factors are related. The apparent density or specific gravity of flocs is a useful predictor of the dewatered cake solids produced by a variety of processes (Table 3-20), with floc density measurements varying from 1.03 to 1.19 for various water treatment residuals. In a study of the effects of organic matter on floc density, when flocs contained more organic matter,

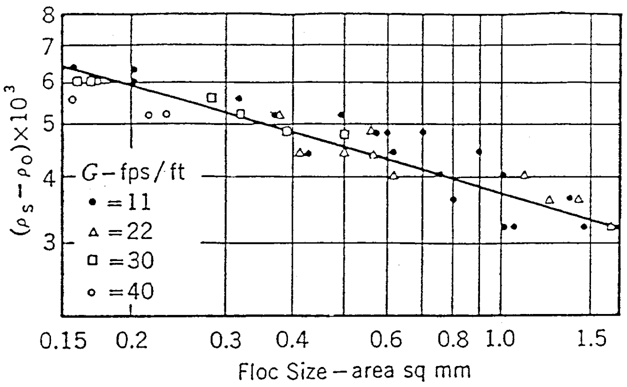


Figure 3-9. Variation of floc density with floc size (Lagvankar and Gemmell, 1968).

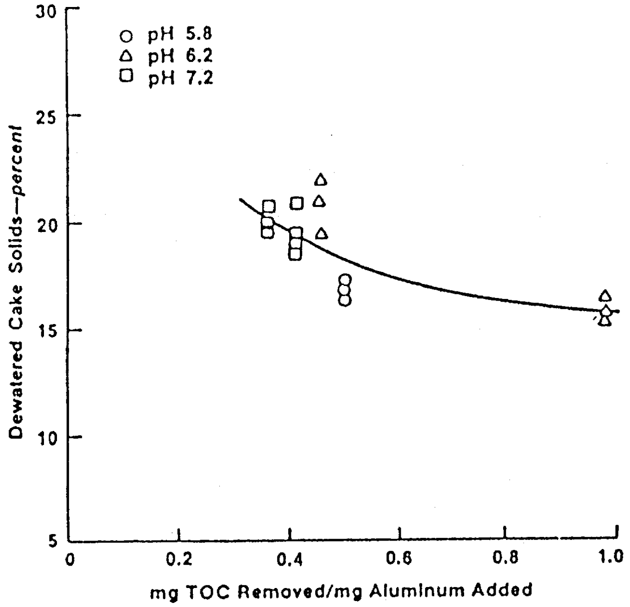


Figure 3-10. Variations in dewatered cake solids concentration of aluminum hydroxide sludges as a function of organic content (Dulin and Knocke, 1989).

their density declined, dewatering rates decreased and dewatered cake solids decreased (see Figures 3-10, 3-11, and 3-12). Floc densities for alum sludges have been reported in the range of 1.14 to 1.22, depending on the amount of total organic content (TOC) incorporated in the flocs.

A recent study comparing various methods of floc density measurement showed that the use of a low osmotic pressure gradient medium to measure floc density is more sound than other commonly used methods such as the sucrose method. High osmotic pressure media are likely to produce densities higher than actual because the high osmotic pressures cause the flow of water out of the floc during measurement. The data presented in Table 3-22 show floc densities ranging from 1.06 to greater than 1.3 g/mL for various water treatment residues. The solids content obtained by various dewatering methods suggests that floc density may be an important determinant in the cake solids obtained by dewatering.

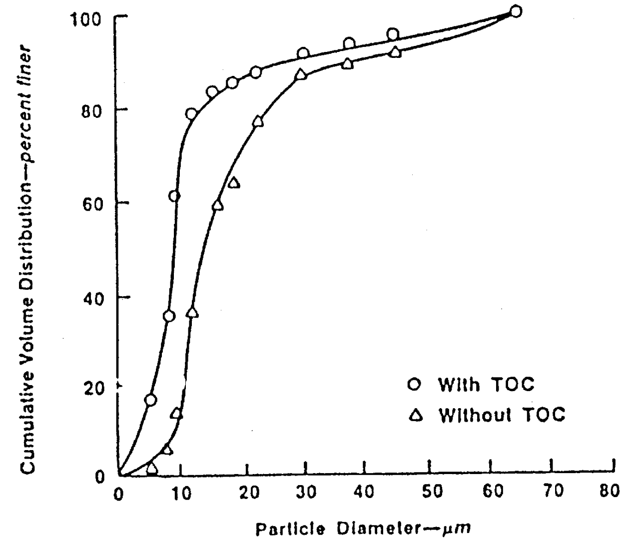


Figure 3-11. Effect of incorporation of organic carbon on the relative size distribution of aluminum hydroxide sludge floc formed at pH 6.5 (Dulin and Knocke, 1989).

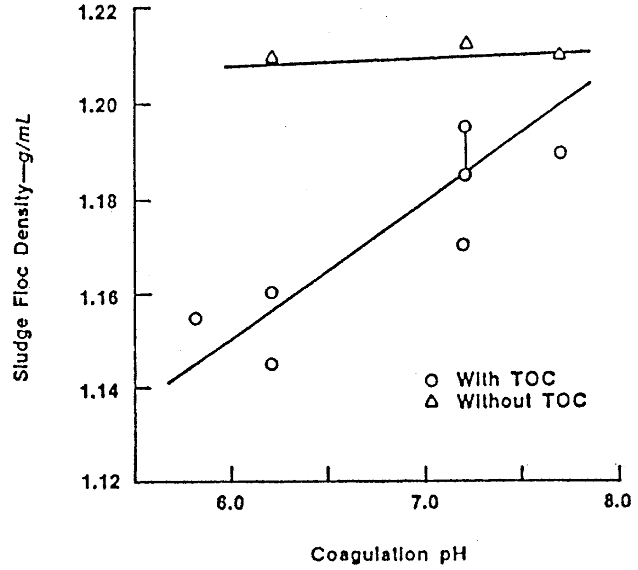


Figure 3-12. Variations in measured floc density as a function of both coagulation pH and presence or absence of TOC from sludge floc matrix (Dulin and Knocke, 1989).

3.2.6 Particle Size Distribution

According to the theory of filtration, the resistance of sludges to filtration is a function of particle size of the flocs in the sludge cake. While several factors such as compressibility and filter media blinding may cause variation from a precise relationship between particle size

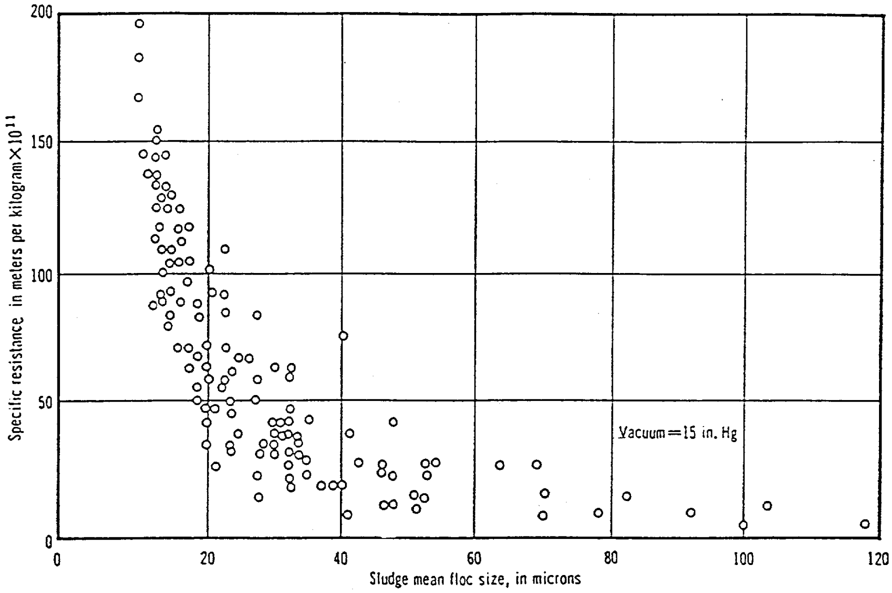


Figure 3-13. Floc size and resistance of metal hydroxide sludges to dewatering by vacuum filtration (Knocke et al., 1980).

and cake resistance, measurements of particle size generally support this theory.

Data presented in Figure 3-13 for various metal hydroxide sludges show the relationship between mean floc size and specific resistance. Additional data shown in Figure 3-14 for alum sludges show a similar trend. These data were measured using a HIAC particle counter and indicate that unconditioned alum sludge has a mean floc size of 20 μ or less.

Conditioning chemicals can also be seen to influence particle size. Data presented in Figure 3-15 show the increase in particle size resulting from addition of polymer and the associated decrease in the specific resistance to filtration.

Although the mean particle size is the primary factor in determining sludge filtration behavior, two other factors are important. If the particle distribution is bimodal, the sludge is susceptible to “blinding,” which is defined as the migration of fines through the cake, resulting in much lower cake permeability near the filtering surface. This phenomenon has been documented for certain sewage sludges but does not appear to be common for WTP residuals.

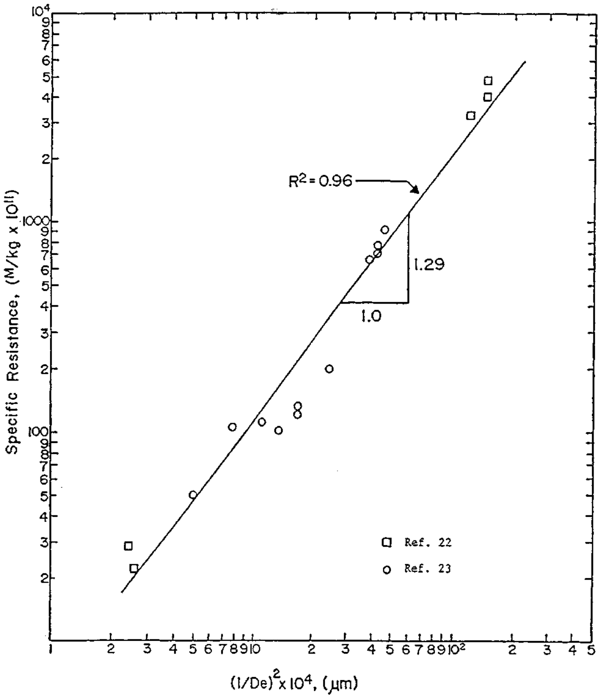


Figure 3-14. Effect of specific surface area on the specific resistance of alum sludges (Knocke et al., 1980).

A more likely problem, especially with alum sludges, is the formation of small particles from the breakup or disaggregation of alum floc due to shear. Alum sludge is very sensitive to shear (Figure 3-16), and shear, G, equal to 500/sec can be attained in filter cakes from the passage of water during vacuum dewatering. Therefore, much of the demand for conditioning chemicals results from the shear associated with the dewatering process and not because alum sludge is comprised of small particles. An important role of conditioning chemicals is to make the sludge resistant to shear.

Particle size, as measured by conventional commercial particle counters, generally verifies that particle counting is a useful means to evaluate sludge dewatering properties. Using microscopic examination and considering the particles to be elliptical, the relationship between

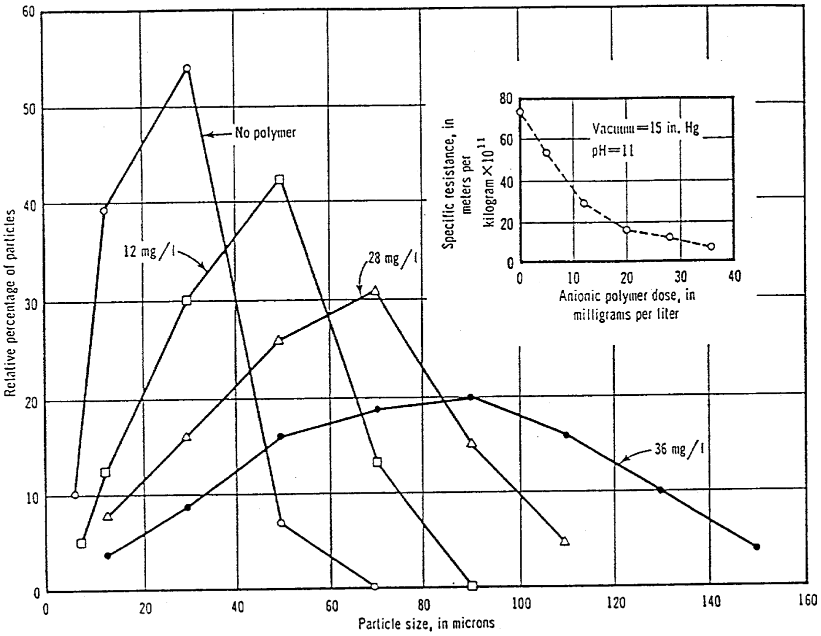


Figure 3-15. Representative results from metal hydroxide sludge conditioning studies (Knocke et al., 1980).

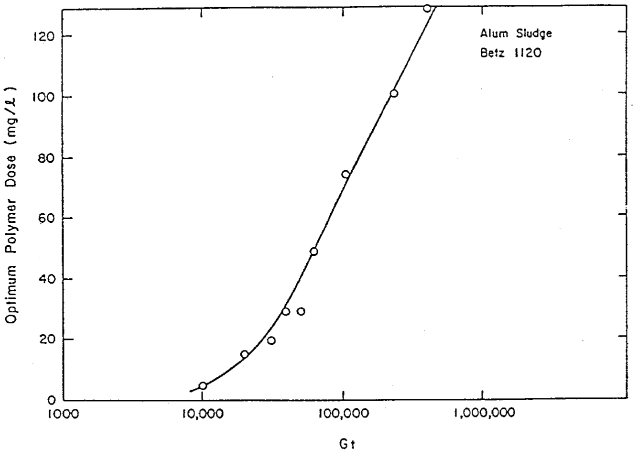


Figure 3-16. Effect of G*t* on optimum polymer dose for alum sludge conditioning (Werle et al., 1984).

particle surface area and specific resistance has been described.

3.3 Chemical Characteristics of Residuals

The chemical characteristics of WTP residuals tend to affect the options for disposal/beneficial reuses more than they affect the ability to handle, thicken, or dewater residuals.

3.3.1 Solids Content

The solids content of WTP residuals varies widely, based on whether the residual is a liquid waste, sludge (i.e., semi-solid waste), or solid waste. Furthermore, the solids content of sludges varies significantly depending on the solids handling processes to which the sludge is subjected (e.g., thickening, dewatering).

The type and concentration of solids affect the distribution of water within a sludge (Cornwell et al., 1987). Because floc water is trapped within the flocs, capillary water is held to sludge flocs by surface tension and attractive forces, and bound water is chemically bound to individual floc particles. Equally important to the solids content is the residual's volatile solids-to-total solids ratio (i.e., VS/TS). Fortunately the majority of the solids in WTP residuals tend to be inert, and the VS-to-TS ratio is typically less than 30 percent. In coagulant sludges, the inert SS tend to be aluminum or iron hydroxide precipitates that can be difficult to dewater because of their gelatinous nature. In softening sludges the inert SS are associated with crystalline calcium carbonate, which drastically reduces the flow water trapped within the floc.

3.3.2 Metals Content

The metals content of WTP residuals is important for a number of reasons: 1) potential impacts on the disposal of the residual in a sanitary landfill; 2) possible inhibitory effects if the residuals are discharged to a WWTP for processing; 3) potential adverse contributions to the residuals from the WWTP based on Part 503 sewage sludge regulations; and 4) possible effects on the whole effluent toxicity of the effluent from the WWTP.

The mean total levels of cadmium, copper, chromium, nickel, lead, and zinc in coagulant sludges from WTPs are generally 10 to 35 percent of the corresponding values for sewage sludges. Except for cadmium, 76 to

87 percent of these were found to be within the oxide or silicate matrix of the alum and iron sludges. Although cadmium could become mobilized under acidic conditions, the levels were measured at levels too low to promote significant leaching of the cadmium. Another investigation into the mobilization of several heavy metals from ground-water and surface water sludges suggests extreme decreases in pH (i.e., less than 2.5) and alternating aerobic/anaerobic conditions are necessary for significant mobilization.

Fortunately, it is known that much of the heavy metals content of WTP sludges is often contributed by impurities in the coagulant. Consequently, heavy metal concentrations in the residuals can be limited by carefully specifying the coagulants and other chemicals added to the water.

3.3.3 Toxicity

Before 1990, the potential toxicity of WTP residuals was determined based on the EP toxicity test. The Toxicity Characteristic Leaching Procedure (TCLP) has now replaced the EP toxicity test. Coagulant sludges from WTPs have been shown to easily meet the TCLP criteria. The metals content of WTP residuals is not anticipated to be a problem under the criteria for the TCLP.

Chapter 4
Water Treatment Residuals Processing

Residuals handling at water treatment plants (WTPs) has traditionally dealt with the handling of waste streams from sedimentation, precipitation, and filtration from conventional coagulation type plants or lime softening facilities. Historically, residuals from these types of facilities have been most commonly disposed of through discharge to sanitary sewers, streams, or similar bodies of water. The changing regulatory environment is leading to an increase in the number of plants incorporating solids handling facilities. This chapter provides information concerning the selection of residuals handling processes for a WTP, including:

- Basic descriptions of standard residuals handling processes.
- Definition of preliminary residuals processing requirements.
- Criteria for the preliminary selection of unit process combinations.
- Discussion of sizing procedures for select unit processes.

This chapter identifies unit processes for treating traditional types of residuals. Changes in the regulatory environment, however, are also requiring an additional form of residuals handling—that of air emissions control. This form of control may be applied to the discharge of gaseous residual byproducts from processes such as ozonation or air stripping.

4.1 Residuals Handling Process Types

Some water treatment processes that may produce solids are grit collection, sedimentation, and filtration (Figure 4-1). These processes can include mechanisms for collection and concentration of solids before conveyance to disposal or to another unit process. These residuals can be handled through a variety of process types. A residuals handling flow schematic with each of the process types is shown in Figure 4-2.

The process types are:

- *Thickening:* A process of concentrating the solids content of a residual stream to reduce the volume before disposal or further treatment.
- *Coagulant recovery:* A treatment technique for improving solids dewatering characteristics and lowering the concentration of metallic ions in the residuals. Recalcination is a related process associated with lime softening sludges (see Chapter 13).
- *Conditioning:* Adding a chemical to a residual or physically altering its nature. Conditioning is traditionally used as a method to optimize the dewatering process.
- *Dewatering:* Similar to thickening in that both processes involve a liquid-solids separation approach with a goal of minimizing the amount of residuals for disposal. Dewatering is defined as a process to increase the solids concentration of residuals (by weight) to greater than 8 percent, typically in the 10 to 20 percent range.
- *Drying:* An extension of the liquid-solids separation approach of thickening and dewatering. Drying is defined as a process to increase the solids concentration of residuals (by weight) to greater than 35 percent.
- *Disposal and reuse:* Removal of residuals from the WTP site or permanent storage of residuals at the WTP site. This category includes hauling to landfill, discharging to sanitary sewer or natural waterway, land application, and various reuse options (e.g., soil supplement, brick manufacture).
- *Recovered and nonrecovered water handling:* Thickening, dewatering, and drying processes produce both liquid and solids components. The solids component may be further treated and disposed of. The liquid component is returned to the main WTP processes if it is recoverable, which means it has little impact on the main treatment process and no harmful effect on the finished water quality. Quality parameters that can affect the recoverable status of the water include the following:
  - Residuals metal concentrations.
  - Disinfectant byproduct formation potential.
  - Use of unapproved polymers in the residuals handling processes.

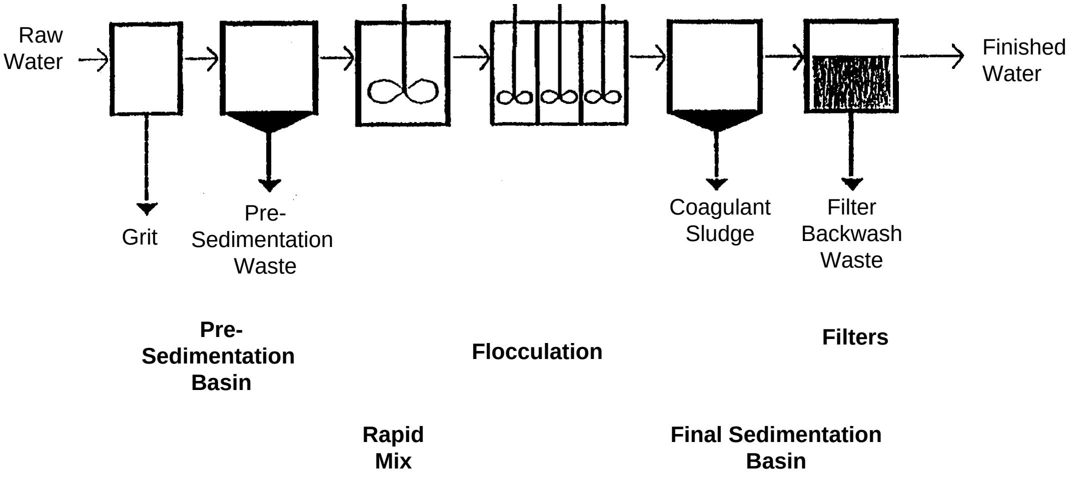


Figure 4-1. Residuals sources in water treatment plants.

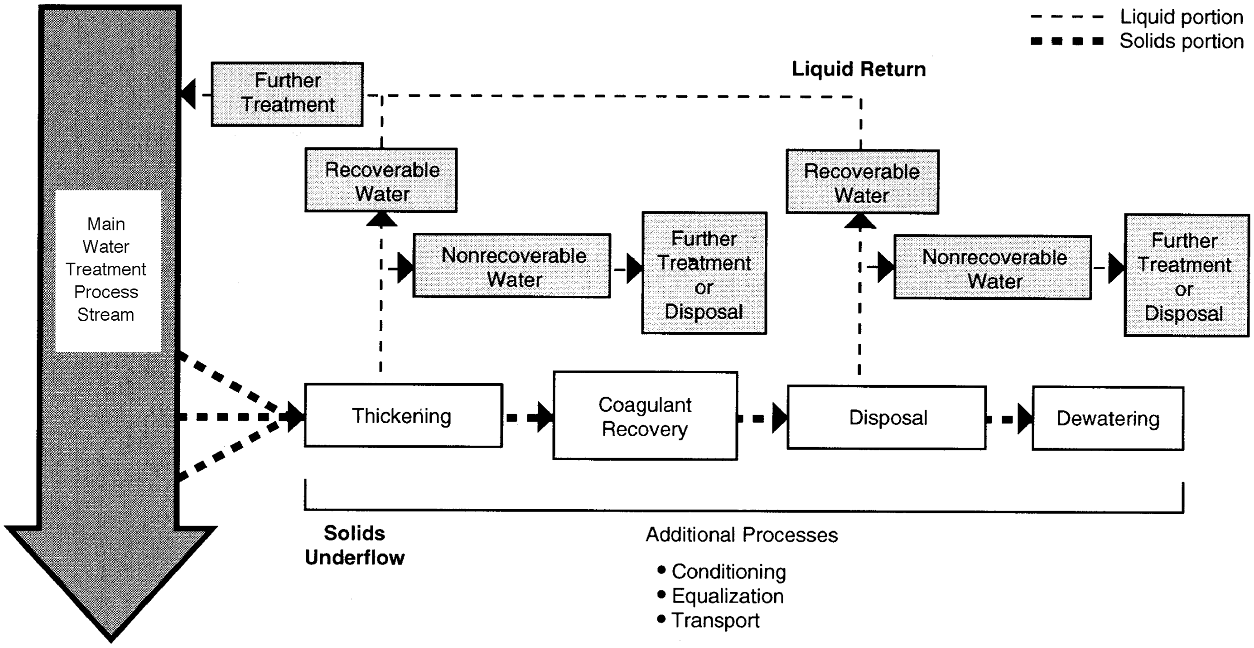


Figure 4-2. Residuals handling process categories.

Nonrecoverable water must either be disposed of or subjected to further treatment.

- *Other processes:* Those that do not readily fit into a category listed above, including equalization, chemical conditioning, and residuals conveyance.

Actual residuals handling facilities may use any or all of these processes in different combinations. Figures 4-3A and 4-3B depict a facility that uses collection, thickening, and dewatering processes. An example of a facility with only one process in operation would be a plant with a sedimentation basin and no sludge collection system. The basin is taken out of operation periodically to manually remove the solids.

4.2 Process Descriptions

4.2.1 Collection Processes

Collection processes are the means by which WTP residuals are collected from the process unit in which they were removed from the water. In the water treatment process, residuals are removed from the process stream by several different mechanisms. Inlet screens remove larger pieces of debris (greater than 1 inch) from the raw water source. Grit basins collect the coarsest, densest material from the raw water source prior to presedimentation. Presedimentation basins collect the denser solids that do not require coagulation and flocculation for solids separation. Sedimentation basins promote gravity settling of solids particles to the bottom of

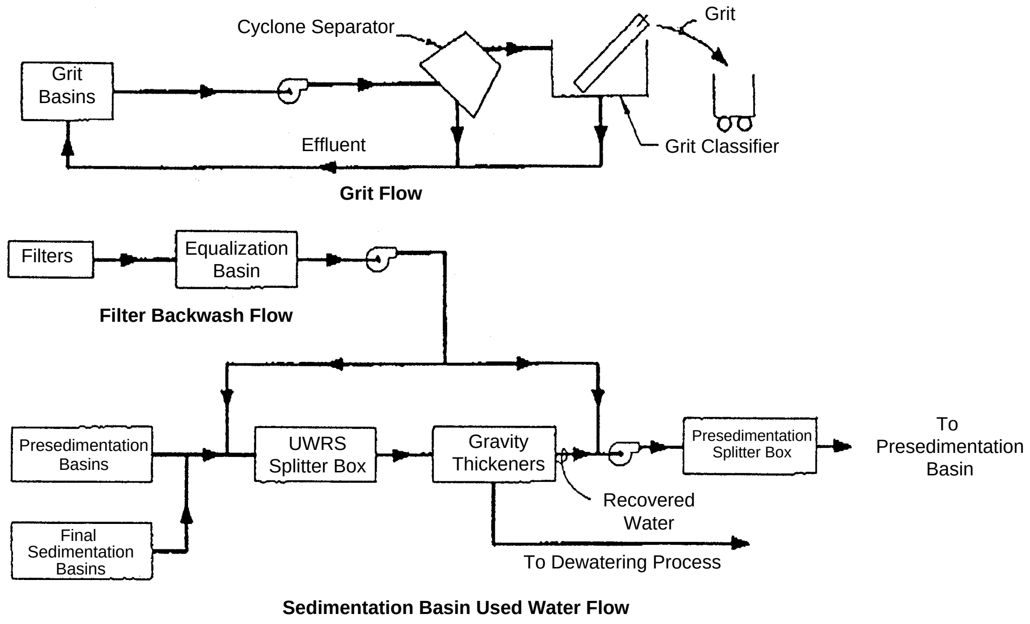


Figure 4-3A. Residuals handling process schematic: sedimentation basin used water flow (Peck et al., 1993).

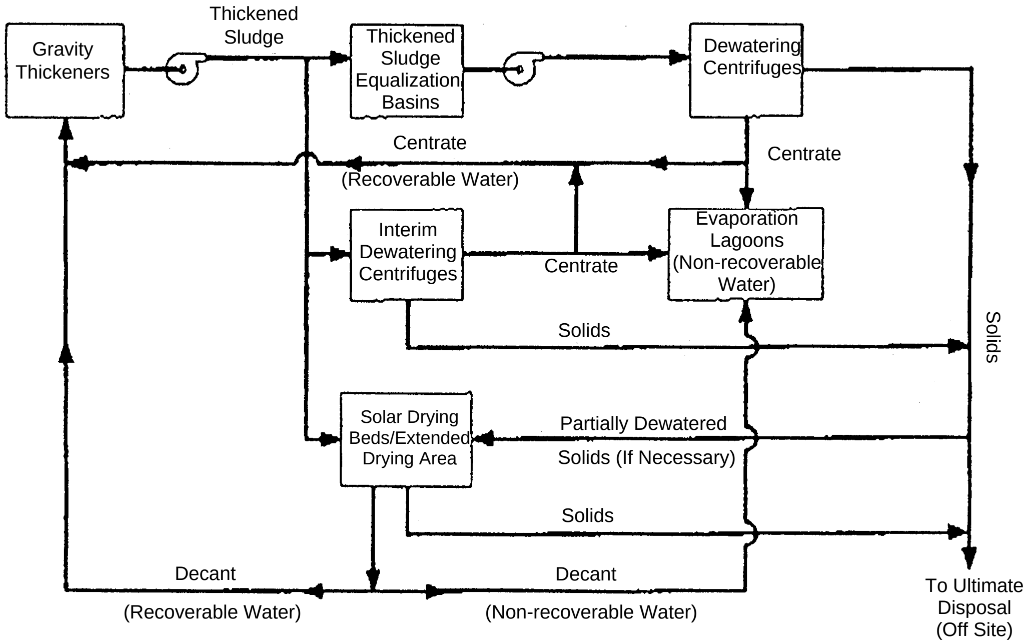


Figure 4-3B. Residuals handling process schematic: solids dewatering (Peck et al., 1993).

a water column where accumulated solids are then removed. Several different types of processes can be used in the sedimentation mode, such as chain and flight, suction, and circular collector units.

In the conventional treatment plant, filtration is generally the last step in the removal of suspended solids. Solids are removed by a bed of granular media (sand, anthracite, and/or garnet) via straining, impingement, gravitational settling, or adsorption. Solids are removed from the bed through a backwash procedure.

4.2.2 Thickening

Residuals concentration, or thickening, processes begin after clarification, sedimentation, filtration, or water softening processes. Concentration processes are critical to the economical removal of solids from the treatment process. Thickening has a direct effect on downstream processes such as conditioning and dewatering, and can make the difference between an efficient, economical operation and an inefficient, high-cost one. WTP residuals are most commonly concentrated using gravity thickeners, but they can also be concentrated in flotation thickeners or by gravity belt thickeners.

4.2.2.1 Gravity Thickening

Thickening of WTP residuals is most commonly performed in gravity thickeners, which work only when the specific gravity of the solids is greater than 1. In this process, both carbonate and metallic hydroxide residuals are conveyed to gravity settling tanks at a flow rate that allows the residuals sufficient retention time to settle.

Gravity thickeners can be either batch feed or continuous flow. Residuals thickened in gravity thickeners may require conditioning.

Thickener tanks (Figure 4-4) are generally circular and are usually concrete, although small tanks can be made from steel. They are typically equipped with rake mechanisms to remove solids. The floors are conical in shape with a slope of between 10 and 20 percent. This slope enables the mechanism to more efficiently move solids to the discharge hopper.

Metallic hydroxide residuals, which come from either clarifier operations or backwashing of filters, thicken to only approximately 1 to 3 percent solids at loadings of 4.0 lb/day/ft^2. The degree of thickening is generally dependent on the hydroxide-to-total suspended solids (TSS) ratio; high TSS solids can thicken to 5 to 30 percent solid. Carbonate residuals produced from water softening processes settle readily and will thicken to concentrations ranging from 15 to 30 percent solids at loadings of 20 to 40 lb/day/ft^2 (Cornwell et al., 1987).

In a continuous feed thickening operation, the solids slurry enters the thickener through ports in a column located in the center of the tank. In theory, the solids are distributed equally, both horizontally and vertically. The solids settle to the bottom of the unit and the clarified supernatant flows over discharge weirs located on the periphery of the tank. These units are equipped with a bottom scraper mechanism that rotates slowly, directing the sludge to the drawoff pipe or sump near the bottom. The thickener bottom is sloped to the center to help collect the sludge. The slow rotation of the scraper also prevents bridging of the solids.

Batch fill thickening tanks are often equipped with bottom hoppers. Sludge flows into these tanks, usually from a batch removal of solids from the sedimentation basin, until the thickening tank is full. The slurry is allowed to settle in a telescoping decant pipe, which may be continuously used to remove supernatant. The decant pipe may be lowered as the solids settle, until the desired solids concentration is reached, or until the slurry will not thicken further. The solids are then pumped out of the bottom hoppers for further treatment or disposal.

4.2.2.2 Flotation Thickening

Flotation thickening is a solids handling option for residuals concentrates consisting of low-density particles. Potential benefits include lower sensitivities to changes in influent feed solids concentration and solids feed rate. The process may also have more applicability for sludges with high hydroxide components (greater than 40 percent by weight). While flotation has been used in the European water industry, it has not been used for long-term, large-scale thickening in the United States (Cornwell and Koppers, 1990). The process is attracting

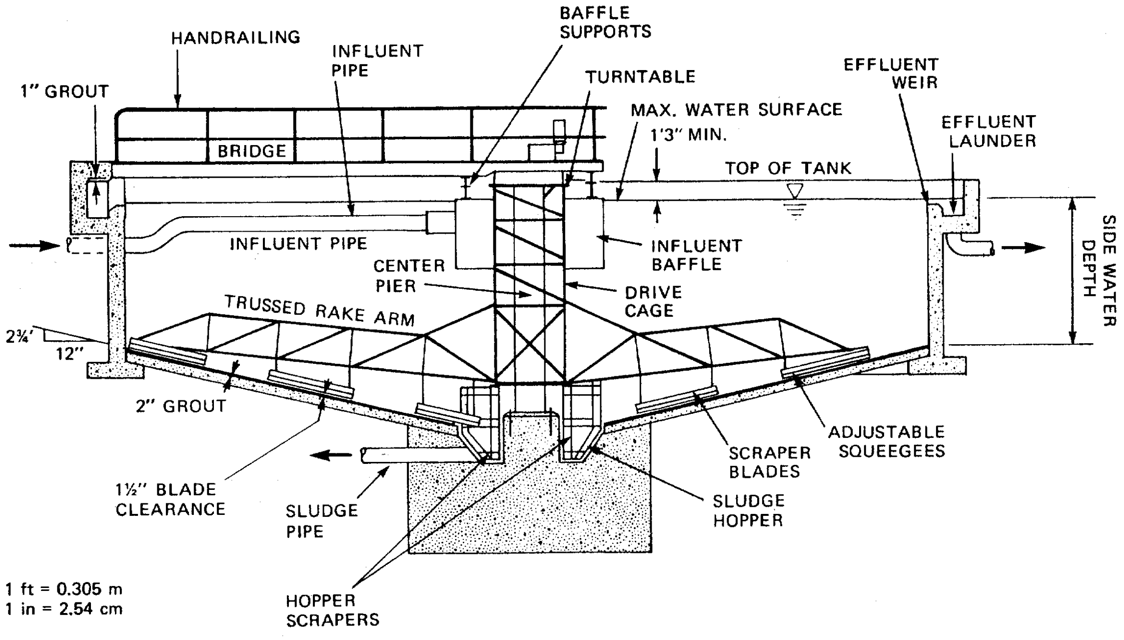


Figure 4-4. Gravity thickener cross-section (U.S. EPA, 1979b).

interest currently as both a concentration process and as a thickening process in the water treatment industry.

Flotation thickening can be performed through any of three techniques.

- *Dissolved air flotation (DAF):* Small air bubbles (50 to 100 µm in diameter) are generated in a basin as the gas returns to the vapor phase in solution after having been supersaturated in the solution.
- *Dispersed air flotation:* Large gas bubbles (500 to 1,000 µm in diameter) are dispersed in solution through a mixer or through a porous media.
- *Vacuum flotation:* Operates on a similar principle to the DAF with the condition of supersaturation being generated through a vacuum.

Each of these techniques uses air bubbles to absorb particles which may then be floated to the water surface for separation from the liquid stream. DAF is the most typically used flotation system in the municipal wastewater industry (U.S. EPA, 1979b).

In the DAF thickening process, air is added at pressures in excess of atmospheric pressure either to the incoming residual stream or to a separate liquid stream. When pressure is reduced and turbulence is created, air in excess of that required for saturation at atmospheric pressure leaves the solution as the 50 to 100 µm-sized bubbles. The bubbles adhere to the suspended particles or become enmeshed in the solids matrix. Because the average density of the solids-air aggregate is less than that of water, the agglomerate floats to the surface. Water drains from the float and affects solids concentration. Float is continuously removed by skimmers.

Usually a recycle flotation system is used for concentrating metallic hydroxide residuals. In this type of system, a portion of the clarified liquor (subnate) or an alternate source containing only minimal suspended matter is pressurized. Once saturated with air, it is combined and mixed with the unthickened residual stream before it is released to the flotation chamber. This system minimizes high shear conditions, an extremely important advantage when dealing with flocculent-type residuals such as metallic hydroxides.

Several sources indicate that European facilities have had success in concentrating hydroxide sludge to levels between 3 to 4 percent solids (ASCE/AWWA, 1990; Brown and Caldwell, 1990). These results appear to include facilities that use flotation both as a concentration process (as an alternative to sedimentation) and as a thickening process. Loading rates for hydroxide sludges vary from 0.4 lb/ft^2/hr to 1.0 lb/ft^2/hr for facilities achieving from 2 to 4 percent float solids concentration. Hydraulic loading of DAF units is reported at less than 2 gpm/ft^2 (Cornwell and Koppers, 1990).

The lack of historic operating data for this process indicates the need for bench- and pilot-scale testing prior to selection of the process.

4.2.2.3 Gravity Belt Thickeners

Although the thickening of hydroxide slurries can be accomplished using gravity belt thickeners, this technology is just beginning to be used. In this process, metallic hydroxide sludge is discharged directly onto a horizontal, porous screen (Figure 4-5). As the sludge moves along the length of the screen, water is removed by gravity. Solids concentrations of 2.5 to 4.5 percent can be achieved using these thickeners. The gravity belt thickening units are made up of a sludge inlet port, drainage screen, scraper blades, discharge outlet for water, and a discharge outlet for the thickened residuals.

4.2.2.4 Other Mechanical Thickening Processes

Although gravity belt thickening is the most common thickening process, some mechanical devices used for dewatering may also be applied to this process. Examples include the continuous-feed polymer thickener, drum thickener, and centrifuges. Solid-bowl-type centrifuges have been used in several pilot-scale studies evaluating residuals thickening. Full-scale operating data on mechanical processes for water treatment solids thickening are not currently available.

4.2.3 Conditioning

Conditioning is a process incorporated into many residuals handling systems to optimize the effectiveness of the dewatering process. Conditioning of WTP residuals is generally done by either chemical conditioning or physical conditioning.

4.2.3.1 Chemical Conditioning

Chemical conditioning is included in most mechanical thickening or dewatering processes. This conditioning involves the addition of ferric chloride, lime, or polymer. The type and dosage of chemical conditioner vary widely with raw water quality, chemical coagulants, pretreatment, desired solids concentration, and thickening/dewatering process used. A recent publication (Malmrose and Wolfe, 1994) identified typical ranges of conditioner use for hydroxide sludges in various mechanical dewatering systems (see Table 4-1). Conditioning agents used for lime sludges are typically lower in volume if used at all. Recorded use of conditioning agents for solids dewatered in open air processes is also minimal.

A wide variety of polymers are available for use in the dewatering processes. The most successful polymers used are anionic with a high molecular weight. Polymers can be obtained in a variety of dry and liquid emulsion

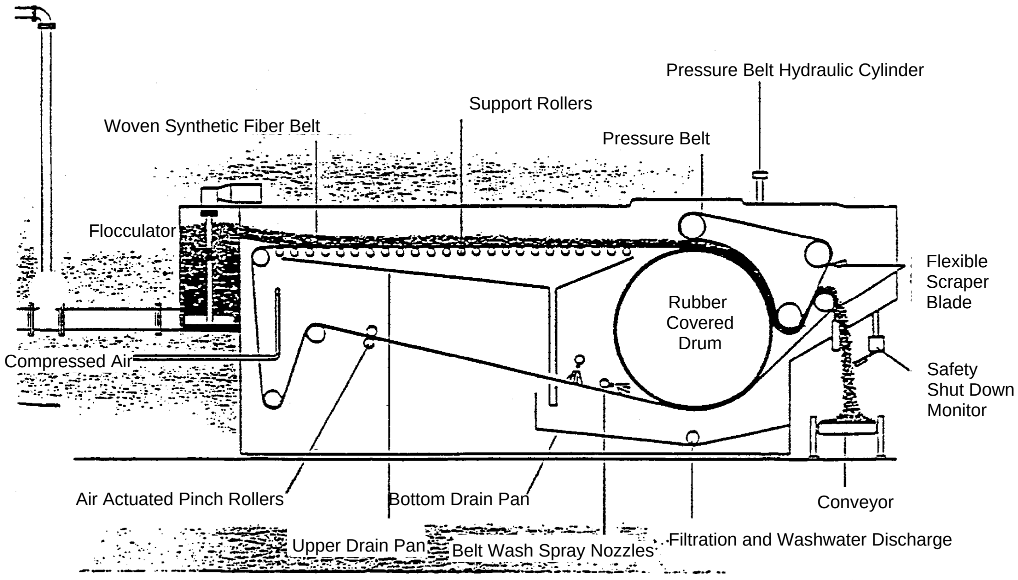


Figure 4-5. Gravity belt thickener cross section (Infilco Degremont, 1994).

Table 4-1. Typical Ranges of Conditioner Use for Hydroxide Sludges in Various Mechanical Dewatering Systems (after Malmrose and Wolfe, 1994)

	Filter Press	Centrifuge	Belt Filter Press
Pyash precoat	10 lb/100 ft^2	—	—
Diatomaceous earth precoat	6 lb/100 ft^2	—	—
Lime	10–30%	—	—
Ferric chloride	4–6	—	1–3
Polymer	3–6	2–4	2–8

Note: All values are in units of lb/ton dry solids unless noted otherwise.

forms. A system should be equipped to feed either form (Malmrose and Wolfe, 1994). Chemical suppliers should be asked for a chemical's NSF rating if process byproduct water is to be returned to the treatment plant stream.

4.2.3.2 Physical Conditioning

The following physical processes may also be used to optimize thickening/dewatering effectiveness (Cornwell and Koppers, 1990):

- *Precoat or nonreactive additives:* Some dewatering systems, primarily vacuum filtration and pressure filters, use a precoat additive in the process, typically diatomaceous earth.
- *Freeze-thaw conditioning:* This process may be accomplished through either an open-air process in cold weather climates or through mechanical equipment.
- *Thermal conditioning at high temperatures (350°F to 400°F) under high pressure (250 to 400 psig):* This conditioning process is typically effective when there is a high organic content present in the solids.

4.2.4 Dewatering

4.2.4.1 Air Drying Processes

Air drying refers to those methods of sludge dewatering that remove moisture either by natural evaporation, gravity, or induced drainage. Most air drying systems were developed for dewatering residuals produced from wastewater treatment plants (WWTPs) but have since been used for the dewatering of WTP residuals. Air drying processes are less complex, easier to operate, and require less operational energy than mechanical systems. They are used less often, however, because they require a great deal of land area, are dependent on climatic conditions, and are labor intensive. The effectiveness of air drying processes is directly related to weather conditions, type of sludge, conditioning chemicals used, and materials used to construct the drying bed.

Sand Drying Beds

Sand beds are commonly used to dewater WTP residuals and have been used successfully for many years. Dewatering on the sand bed occurs through gravity drainage of free water (interstitial water in the residuals slurry), followed by evaporation to the desired solids concentration level. Figure 4-6 illustrates details of a typical sand bed. In areas of high precipitation, covered sand beds have been used.

Residuals on sand beds dewater primarily by drainage and evaporation. Initially, water is drained through the material, into the sand and removed through underdrains. This step, normally a few days in duration, lasts

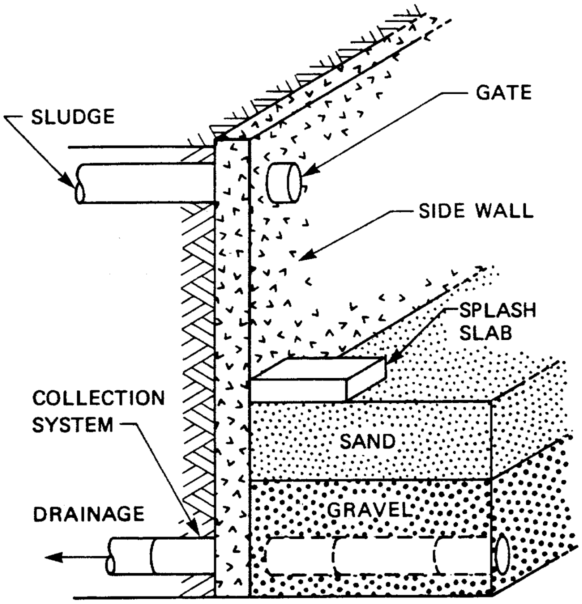


Figure 4-6. Sand drying bed section (U.S. EPA, 1979b).

until the sand is clogged with fine particles or until all the free water has drained away. A decanting process removes any surface water. This decanting step can be especially important for removing rain which, if allowed to accumulate on the surface, can slow the drying process. Water remaining after initial drainage and decanting is removed by evaporation.

Sand beds are more effective for dewatering lime residuals than residuals produced by coagulation with alum. In both cases, however, conditioning the residual slurry before discharging it into the sand bed helps the dewatering process.

When designing a sand bed, the following factors should be considered:

- Required solids concentration of the dewatered residuals.
- Solids concentration of the residual slurry applied to the bed.
- Type of residuals supplied (lime or alum).
- Drainage and evaporation rates.

The required solids concentration depends on the technical or regulatory requirements for final residuals disposal.

The amount of water that can be removed by drainage is strongly influenced by the type of residuals applied to the bed. The rate of evaporation varies with local climatic conditions and the solids surface characteristics. Seasonal evaporation rates can be obtained from local pan values. These values are tested and recorded by the National Weather Service as measures of the local evaporation rate. Because the crust that forms on the surface of the sand bed inhibits evaporation, the pan values must be adjusted when designing the sand bed. An adjustment factor of 0.6 was experimentally derived. Once cracking of the surface occurs, the evaporation rate should again approach the pan value.

Thin layers of solids dry faster than a thick layer, but the annual solids loading is of the depth of the individual layers applied. Using too thin a layer has several disadvantages, including more frequent operation and maintenance, greater sand loss from the bed, and increased costs.

To keep operation and maintenance costs as low as possible, the design goal is to achieve the maximum solids loading with the minimum number of application and removal cycles.

Freeze-Assisted Sand Beds

Alum residuals have a gelatinous consistency that makes them extremely difficult to dewater. By freezing and then thawing alum residuals, the bound water is released from the cells, changing the consistency to a more granular type of material that is much easier to dewater.

Freezing alum residuals changes the structure of the residuals slurry and the characteristics of the solids themselves. In effect, the solids matter is compressed into large discrete conglomerates surrounded by frozen water. When thawing commences, drainage occurs instantaneously through the large pores and channels created by the frozen water. Cracks in the frozen mass also act as conduits to carry off the melt water.

Freezing can be done mechanically or naturally. Because of the high cost associated with mechanical systems, natural systems are used most frequently.

The maximum potential response during both the freezing and thawing portions of the cycle can be obtained by exposing the solids on uncovered beds. The drainage water during thawing may move at a faster rate, and will produce a greater volume than if applying the same unconditioned solids to a conventional sand bed.

The critical operational requirement is that the solids layer be completely frozen before the next layer is applied. Hand probing with a small pick or axe is the easiest way to determine if this has been accomplished.

Solar Drying Beds

Until recently, paved beds used an asphalt or concrete pavement on top of a porous gravel subbase. Unpaved areas constructed as sand drains were placed around the perimeter or along the center of the bed to collect and convey drainage water. The main advantage of this approach was the ability to use relatively heavy equipment for solids removal. Experience showed that the pavement inhibited drainage, so the total bed area had to be greater than that of conventional sand beds to achieve the same results in the same period.

Recent improvements to the paved bed process include a tractor-mounted horizontal auger, or other device, to regularly mix and aerate the sludge. The mixing and aeration break up surface crust that inhibits evaporation, allowing more rapid dewatering than conventional sand beds.

Vacuum-Assisted Drying Beds

This dewatering technology applies a vacuum to the underside of rigid, porous media plates on which chemically conditioned residuals have been placed. The vacuum draws free water through the plates, retaining sludge solids on top and forming a cake of fairly uniform thickness.

Cake solids concentrations of 11 to 17 percent can be obtained on a vacuum-assisted drying bed, depending on the type of solids being dewatered and the kind and amount of conditioning agents used.

Problems with this method stem from two sources: improper conditioning and plate cleaning. The wrong types of polymer, ineffective mixing of polymer and solids slurry, and incorrect dosage result in poor performance of the bed. Overdosing of polymer may lead to progressive plate clogging and the need for special cleaning procedures to regain plate permeability. Plate cleaning is critical. If not performed regularly and properly, the media plates are certain to clog and the beds will not perform as expected. Costly, time-consuming cleaning measures are then required. Removal of dewatered solids tends to be a constant, time-consuming operation.

Wedgewire Beds

The wedgewire, or wedgewater, process is physically similar to the vacuum-assisted drying beds. The medium consists of a septum with wedge-shaped slots approximately 0.01 inches (0.25 mm) wide. This septum serves to support the sludge cake and allow drainage through the slots (Figure 4-7). Through a controlled drainage process, a small amount of hydrostatic suction is exerted on the bed, thus removing water from the sludge.

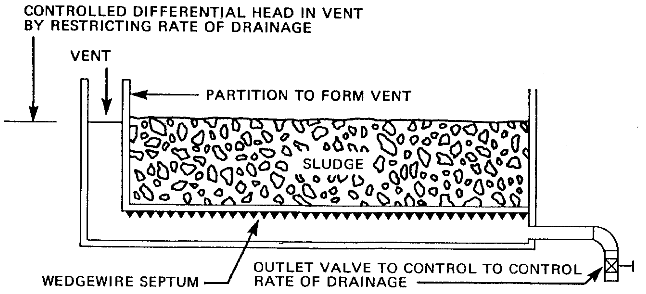


Figure 4-7. Wedgewire drying bed cross section (U.S. EPA, 1979b).

4.2.4.2 Lagoons

Lagoons are one of the oldest processes used to handle water treatment residuals. Lagoons can be used for storage, thickening, dewatering, or drying. In some instances, lagoons have also been used for final disposal of residuals.

The lagoon process involves discharging residuals into a large body of water. Solids settle to the bottom and are retained in the lagoon for a long period. Sedimentation and compression are two mechanisms used to separate the solids from the liquid. Liquid can be decanted from various points and levels in the lagoon. Evaporation may also be used in the separation process if the residuals are to be retained in the lagoon for an extended period.

The traditional lagoon consists either of earthen berms built on the ground surface, or of a large basin excavated from the ground. Various types of systems are installed in lagoons to decant the supernatant and, ultimately, drain the lagoon. State and local regulations have become more stringent about preventing ground pollution, and in some areas, laws affect the design of WTP residual lagoons. Liners made of high-density polyethylene (HDPE), leachate collection systems, and monitoring wells are becoming common features of lagoon designs (see Figure 4-8). Lagoon depth typically varies from 4 to 20 feet and the surface area ranges from 0.5 to 15 acres (Cornwell et al., 1987).

The effectiveness of lagoons in concentrating solids typically depends on the method of operation. For metal hydroxide solids retained in a lagoon for 1 to 3 months, operating the lagoon at full water depth without further air drying of the solids typically results in a solids concentration of 6 to 10 percent. Solids concentrations of 20 to 30 percent may be achieved for lime sludge under the same conditions. Some facilities have achieved solids concentrations above 50 percent by stopping the influent into the lagoon and allowing drying through

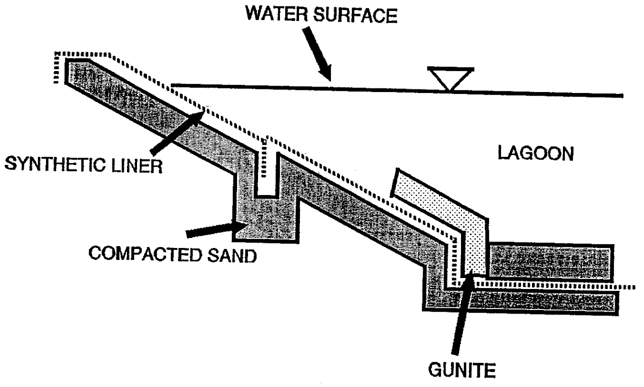


Figure 4-8. Dewatering lagoon cross section.

evaporation. This process may require well over a year of holding the solids in the dewatering lagoon.

The lagoon process may incorporate certain modifications similar to sand and/or solar drying bed systems. Adaptation of a freeze-thaw process to lagoons is common in northern climates.

4.2.4.3 Mechanical Dewatering Equipment

Belt Filter Presses

Belt filter press design is based on a very simple concept. Sludge sandwiched between two porous belts is passed over and under rollers of various diameters. As the roller diameter decreases, pressure is exerted on the sludge, squeezing out water. Although many different belt filter press designs are used, they all incorporate the same basic features—a polymer conditioning zone, a gravity drainage zone, a low pressure zone, and a high pressure zone.

The polymer conditioning zone can be either a small tank with a variable speed mixer (approximately 70 to 100 gallons) located 2 to 3 feet from the press, a rotating drum attached to the top of the press, or an in-line injector. Press manufacturers usually supply the polymer conditioning unit with the belt filter press.

The gravity drainage zone is a flat or slightly inclined belt that is unique to each press model. In this zone, solids are dewatered by the gravity drainage of the free water. If the solids do not drain well in this zone, problems such as solids extruding from between the belts and binding the belt mesh can occur.

The low pressure zone, also called the wedge zone by some manufacturers, is the area where the upper and lower belts come together with the solids in between, thus forming the solids “sandwich.” The low pressure zone prepares the solids by forming a firm cake able to withstand the forces within the high pressure zone.

In the high pressure zone, forces are exerted on the solids by the movement of the upper and lower belts as they go over and under a series of rollers of decreasing diameters. Some manufacturers have an independent high pressure zone that uses belts or hydraulic rams to further increase the pressure on the solids, thus producing a drier cake.

Belt filter presses can be used to dewater the residuals produced from either lime softening processes or alum coagulation. Performance, however, can be affected by many variables, including solids type and characteristics, conditioning requirements, pressure requirements, and belt speed, tension, type, and mesh.

The type and characteristics of the solids to be dewatered are very important in determining the effectiveness of belt press dewatering. Lime softening residuals dewater very readily and are efficiently dewatered on belt filter presses. Since these residuals are more granular in nature, they can withstand high pressures and easily dewater to 50 to 60 percent solids.

Conversely, alum residuals are more difficult to dewater because of the gelatinous nature of the solids. The dewatering results are variable, depending on the source of the water coagulated with alum. An almost pure alum residual is the most difficult to dewater and must be dewatered at low pressures. The pure alum residual will dewater to 15 to 20 percent solids. If the source of the water is a river, and silt and sand are mixed in with the aluminum hydroxide, the slurry will be more easily dewatered, with the resulting cake solids between 40 and 50 percent. Even this type of alum residual, though, must be dewatered at low pressure.

To ensure optimum performance on a belt filter press, lime and alum solids must first be conditioned with polymer. Polymer produces a large, strong floc that allows free water to drain easily from the solids in the gravity drainage zone of the belt filter press. A typical belt filter press is shown in Figure 4-9.

Centrifuges

Centrifugal dewatering of solids is a process that uses the force developed by fast rotation of a cylindrical bowl to separate solids from liquids. When a mixture of solids and water enters the centrifuge, it is forced against the bowl’s interior walls, forming a pool of liquid that separates into two distinct layers. The solid cake and the liquid centrate are then separately discharged from the unit. Both types of centrifuges used to dewater WTP residuals—basket and solid-bowl—use these basic principles. They are differentiated by method of solids feed, magnitude of applied centrifugal force, cost, and performance.

Although commonly used in the past, basket centrifuges are now rarely used to dewater WTP residuals because they are a batch process, are more difficult to operate, and do not perform as well as solid-bowl centrifuges.

The solid-bowl centrifuge, also known as the decanter, conveyor, or scroll centrifuge, is characterized by a rotating cylindrical conical bowl (Figure 4-10). A helical screw conveyor fits inside the bowl with a small clearance between its outer edge and the inner surface of the bowl. The conveyor rotates at a lower or higher speed than that at which the bowl is rotating. This difference in revolutions per minute (rpm) between the bowl and scroll is known as the differential speed and causes the solids to be conveyed from the zone of the stationary feed pipe, where the sludge enters, to the dewatering beach, where the sludge is discharged. The scroll pushes the collected solids along the bowl wall and up

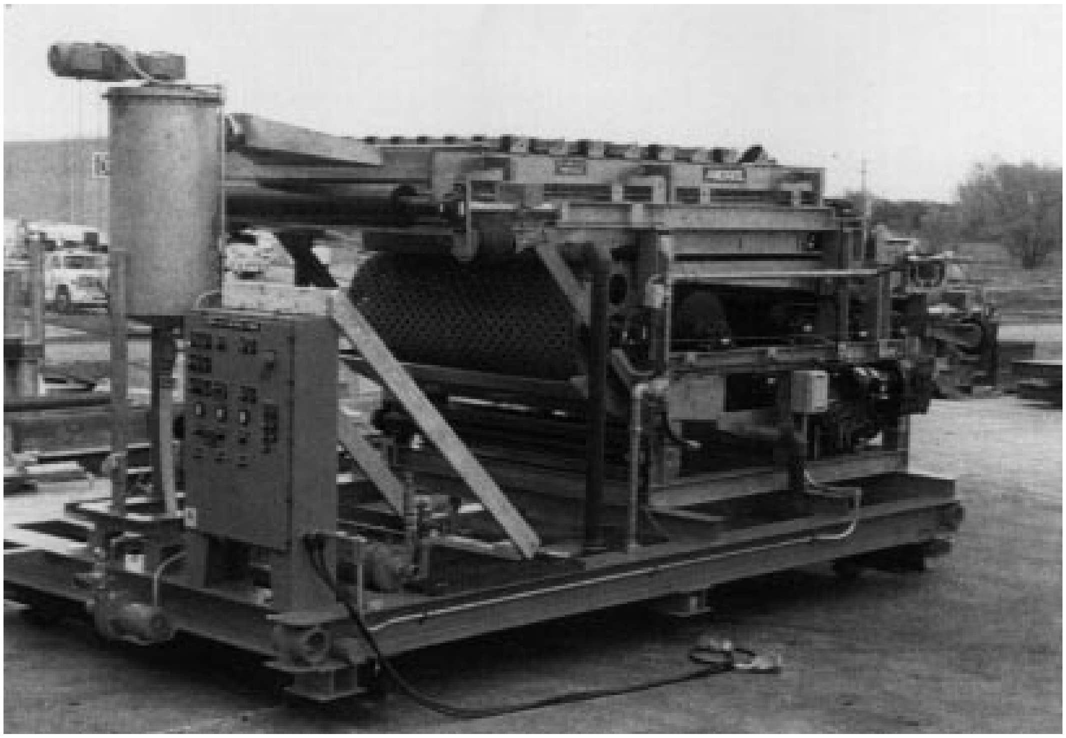


Figure 4-9. Belt filter press (Andritz Ruthner, 1994).

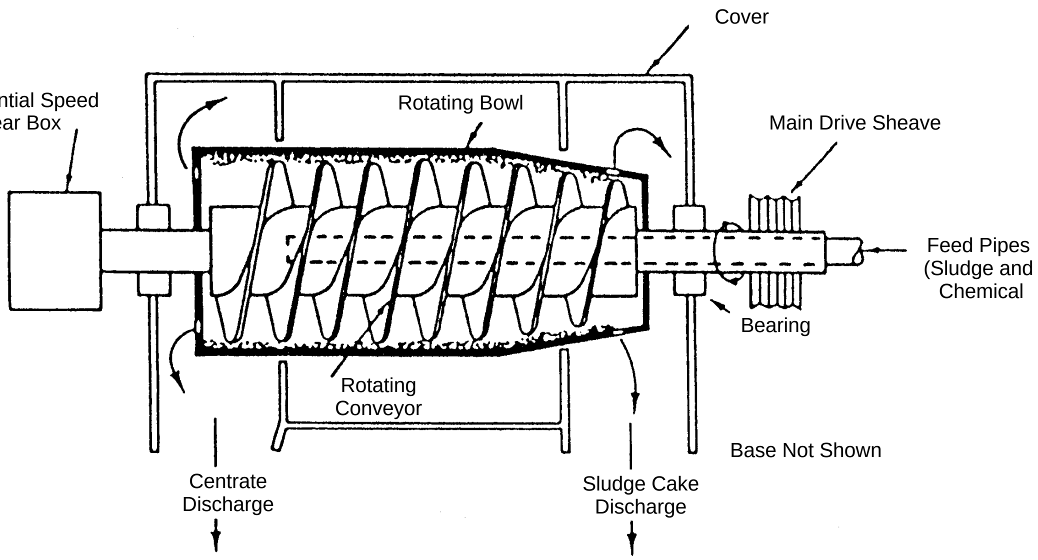


Figure 4-10. Solid-bowl-type centrifuge schematic (Cornwell et al., 1987).

the dewatering beach at the tapered end of the bowl for final dewatering and discharge.

The differential speed between the bowl and conveyor is maintained by several methods. Earlier designs used a double output gear box that imparted different speeds as a function of the gear ratio. It was possible to vary the output ratio by driving two separate input shafts. Eddy current brakes are also used to control the differential. The latest designs can maximize solids concentration through automatic speed control as a function of conveyor torque.

The solid-bowl centrifuge operates in one of two modes: counter-current or continuous concurrent. The major differences in design are the location of the sludge feed ports, the removal of centrate, and the internal flow patterns of the liquid and solid phases.

For the most part, solid-bowl centrifuges use organic polyelectrolytes for flocculating purposes. Polymer use improves centrate clarity, increases capacity, often improves the conveying characteristics of the solids being discharged, and often increases cake dryness. Cake solids concentrations vary considerably, depending on the type of alum residuals being dewatered and the source of the water. High turbidity water yields much higher cake solids concentrations than does low turbidity water. Lime residuals with high cake solids concentrations dewater very easily. Polymer dosages also vary, depending on the source of water and the magnesium and calcium concentrations.

The machine variables that affect performance include:

- Bowl diameter
- Scroll rotational speed
- Bowl length
- Scroll pitch
- Bowl flotational speed
- Feed point of sludge
- Beach angle
- Feed point of chemicals
- Pool depth
- Condition of scroll blades

Many of these variables are preset by the manufacturer, although some can be adjusted by the operator. A typical solid-bowl centrifuge is pictured in Figure 4-11.

Pressure Filters

Filter presses for dewatering were first developed for industrial applications and, until development of diaphragm presses, were only slightly modified for municipal applications. The original models of the press were sometimes called plate and frame filters because they consisted of alternating frames and plates on which filter media rested or were secured. There are currently 20 U.S. installations of filter presses being used to dewater residuals.

The equipment commonly used to dewater WTP residuals is either the fixed-volume recessed plate filter or the diaphragm filter press. The diaphragm filter press was introduced within the last 10 years.

A recessed plate filter press consists of a series of plates, each with a recessed section that forms the void into which the solids are pumped for dewatering. Filter media are placed against each wall and retain the solids while allowing passage of the filtrate.

The surface under the filter media is specifically designed to ease the passage of the filtrate while holding the filter cloth. Solids are pumped with high-pressure pumps into the space between the two plates and individual pieces of filter media. The filtrate passes through the cake and filter media and out of the press through special ports on the filtrate side of the media.

The pumping of solids into the press continues up to a given pressure. When solids and water fill the space between the filter cloths and no further filtrate flow occurs, pumping is stopped. The press is then opened mechanically and the cake is removed.

The diaphragm filter press is a machine that combines the high pressure pumping of the recessed plate filter press with the capability of varying the volume of the press chamber. A flexible diaphragm is used to compress the cake held within the chamber. A two-step process is used, with the compression of the diaphragm taking place after the initial pumping stage. The release of water at low pressures helps maintain the integrity of the floc. After water release appears complete following the initial filling period, pumping is stopped and the diaphragm cycle is initiated. The diaphragm pressure is applied, using either air or water on the reverse side of the diaphragm. Pressures of up to 200 to 250 psi (1,380 to 1,730 kPa) are applied at this stage for additional dewatering.

When dewatering alum residuals, lime is added as a conditioning agent. Cake solids ranging from 30 to 60 percent can be achieved, depending on the source of the alum residual. Lime softening residuals do not need any conditioning and can be dewatered to 50 to 70 percent solids.

Precoat is generally not needed when inorganic conditioning chemicals, particularly lime, are used. Precoat is normally used in cases where particle size is extremely small, or when considerable variability in filterability and substantial loss of fine solids to and through the filter media are anticipated. When substantial quantities of lime are used, cloth washing may require both an acid and a water wash. A medium is needed, therefore, that is resistant to both acid and alkaline environments.

Vacuum Filters

Vacuum filtration was the most common means of mechanically dewatering WTP residuals until the mid-1970s. It has been used to some extent in the water treatment industry to dewater lime residuals. Alum solids have not been successfully dewatered on vacuum filters.

A vacuum filter consists of a horizontal cylindrical drum (Figure 4-12) which rotates while partially submerged in a vat of solids slurry. The filter drum is partitioned into several compartments or sections. Each compartment is connected to a rotary valve by a pipe. Bridge blocks in the valve divide the drum compartments into three zones, which are referred to as the cake formation zone, the cake drying zone, and the cake discharge zone. The filter drum is submerged to roughly 20 to 35 percent of

Figure 4-11. Solid-bowl-type centrifuge (Alfa-Laval Sharples, 1994).

its depth in a vat of solids; this submerged zone is the cake formation zone. Vacuum applied to this submerged zone causes the filtrate to pass through the media, retaining solids particles on the media. As the drum rotates, each section is successively carried through the cake formation zone to the cake drying zone. This latter zone begins when the filter drum emerges from the sludge vat. It represents from 40 to 60 percent of the drum surface and ends at the point where the internal vacuum is shut off. At this point, the sludge cake and drum section enter the cake discharge zone, where the sludge cake is removed from the media. Figure 4-12 illustrates the various operating zones encountered during a complete revolution of the drum.

No conditioning is required when dewatering lime residuals. As with all types of mechanical dewatering equipment, optimum performance depends on the type of solids and how the filter is operated. Selection of vacuum level, degree of drum submergence, types of media, and cycle times are all critical to optimum performance.

4.2.5 Drying

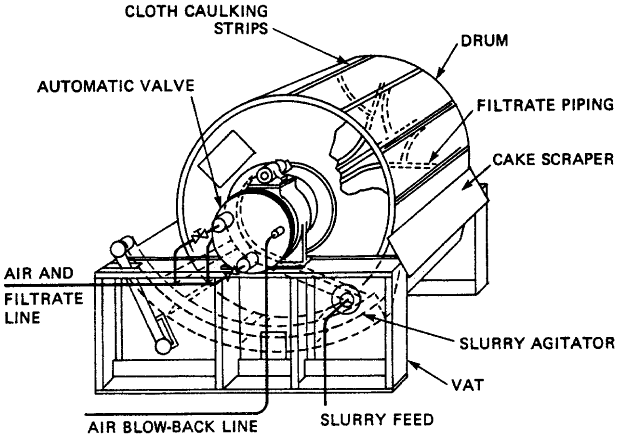
The drying of dewatered WTP residuals has historically revolved around the economics of reducing transportation and disposal costs by reducing solids volume and water content. Drying to solids concentrations greater than 35 percent is becoming a regulatory issue in many areas. For instance, the State of California requires that the solids concentration of a WTP waste be at least 50 percent before disposal in a landfill. Similar to the dewatering process, the drying process may be carried out through either open air means or through mechanical devices.

4.2.5.1 Open Air Drying

Any of the solar drying or lagoon procedures may be applied to the drying process. Drying depends on the evaporation mechanism. An extended drying process may require years to achieve the desired solids concentrations, although various innovations have been used with extended drying to accelerate the drying process. One such device uses specially mounted tractors to furrow and mix the solids to increase their exposure to sun and air.

4.2.5.2 Mechanical Drying

Of the mechanical drying techniques presented in the discussion of dewatering only, the filter press has shown the ability to consistently produce solids concentrations



CUTAWAY VIEW OF A DRUM OR SCRAPER-TYPE ROTARY VACUUM FILTER

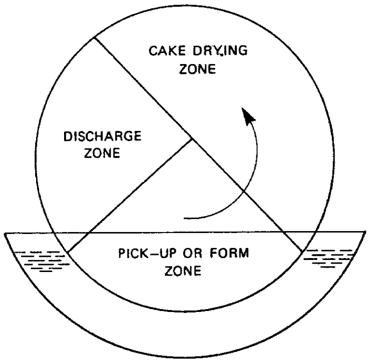


Figure 4-12. Vacuum filter (U.S. EPA, 1979b).

greater than 35 percent. The ability of this process to achieve solids concentrations greater than 50 percent, as would be necessary in California, is not proven.

Thermal drying of solids from WTPs has not been practiced on a full-scale operation basis in the United States. Cornwell and Koppers (1990) identify the potential for using steam-operated dryers to raise the solids concentration of a dewatered metal hydroxide sludge to the 65 to 75 percent range, but this process is untried in full-scale operation. Although many thermal processes have been used with wastewater solids, including the Best Process, the Carver Greenfield Process, and various forms of incineration, they are more applicable to a solid with a high organic content. No conclusive information is available to evaluate the effectiveness of mixing wastewater and water solids together before feeding into a thermal process.

4.2.6 Additional Residuals Handling Processes

4.2.6.1 Conveyance

Movement of a solids stream from one process to either another process or to a disposal point may be accomplished by gravity through a pipeline, pumping through a pipeline, transporting along a mechanical conveyor, or transporting by vehicle. The type of conveyance used depends on the form and concentration of the solids stream and on the transport distance.

4.2.6.2 Equalization Basins

Equalization basins even out the flow of waste streams. Equalization takes place to prevent surges of water from being reintroduced at the head of the treatment plant or at inlets to other residuals handling processes. Equalization basins can also be used to equalize peak daily, weekly, or monthly loads.

4.3 Residuals Handling Process Performance

4.3.1 Process Performance

A preliminary screening of processes may be based on the percent concentrations that the processes can achieve. This section provides typical ranges of values for percent solids that each thickening and dewatering process may generate; insufficient information exists for drying processes. Many factors can influence the performance of the process: the type of solids, characteristic of solids, solids concentration of the influent, climatological conditions for open air processes, variations in influent flow rates, and type and dosage of chemical conditioner. The values presented here should only be used for screening purposes. The following value ranges are used to suggest whether thickening, dewatering, or drying processes should be used to process metal hydroxide solids:

Desired Residual Metal Hydroxide Solids Concentration	Suggested Process
≤8%	Thickening
8–35%	Dewatering
>35%	Drying

These processes will generally yield a higher percent solids concentration for a lime sludge than for a metal hydroxide sludge. For example, a centrifuge may only dewater a metal hydroxide sludge to a 15 percent solids concentration, while the lime sludge is dewatered to 50 percent. When selecting a handling process for a lime softening WTP, the following concentrations apply:

Desired Residual Lime Solids Concentration	Suggested Process
≤30%	Thickening
30–60%	Dewatering
>60%	Drying

4.3.2 Comparison of Thickening Processes

A comparison of the various thickening processes, including solids loading on the units and the solids concentration of various types of residuals, is shown in Table 4-2.

4.3.3 Comparison of Dewatering Processes

A comparison of dewatering process performance is shown in Table 4-3.

4.4 Developing Preliminary Residuals Processing Alternatives

Many unit process combinations are capable of meeting any set of residuals processing requirements. Most facilities need to consider the addition of residuals processing, however, because of increased regulatory requirements, changes in finished water-quality goals, increased external disposal costs, and community relations issues. Two technical factors are used to determine the residuals processing requirements for a utility. The most definitive is the solids concentration of the residual flow stream. The second factor is the solids concentration required for different types of residuals disposal strategies.

Table 4-2. Comparison of Thickening Processes (Cornwell et al., 1987; Cornwell and Koppers, 1990; Brown and Caldwell, 1990)

Process	Residual	Solids Loading	Solids Concentration (%)
Gravity	Carbonate	30 lb/day/ft^2	15–30
Gravity	Hydroxide	4.0 lb/day/ft^2	1–3
Flotation	Hydroxide	20 lb/day/ft^2	2–4
Gravity belt	Hydroxide	N/A[a]	2.5–4.5

[a] No solids loading rate is shown for the gravity belt thickener as it is not comparable to the values for the gravity thickener and the DAF unit. Care must be taken in the use of solids loading and percent solids values for both the flotation and gravity belt thickening, due to the absence of operating experience for those processes.

Table 4-3. Comparison of Dewatering Processes (Cornwell et al., 1987)

	Solids Concentration (%)	
Process	Lime Sludge	Coagulant Sludge
Gravity thickening	15–30	3–4
Scroll centrifuge	55–65	20–30
Belt filter press	10–15	20–25
Vacuum filter	45–65	25–35
Pressure filter	55–70	35–45
Diaphragm filter press	N/A	30–40
Sand drying beds	50	20–25
Storage lagoons	50–60	7–15

A water utility must identify preliminary combinations of unit processes that can achieve generic residual processing requirements according to a limited set of selection parameters. The approach needed to achieve the goal includes these steps:

1. Define the fundamental information needed, attempting to identify a preliminary residual processing alternative.
2. Define a methodology of using the fundamental information to make a preliminary selection of a residuals processing alternative that has the potential of achieving the desired needs.
3. Present a Preliminary Residuals Processing Selection Matrix that can be used with other fundamental information to make a preliminary selection.

4.4.1 Residuals Processing Requirements

To identify the ideal combination of unit processes for a preliminary residuals processing alternative, the specific requirements of the residuals processing system must first be defined. To do this, information must be obtained on residuals disposal limitations, quantity and quality of residuals sources, resource recovery potential, and residuals mass balance.

4.4.1.1 Residuals Disposal Limitations

Often, the ultimate method of residuals disposal determines the limitations, which in turn define the process design requirements. The percent solids content of a residual is the primary criterion used to define the acceptable limits of a disposal option. Of course, landfill and land application disposal options can be accomplished over a wide range of solids content, but both disposal options have different equipment requirements at the low and high solids limits of their ranges. The following tasks will help establish the disposal limitations that a water utility must consider for further evaluation of residual process alternatives:

- *Identify available residuals disposal alternatives:* The six most common methods of WTP residuals disposal used in the water industry are:
  - Land application
  - Landfilling (monofill)

- Direct stream discharge
  - Landfilling (co-disposal)
  - Discharge to sewer
  - Residual reuse

- *Identify disposal limitations for each alternative:* The normal range of acceptable residual solids content for the six common methods of disposal are:
  - Land application, <1% to 15% solids
  - Landfilling (co-disposal), <15 to 25% solids
  - Landfilling (monofill), >25% solids
  - Discharge to sewer, <1% to 8% solids
  - Direct stream discharge, <1% to 8% solids (Note: Allowable solids concentration for this disposal method may be lower than 8 percent due to regulatory requirements or sewerage concerns such as deposition.)
  - Residual reuse, <1% to >25% solids

- *Identify external disposal costs, if they apply:* Disposal in a co-disposal municipal landfill may involve a per unit weight tipping fee levied by an agency external to the water utility. A user charge can be assessed for discharges to a sewer, based on flow rate units and/or the combination flow rate and suspended solids concentrations. In some instances of residuals reuse, the water utility pays a fee to the user to cover the additional costs of special handling or the difference in production costs associated with accommodating sludge over a more traditional material.

4.4.1.2 Quantity and Quality of Residuals Sources

For many water utilities, monitoring the quantity and quality of residuals sources either has not been done, or has not achieved the same level of accuracy or frequency as has monitoring of the finished water. This deficiency may require that additional effort and time be spent to collect the following information:

- *Residuals sources:* The typical residuals sources associated with conventional WTPs are:
  - Headworks/bar screens
  - Presedimentation basins
  - Filter backwash
  - Grit basins
  - Sedimentation basins
  - Filter-to-waste

- *Flow rates and quantities of residuals sources:* Flow rate data collected for each residuals source should include the following:
  - Maximum flow rate
  - Average flow rate
  - Maximum volume per release event
  - Frequency of events
  - Maximum number of events per day
  - Potential for concurrent events
  - Minimum flow rate
  - Average volume of release
  - Minimum volume per event
  - Average event duration

- *Quality of each residuals source:* The primary quality parameter to monitor is percent solids which, in analytical terms, represents total suspended solids. Other parameters that should be analyzed on a limited basis are:
  - Aluminum and/or iron
  - Total dissolved solids
  - Trihalomethane formation potential
  - Trace metals
  - Toxicity Characteristic Leachate Procedure (TCLP)
  - pH
  - Total organic carbon

4.4.1.3 Resource Recovery Potential

Although standard water conservation strategies support the recovery of reusable residuals, the value of recycle water is being challenged by the potential risks of recycling infectious organisms, heavy metals, manganese, and DBPs. Typical recycling of untreated residuals streams (e.g., backwash and filter-to-waste residuals) to the plant influent may result in the accumulation of these contaminants in the treatment processes. Little documentation as to the extent of this is currently available. Because of these concerns, the value of water continues to increase with time. The recovery potential for reusable residuals streams should be evaluated using the following procedure:

- *Assess the value of recovered water:* Essentially, the value of recovered water depends on the value of raw water and the relative abundance of supply. Establish whether the cost of recovery is justified.

- *Identify residuals sources easily recovered:* The backwash and filter-to-waste residuals streams generally comprise the largest volume of residuals generated with the lowest percent solids concentration. Treatment of these residual streams is usually justified for the value of the water recovered. Other residuals streams need to be evaluated closely for their potential effect on finished water quality.

- *Identify residuals sources unsuitable for recovery:* The most difficult residuals streams to justify are the sidestreams of the mechanical dewatering processes, which usually contain many components that are not desirable to return to the plant.

4.4.1.4 Residuals Mass Balance

The most valuable exercise in evaluating the residuals generated at a WTP is the development of a mass balance. A mass balance is a theoretical accounting of the water and solids content that come into a plant; solids added to the water; and the water and solids content that come out of the plant in the finished water and residuals streams. This assessment must include the effects that normal variations in raw water quality and associated chemical feed rates have on residuals production rates. The objective of the mass balance is to specifically define the following parameters, which become more useful in further evaluations:

- Maximum, minimum, and average solids production rates for the raw water and each residuals side stream in terms of percent solids, flow rates, and total solids produced per million gallons of water treated.
- Allowable return residuals stream limits in terms of hydraulic and percent solids limit (e.g., maximum backwash return to plant influent should not exceed 5 percent of operating production capacity).
- Allowable solids content in the finished water.
- Allowable flow rate and solids content for available residuals disposal options.
- Required residuals processing efficiency for each residual stream (based on the allowable return limits, effluent limits, and disposal limits for each residual stream).

The product of this exercise is the identification of potential solids content objectives for each residual stream process.

4.4.2 Preliminary Selection of Residuals Processing Alternatives

The next step in developing a preliminary residuals processing alternative is to select a unit process or combination of unit processes that can fulfill the defined processing requirements. There are four general types of criteria on which the selection process is based. These are:

- Specific problem-based criteria
- Residuals disposal-based criteria
- Solids concentration-based criteria
- Other selection criteria

In the event that the problem-specific approach does not apply, the secondary level of selective review addressed the available disposal methodology. The third selective review focuses on the residuals solids concentration categories.

The Preliminary Residuals Processing Selection Matrix shown in Table 4-4 provides a finite set of unit process combinations categorized according to three ranges of residual solids concentrations that they would be used to treat.

The rules of selection generally follow the basic premise that most water utilities will want to minimize the capital and labor costs of residuals processing. The preliminary selection should attempt to find the lowest solids content, operationally simple, single-unit process that can fulfill the requirements. The preferred alternatives, from best to worst, are:

- Low solids; simple operations; single unit process.
- Low solids; simple operations; combined unit process.
- Low solids; complex operations; single unit process.
- Low solids; complex operations; combined unit process.
- Medium solids; simple operations; single unit process.
- Medium solids; simple operations; combined unit process.
- Medium solids; complex operations; single unit process.
- Medium solids; complex operations; combined unit process.
- High solids; simple operations; single unit process.
- High solids; simple operations; combined unit process.
- High solids; complex operations; single unit process.
- High solids; complex operations; combined unit process.

The following approach can be used to select a preliminary residuals processing alternative that can be evaluated in detail. Site-specific factors such as land availability, climate, community, and environmental concerns will play a significant role in the selection process.

4.4.2.1 Specific Problem-Based Selection

At the first level of selective review a WTP operation must consider the following residuals processing issues:

- *Regulatory related issues:* These issues result from the ever-changing regulatory environment. Typical concerns are land application and landfill disposal regulations.
- *Finished water-quality issues:* With the recent publication of the draft Disinfection/Disinfection Byproduct Rule (D/DBP), the most prevalent concern facing the water industry is how to deal with the residuals byproducts of enhanced coagulation and the increased

Table 4-4. Preliminary Residuals Processing Selection Matrix

		Problem-Specific Alternative Categories				Single Unit Process Alternatives[b]		Combined Unit Process Alternatives[a]	
Residual Solids Concentration Categories	**Residuals Disposal Alternative Categories**	**Regulatory Related Issues**	**Finished Water Quality Issues**	**Disposal Economic Issues**	**Community Relation Issues**	**Simple Operations**	**Complex Operations**	**Simple Operations**	**Complex Operations**
Low solids content (<1 to 8%) by dry weight	Direct discharge • To source water • To water of USA • Intermittent stream Sewer disposal Land application • Agricultural • Forest land • Disposal site Residual reuse • Landscape tree nursery • Irrigation of turf grass	NPDES permit for direct discharge Water supplier policy change on return water quality (i.e., “zero discharge”) Residual disposal regulations (if applicable) Aquifer protection requirements	Increased sludge production due to enhanced coagulation (EC) Increased volume of residuals due to implementation of “filer-to-waste” Concern for increased arsenic concentrations associated with EC arsenic removal Increased concern with recycling of infectious organisms in the recovered water	Change in sewer service charges for sludge disposal Restrictions on solids content in sludge for sewer disposal Land application disposal requires nutrient content augmentation Loss of market for reuse product	“Not in my back yard” syndrome (NIMBY) Potential odors Concern with traffic of transport vehicles Concern for risk of incidental exposure Concern for aerial dispersion of biological contaminants General environmental concerns of public	Direct discharge from process basin (DDPB) Lagoon (LAG) Sand drying bed (SDB) Solar drying basin with decant (DB/D) Wedge wire filter bed (WFB)	Gravity thickener (GT)[h] Dissolved air flotation thickener (DAF)[h] Mechanical thickening (MT)[d,h] Vacuum assisted plate filter bed (VAFB)[g,h] Backwash water sedimentation basin (BWSB)[c]	DDPB + residuals pump station (PS) PS + LAG + decant pump station (DPS) PS + DB/D + DPS PS + WFB + DPS PS + SDB + DPS Backwash recovery pump station (BRPS)[c] Equalization basin and pump station (EB/PS)[c]	PS + GT[h] PS + DAF[h] PS + MT[d,h] PS + VAFB[g,h] BRPS + BWSB[c] EB/PS + GT[c,h]
Medium solids content (<8 to 25%)	Land application • Disposal • Agricultural • Forest Residual reuse • Sod amendment for turf farms • Landscape tree nursery • Coagulant recovery Landfill • Co-disposal with municipal solid waste (low moisture content refuse)	Residual disposal regulation (if applicable) Residual reuse regulations on agricultural land Landfill operation requirements Disposal of hazardous waste byproducts from coagulant recovery	Increased sludge production due to EC Potential for hazardous waste classification with arsenic Organics, color, and heavy metal concentrations in recovered coagulant Concern with recycling of infectious organisms in recovered water	Limit on minimum solids content Increase in tipping fee per ton at landfill Loss of reuse product market Future refusal to accept sludge at landfill Land application may require nutrient augmentation	NIMBY Potential odors Noise pollution with sludge handling equipment Traffic concerns Incident exposure risk Environmental concerns	LAG SDB DB/D WFB	MT[d] VAFB[g] Mechanical Dewatering (MD)[e]	PS + LAG + DPS PS + SDB + DPS PS + DB/D + DPS PS + WFB + DPS BRPS + LAG + DPSc	PS + GT + MT[d,h] PS + DAF + MT[d,h] PS + VAFB[g,h] PS + GT + MD[e,h] PS + DAF + MD[e,h] PS + MT[d] PS + MD[e] PS + LAG + DPS + MT[d] PS + LAG + DPS + MD[e] EB/PS + GT + MD[c,e,h] BRPS + LAG + DPS + MT[c,d]

Table 4-4. Preliminary Residuals Processing Selection Matrix (Continued)

		Problem-Specific Alternative Categories				Single Unit Process Alternatives[b]		Combined Unit Process Alternatives[a]	
Residual Solids Concentration Categories	**Residuals Disposal Alternative Categories**	**Regulatory Related Issues**	**Finished Water Quality Issues**	**Disposal Economic Issues**	**Community Relation Issues**	**Simple Operations**	**Complex Operations**	**Simple Operations**	**Complex Operations**
High solids content (>25%)	Landfill • Monofill • Use as landfill cover extender Residual reuse • Coagulant recovery • Integrated into masonry products • Filler in Portland cement	Landfill design requirements Landfill operation requirements Disposal of hazardous waste byproduct from coagulant recovery Residual reuse requirements (if any)	Increased sludge production due to EC Need to segregate unrecoverable sidestreams of high solids unit processes Potential hazardous classification of sludge due to arsenic and other metal concentrations	Future refusal to accept sludge at landfill as cover extender Refusal to accept future sludge in landfill Loss of reuse product market	NIMBY Traffic concerns Noise pollution from dewatering equipment Long-term environmental concerns	LAG + extended drying activities (ED) SDB + ED DB/D + ED WFB + ED	VAFP + ED[h] MD[e]	PS + LAG + DPS + ED PS + SDB + DPS + ED PS + DB/D + DPS + ED PS + WFB + DPS + ED PS + LAG + DPS + extended drying area (EDA) PS + SDB + DPS + EDA PS + DB/D + DPS + EDA PS + WFB + DPS + EDA BRPS + LAG + DPS + EDA	PS + LAG + DPS + MT + EDA[d] PS + LAG + DPS + MD[e] PS + GT + DPS + MT + EDA[d] PS + GT + DPS + MD[e] PS + DAF + DPS + MT + EDA[d] PS + DAF + DPS + MD[e] PS + MT + MD[d,e] PS + MD + EDA[e] EB/PS + GT + MD[c,e,h] EB/PS + DAF + MD[c,e,h] BRPS + LAG + DPS + MD[c,e] BRPS + LAG + DPS + MT + EDA[c,d]

---

[a] Assumes residuals collection systems exist in process basins with gravity flow to residual processing and excludes sludge storage, transport, and disposal related equipment.
[b] Assumes residuals collection systems exist in process basins with gravity flow to residuals pump station and excludes sludge storage, transport, and disposal equipment.
[c] These alternatives are additional unit processes necessary to accommodate backwash flow rates.
[d] Mechanical thickening includes gravity belt thickeners, centrifuges, etc.
[e] Mechanical dewatering includes belt filter presses, centrifuges, filter presses, etc., plus cake handling equipment.
[f] This combination offers good potential for optimum recovery of used water.
[g] This combination is commonly used for coagulant recovery.
[h] These unit processes include thickened sludge pump stations.

Key

BRPS = Backwash recovery pump station
BWSB = Backwash water sedimentation basin
DAF = Dissolved air flotation
DB/D = Solar drying basin with decant
DDPB = Direct discharge from process basin
DPS = Decant pump station
EB/PS = Equalization basin and pump station
EC = Enhanced coagulation
ED = Extended drying
EDA = Extended drying area
GT = Gravity thickening
LAG = Lagoon
MD = Mechanical dewatering
MT = Mechanical thickening
PS = Residuals pump station
SDB = Sand drying bed
VAFB = Vacuum-assisted plate filter bed
WFB = Wedge wire filter bed

volume of residuals that will be processed and disposed of. This category also addresses concerns associated with returning untreated residuals to the treatment process.

- *Disposal economics issues:* These issues require existing residuals processing practices to be revisited in response to cost increases for disposal activities that are interutility and external to the water utility.
- *Community relations issues.*

Table 4-4 provides a brief description of these issues and identifies residuals disposal alternatives associated with low-, medium-, and high-solids content residuals.

To use the matrix for the selective review process, a utility should first identify the Residuals Solids Concentration category (low, medium, high) that applies to its current capabilities. Next, the utility should identify, within its concentration category, a specific problem that is similar to one currently facing the utility. After these two steps, a preliminary selection can be made from the four right-hand columns, using the priority protocol listed in the previous section. Ideally, the selection made will use the maximum number of existing unit processes. This approach can be rationalized by the obvious economic benefit of reusing existing facilities in a process group to meet the processing requirements.

4.4.2.2 Residuals Disposal-Based Selection

If a relevant problem cannot be identified on Table 4-4, the utility should select a preliminary alternative based on the available solids disposal option with the highest probability of being implemented. Selection of the preliminary alternative follows the same procedure described in the previous section, using the Residuals Disposal Alternatives column instead of the Problem-Specific one. The scope of each disposal alternative is outlined below:

- *Landfilling:* The four most common landfill disposal techniques for WTP residuals are:
  - Co-disposal with municipal solid waste (refuse)
  - Use as daily landfill cover extender
  - Monofill of WTP residuals only
  - Co-disposal with WWTP biosolids
- *Land application:* The three most common land application techniques for WTP residuals are:
  - Land application on agricultural land
  - Land application on forest land
  - Land application on a designated disposal site
- *Sewer disposal:* This alternative assumes that the sewer ends at an existing WWTP.
- *Direct discharge to source stream:* Several configurations of direct discharge are possible:
  - Discharge to waters of the United States.
  - Discharge to a water district supply canal.
  - Discharge to an intermittent stream bed.
  - Discharge to a dry arroyo (a watercourse in the arid southwest that only typically flows during, or directly after, a storm event).
- *Residuals reuse:* Although WTP residuals do not have the inherent fertilizer value of WWTP residuals, these reuse alternatives are being practiced:
  - Recovery of coagulants
  - Recovery of lime at softening plants
  - Use in landscape tree nursery (low solids)
  - Use in making bricks (high solids)
  - Use in Portland cement (high solids)
  - Use in turf farming

4.4.2.3 Solids Concentration-Based Selection

The most basic level of review is to identify preliminary processes that can produce a specific range of solids concentrations. Three possible categories of solids concentration are:

- *Low:* Low processes can produce solids concentrations ranging from less than 1 percent to 8 percent solids, by weight. This group represents the general limits of the thickening processes, specifically the common gravity thickener.
- *Medium:* Medium processes can produce solids concentrations ranging from greater than 8 percent to 25 percent solids, by weight. This group represents the general limits of the simple dewatering processes, such as sand drying beds, and general mechanical dewatering with belt filter presses and general service centrifuges.
- *High:* High processes can produce solids concentrations of greater than 25 percent solids, by weight. These are the general limits of the high solids dewatering processes, such as filter presses and high-solids centrifuges. These limits are also associated with the minimum solids content for a monofill, although 50 percent solids is preferable.

4.4.2.4 Other Selection Criteria

The matrix also identifies unit processes and combined unit processes used to accomplish optimal water resource recovery alternatives and coagulant recovery alternatives.

4.5 Specific Residuals Unit Process Selection Criteria

Selection of residuals handling processes is a more complicated task than selection of traditional drinking water treatment processes for the following reasons:

- Little operating experience with residuals handling processes exists for use as a basis of comparison.
- Residuals handling processes are more difficult to test with a procedure such as a jar test or a pilot filter because of the interdependence of solids handling on other unit processes (e.g., coagulation, sedimentation, and thickening).

Five selection criteria may be used in screening and selecting residuals handling processes:

- Discharge limitations and the effective operating range of the residuals handling process.
- Similar operating experience with unit process.
- Bench- and pilot-scale testing of unit process.
- Construction, operation, and maintenance costs.
- Environmental impact of unit process.

Each of these selection criteria is described in more detail in this chapter. The flowchart in Figure 4-13 shows how the five selection criteria (in boxes) are used to identify a desired process alternative. No attempt is made to weigh the relative impacts of these criteria, which vary from facility to facility.

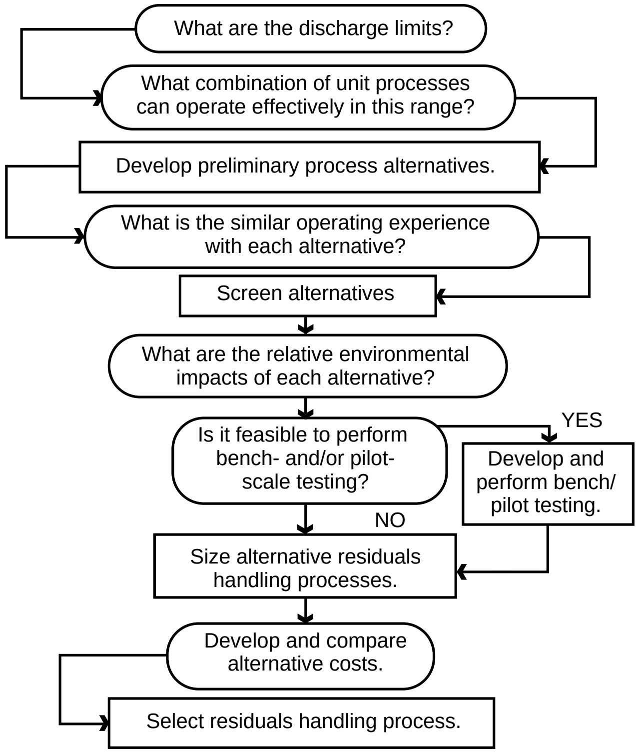


Figure 4-13. Residuals handling process selection flow chart.

4.5.1 Operating Experience

The effectiveness of residuals handling processes are very specific to the characteristics of the raw and processed water and, in some cases, to environmental conditions. Bench-, pilot-, and full-scale tests provide information needed to select processes. Circumstances may dictate that no testing or very little testing can be performed by a utility; in this case, the experience of other utilities with various residuals handling processes may be the best source of information. The Water Industry Data Base (WIDB), developed by the American Water Works Research Foundation (AWWARF) and the American Water Works Association (AWWA), makes this information available (Kawczyinski and Achterrman, 1991).

This survey covered 438 utilities and 347 WTPs. Treatment methods and disposal practices were categorized for each facility. The different thickening and watering categories are summarized in Tables 4-5 and 4-6. Note that, since the database was published in 1991, a series of new facilities have started operation using processes such as vacuum-assisted drying beds. Consult with equipment manufacturers for more recent operating histories.

Table 4-5. Survey of Thickening Methods at Water Treatment Plants in the United States (Kawczyinski and Achterrman, 1991)

Treatment Methods	Existing Number of Facilities
Lagoons	180
Gravity thickening	48
Dissolved air flotation	0
Gravity belt thickeners	0

Table 4-6. Survey of Dewatering Methods at Water Treatment Plants in the United States (Kawczyinski and Achterrman, 1991)

Treatment Methods	Existing Number of Facilities
Lagoons	180
Sand drying beds	26
Freeze assisted drying beds	33
Solar drying beds	Number included with sand drying beds
Belt filter press	Number included with filter presses
Centrifuges	10
Filter presses	20
Vacuum filters	4
Screw press	0

4.5.2 Bench-Scale and Pilot-Scale Tests

4.5.2.1 General

Bench- and pilot-scale tests serve two functions in the selection of residuals handling processes. First, they provide additional data for the sizing of the full-scale equipment. Second, they provide an indication of the process performance (i.e., the solids concentration that can be achieved).

Figure 4-14 is a guide to the use of bench- and pilot-scale tests for this application. Bench- and/or pilot-scale testing should be considered after the alternative residual handling schemes have been screened and narrowed down to a few options. Pilot- and bench-scale testing may not be needed if there is similar experience with the process at a comparable facility, defined as one of similar size with approximately the same raw water quality and unit processes. Short implementation schedules and limited study budgets may curtail bench- and pilot-scale testing, but the lack of related experience can make bench- and pilot-scale testing desirable.

4.5.2.2 Bench-Scale Testing

These bench-scale tests are generally used for thickening and dewatering processes (Dentel et al., 1993):

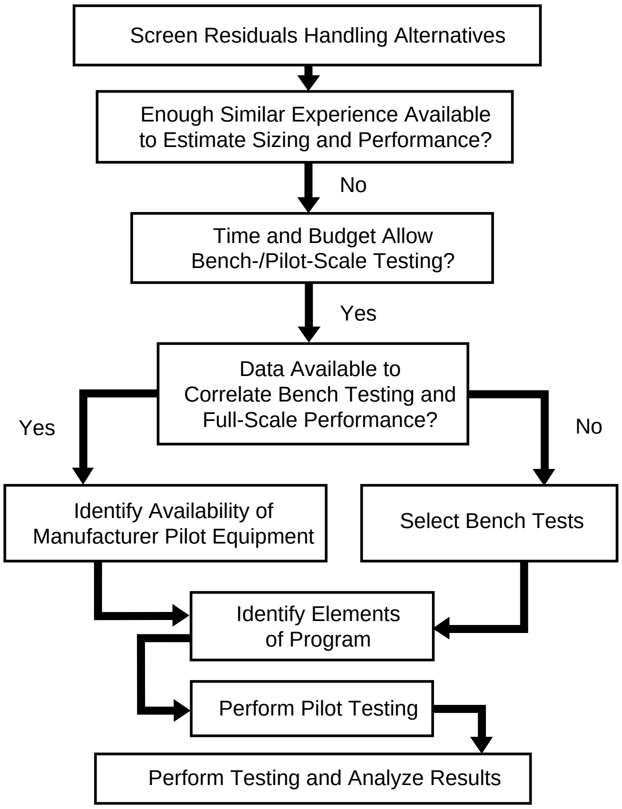


Figure 4-14. Bench/pilot testing decision flow (Dentel et al., 1993).

- *For thickening:* settling tests, flotation test, and capillary suction time test.
- *For dewatering:* time-to-filter test, filter leaf test, capillary suction time test, and settling tests.

Detailed lists of equipment and test procedures for each of these tests are available (Cornwell et al., 1987; Dentel et al., 1993). The performance of a bench-scale test does not have any direct correlation to the performance of a given process. Bench-scale tests typically simulate full-scale operations. Thus, a correlation may be developed, for instance, between the results of capillary suction time tests and the performance of centrifuges. The application of the bench-scale test is then to analyze the effect of different sludge conditioners. Bench-scale tests may have application to the selection of a residuals handling process if the bench-scale tests can be correlated to process performance based on previous testing. Equipment suppliers should be asked about the availability of past bench-test comparisons. Suppliers, often times, can bench test units at their own facilities.

4.5.2.3 Pilot-Scale Testing

Pilot-scale testing involves the use of test equipment similar enough in size to full-scale operating equipment that the test results can be directly compared between pilot-scale test and full-scale operation. This testing may involve the construction of a scaled-down version of a process basin such as a gravity thickener or a sand drying bed. Suppliers of mechanical thickening and dewatering equipment typically have test units available for rent or loan, and can test either on site or at the supplier’s facilities.

4.5.2.4 Test Procedures and Analysis

The use of bench- and pilot-scale tests has many pitfalls. The test must be as representative as possible of the real world conditions, and must simulate all of the processes involved with the alternative in question. This is particularly a concern with thickening and dewatering, where the incoming solids must represent the solids that will actually be concentrated and transported from the previous process. Even when the bench- or pilot-scale testing successfully models the performance of a process, the results may not reveal operational problems that are part of the full-scale process.

The duration of testing is a particular concern. Year-round operation may involve significant changes in raw water quality that can affect the operation of the residual handling process. One week of pilot-scale testing may not reflect this impact, and year-round pilot-scale testing is usually not feasible. The use of short-term pilot-scale testing to correlate the results with bench-scale tests may come close to approximating the results of long-term testing, though on a lesser scale.

When analyzing the results, the limitations of the test setup and test protocol should be considered. If limited testing is done, less emphasis should be placed on the results.

4.5.3 Environmental Impacts

Environmental impacts have become a standard consideration for public facilities, and are typically part of this selection process. Environmental criteria, however, are subjective, making them difficult to use as a basis for comparison. The environmental impacts of a WTP residuals handling process can vary greatly, depending on whether one's viewpoint is that of an operator, a neighbor, or even a migrating bird! Some relevant environmental considerations for this selection process are:

- *Effectiveness in meeting discharge requirements:* The ultimate criterion for the selection of a process or a combination of processes is whether the process will reliably and consistently meet the regulatory discharge requirements. This depends on many factors, including the size of the application and process loading conditions.
- *Ground-water quality:* Any process that might release contaminants that could migrate to the ground water should be approached with caution. Unlined lagoons and drying beds are typically of greatest concern.
- *Noise:* Of concern to both plant employees and site neighbors, noise is typically associated with thickening and dewatering processes that use mechanical equipment. Open-air processes may also involve noise from front-end loaders or other devices used to move and remove solids. Offsite transport of solids is another solids concern. Mitigation techniques include hearing safety equipment for workers, acoustically sealed buildings for mechanical devices, and specified operating hours for front-end loaders and other vehicles.
- *Odors:* Many operators emphatically say that no odors are associated with WTP residuals and, frequently, they are correct. With open-air processes (and sometimes even mechanical systems), however, odors are a possible consequence. The traditional technique used to mitigate odors is an operational change in the loading and removal of solids from process units. Chemicals such as caustic and chlorine may be needed to stabilize and destroy odor causing organisms in the sludge. Odor control units may also be required for systems installed indoors.
- *Energy use:* Energy use varies with both equipment usage and the cost of removing solids from the site. Mechanical dewatering techniques require the highest energy consumption, although consumption varies widely between the different processes. Different applications of processes can also affect the percent solids concentration of the residual before disposal. A higher concentration translates to a lower volume for disposal. This lower volume then results in the use of less transport energy.
- *Insects and other pests:* Insects are a potential concern with open-air processes. Submerged lagoons or gravity thickeners may serve as breeding grounds for mosquitoes and other flying insects. Flies and gnats may be a concern when drying residuals in the open air.
- *Impact on the neighborhood:* This broad category encompasses many environmental concerns that can result in the lowering of local property values.
- *Air pollution:* A typical concern when evaluating wastewater processes, air pollution (stripping volatile contaminants into the air) is a small concern with water treatment systems. Some concerns may arise from the potential use of mechanical equipment having gas or diesel engines.
- *Space requirement:* Many plant sites do not have a large property area or the ability to expand onto adjacent vacant land. Limited space typically drives process selection away from open-air processes and toward mechanical processes.
- *Employee and public safety:* Mechanical handling processes typically involve the highest safety concerns because they use heavy equipment with the potential to cause serious accidents. All processes involve some potential hazard, however, ranging from suffering heat stroke while working in a solar drying bed to drowning in a thickener.

In comparing environmental criteria, the interrelationship of these criteria with both construction and operation and maintenance costs is noteworthy. For example, the ground-water quality concerns associated with lagoons can be resolved by the installation of a relatively impermeable liner beneath the lagoons; the liner mitigates the environmental concern at a significant increase in construction cost. The relationship between environmental controls and operation and maintenance costs should be considered in the analysis of different residuals handling approaches.

4.6 Final Screening of Residuals Handling Processes

4.6.1 Additional Selection Criteria

Previous operating experience, environmental impacts, and bench- and pilot-scale testing are criteria common to most residual handling process selections. Additional criteria applicable to both wastewater processes and water treatment process are (U.S. EPA, 1979b):

- *Compatibility:* Compatibility with existing land-use plans; areawide, solid waste, water, and air pollution controls; and with existing treatment facilities.
- *Implementability:* Available sites; available personnel (including specially skilled labor); sufficient or expandable existing utilities; meets regulatory review requirements; can be achieved within the required schedule; whether existing processes can be interrupted for construction connection.
- *Flexibility:* Ability to respond to new technology, changing regulations; changing loads; incremental expansion.
- *Reliability:* Vulnerability to disasters; probable failure rate; backup requirements; operator attention required.
- *Impact of residual handling process sidestreams on existing process.*

4.6.2 Process Alternative Selection Matrix

Process combinations that achieve different degrees of residuals solids concentrations are shown in Table 4-4. The final selection of the process combination depends on the relative importance of the selection criteria. One approach uses a weighting table to aid a decision-maker in analyzing different process combinations in terms of weighted criteria (U.S. EPA, 1979b). An example of this approach is presented in Table 4-7. The weights presented in this table are examples only; each facility must assign the appropriate relative weights according to its own priorities.

4.7 Residuals Handling Process Design Issues

4.7.1 Mass Balance Diagrams

4.7.1.1 Estimating Solids Production

Solids mass balances are used to develop estimates of hydraulic and solids loading on residuals handling unit processes. Solids production from a WTP may be estimated based on the raw water suspended solids removal and the quantity of process chemical added. Refer to Chapter 3 for equations used to estimate solids product.

When estimating solids production for the preliminary design of a facility, the following should be considered:

- Historical trends of raw water suspended solids loading must be analyzed to determine the average day, maximum day, and peak hour loadings. These values are typically used in evaluating the following design issues:
  - Average day: Used to analyze annual solids disposal fees and land availability for open-air dewatering and drying processes.
  - Maximum day: Used to size thickening, dewatering, and drying processes.
  - Peak hour: Used to size piping and pumping. If equipment such as centrifuges or belt filter presses are sized based on maximum day loadings, then the residual handling systems must be designed with the capacity to absorb the difference between the peak hour and the maximum day loadings. This may be done by incorporating equalization basins into the design. Sedimentation basins and thickeners can also equalize peak hourly, daily, and weekly flows. An alternate approach is to consider the ability of a given process to handle short-term spikes in the solids loading rate.
- Most water treatment facilities record solids loadings in terms of turbidity (NTU) rather than suspended solids. Methods of converting turbidity values to suspended solids values are available (Cornwell et al., 1987). Generally, the ratio of suspended solids to NTU is 1 to 2.
- Estimating raw water suspended solids loadings is inherently more difficult for new WTPs than is adding residuals handling facilities to existing WTPs. The options are:
  - To use the historical records of a WTP that uses similar processes with a comparable water source.
  - To develop a water-quality database through a program of sampling, jar testing, filtration, and analysis. Because a short-term program will represent conditions only for one period in time, such a program may not reflect long-term water quality at the site.

4.7.1.2 Developing the Diagram

Examples of a mass balance diagram and calculation are shown in Figures 4-15A and 4-15B, respectively. The percent of solids captured for the gravity thickeners and the filter presses are based on the typical values in Tables 4-2 and 4-3. These values should be verified in a bench- or pilot-scale testing program. The diagram can also provide valuable data regarding the quantity of chemical conditioner—polymer, in this case—which is required in the process.

4.7.2 Equipment Sizing

WTP residuals handling systems can vary from simple sewer/stream disposal systems to elaborate thickening/dewatering/drying complexes. The mass balance examples in Figures 4-15A and 4-15B are only the starting point; the next step is the generation of a schematic such as that shown in Figure 4-16, which is based

Table 4-7. Example of Weighting System for Alternative Analysis (U.S. EPA, 1979b)

		Alternative 1		Alternative 2					Alternative n	
Categories and Criteria	**Relative Weight[a]**	**AR[b]**	**WR[c]**	**AR**	**WR**	**.**	**.**	**.**	**AR**	**WR**
Effectiveness:										
Flexibility	3	4	12	6	18	.	.	.	6	18
Reliability	5	3	15	5	25	.	.	.	2	10
Sidestream effects	3	10	30	9	27	.	.	.	7	21
Compatibility:										
With existing land use plans	2	8	16	8	16	.	.	.	4	8
With areawide wastewater, solid waste, and air pollution programs	3	3	9	6	18	.	.	.	7	21
With existing treatment facilities	4	5	20	5	20	.	.	.	3	12
Economic impacts: Construction cost	4	7	28	8	32	.	.	.	7	28
O&M cost	1	8	8	9	9	.	.	.	8	8
Total weighted alternative rating[d]	-	-	1,576	-	1,430	-	-	-	-	1,317

[a] Relative importance of criteria as perceived by reviewer; scale, 0 to 5; no importance rated zero, most important rated 5.

[b] Alternative rating. Rates the alternative according to their anticipated performance with respect to the various criteria; scale 1 to 10; least favorable rated 10.

[c] Weighted rating. Relative weight for each criteria multiplied by alternative rating.

[d] Sum of weighted ratings for each alternative.

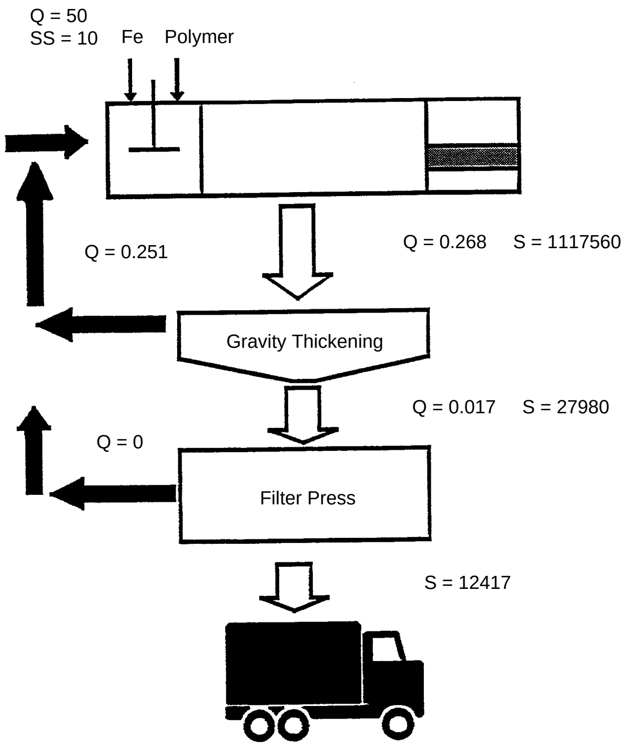


Figure 4-15A. Mass balance schematic.

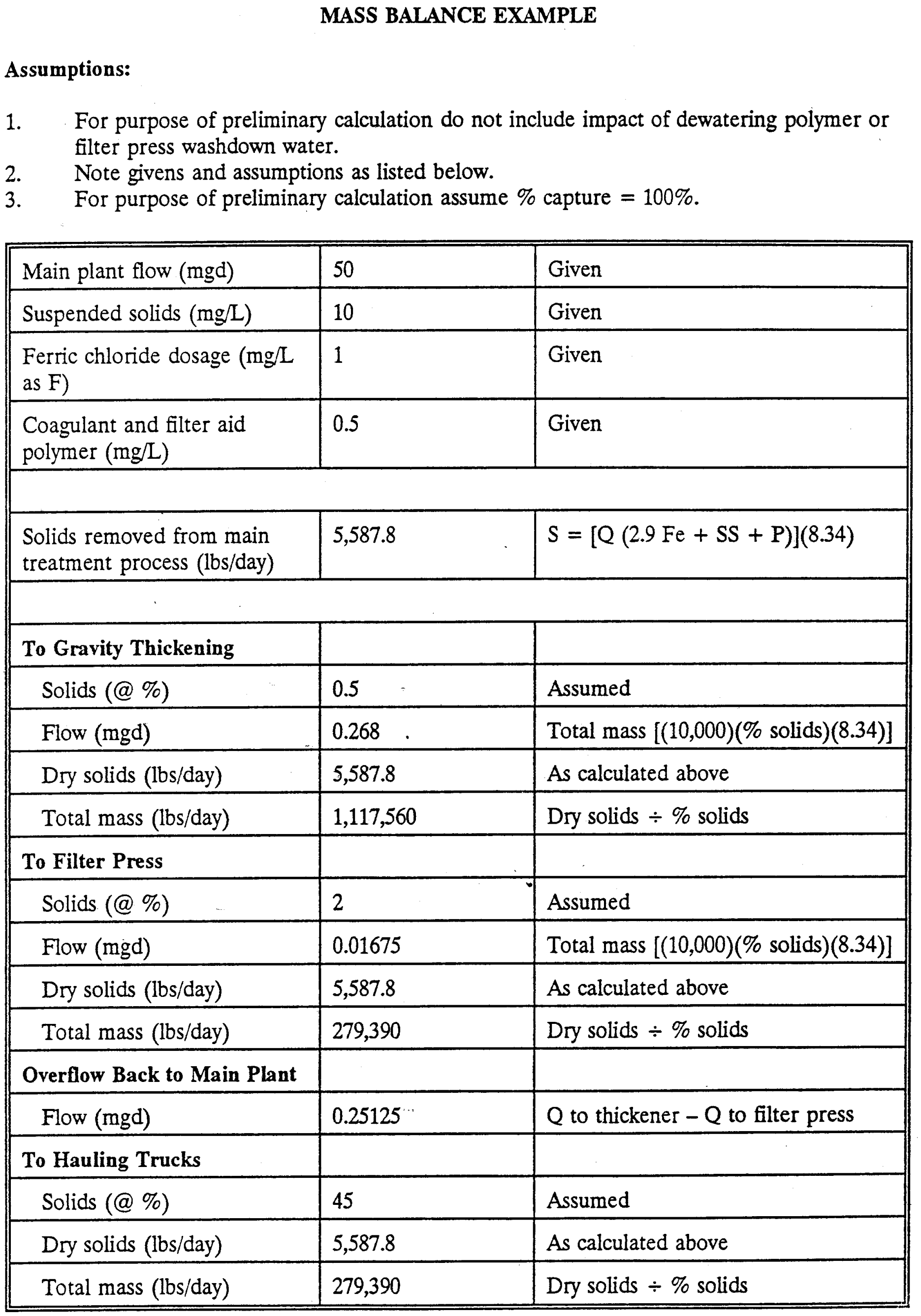
on refined flow rates and individual equipment loadings. Issues that must be identified at this preliminary design stage are the number of anticipated shifts, coordination with main plant operations, and provision for redundant process trains. Equipment size can also be reduced by providing adequate equalization facilities. Projected equipment suppliers also play a key role in this phase as the specific equipment capabilities and requirements are identified.

Open air dewatering/drying systems are the most common form of residual handling process. While the process approach and the system equipment are relatively simple, the actual design is dependent on local climatological conditions, regulatory requirements, and projected use. Detailed information on design details for these systems exists in the literature (Cornwell et al., 1987; Cornwell and Koppers, 1990).

4.7.3 Contingency Planning

The *Process Design Manual for Sludge Treatment and Disposal* (U.S. EPA, 1979b) presents the contingency planning design issues that should be incorporated into a WWTP solids handling facility. These issues are equally applicable to a water treatment facility.

Contingency planning concerns and possible design resolutions are listed in Table 4-8.



MASS BALANCE EXAMPLE

Assumptions:

1. For purpose of preliminary calculation do not include impact of dewatering polymer or filter press washdown water.
2. Note givens and assumptions as listed below.
3. For purpose of preliminary calculation assume % capture = 100%.

Main plant flow (mgd)	50	Given
Suspended solids (mg/L)	10	Given
Ferric chloride dosage (mg/L as F)	1	Given
Coagulant and filter aid polymer (mg/L)	0.5	Given
Solids removed from main treatment process (lbs/day)	5,587.8	S = [Q (2.9 Fe + SS + P)](8.34)
To Gravity Thickening		
Solids (@ %)	0.5	Assumed
Flow (mgd)	0.268	Total mass [(10,000)(% solids)(8.34)]
Dry solids (lbs/day)	5,587.8	As calculated above
Total mass (lbs/day)	1,117,560	Dry solids ÷ % solids
To Filter Press		
Solids (@ %)	2	Assumed
Flow (mgd)	0.01675	Total mass [(10,000)(% solids)(8.34)]
Dry solids (lbs/day)	5,587.8	As calculated above
Total mass (lbs/day)	279,390	Dry solids ÷ % solids
Overflow Back to Main Plant		
Flow (mgd)	0.25125	Q to thickener – Q to filter press
To Hauling Trucks		
Solids (@ %)	45	Assumed
Dry solids (lbs/day)	5,587.8	As calculated above
Total mass (lbs/day)	279,390	Dry solids ÷ % solids

Figure 4-15B. Mass balance calculation.

Backup systems and storage spaces are key contingency planning items that are often incorporated into residuals handling facility designs. An aerial view of a residuals handling facility is shown in Figure 4-17. This facility uses centrifuges as the primary mode of dewatering along with solar drying beds. The drying beds serve the following purposes:

- Handle peak solids loadings that the centrifuges may not be able to effectively treat.
- Serve as a dewatering process in case of centrifuge failure.
- Provide a dewatered solids storage area in the event that offsite solids hauling is disrupted.

4.7.4 Specific Design Elements of Mechanical Dewatering Systems

Mechanical dewatering systems are generally the most complex of the residuals handling systems. Layout examples for belt filter presses, centrifuges, filter presses, and vacuum filters are shown in Figures 4-18, 4-19, 4-20, and 4-21.

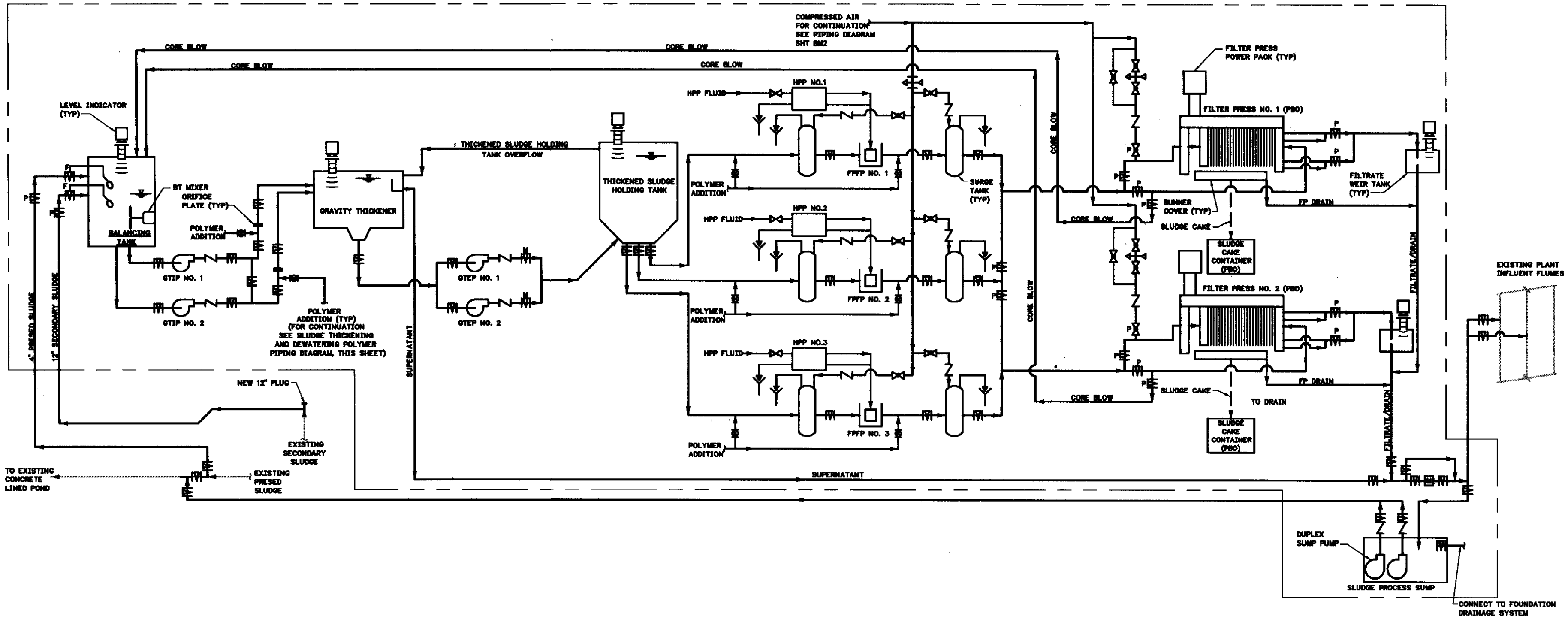


Figure 4-16. Preliminary residuals handling process schematic (Bratby et al., 1993).

Table 4-8. Residual Handling Facility Contingency Planning Issues (U.S. EPA, 1979b)

Potential Problems	Potential Resolutions
Equipment breakdowns	• Provide redundant units and piping. • Allow for solids storage during repair. • Base design on single shift; second or third shift can be utilized on an emergency basis.
Solids disposal disruption	• Plan for a secondary disposal approach. • Allow for solids storage during disruption.
Solids production greater than expected	• Provide for safety factors in process design. • Allow for solids storage during peak periods. • Base design on single shift; second or third shift can be used on emergency basis.

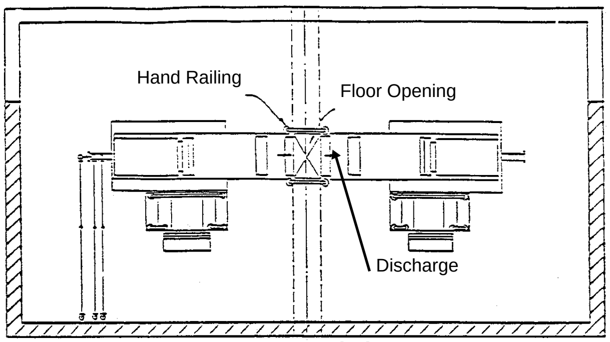
These schematics identify common elements of mechanical dewatering system design:

- Redundant dewatering units.
- Chemical conditioner (polymer) feed system.
- Simplified approach for removing dewatered solids from the unit.
- Byproduct water removal system.
- Mechanism for removing dewatering unit from building for repair.

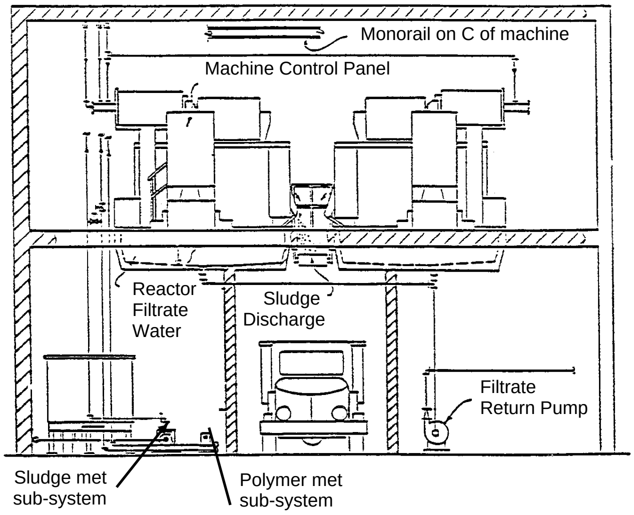
Additional design items that should be considered are:

- Local control of sludge feed into the units.
- Provisions for equipment washdown and drainage.
- Acoustic paneling in room to dampen noise.
- Odor control system (rarely implemented in water production facilities).
- Local shower facilities for staff.

Figure 4-17. Aerial view of residuals handling process system, Val Vista WTP, Cities of Phoenix/Mesa, AZ (John Carollo Engineers, 1994).



Upper Level Plan



Elevation

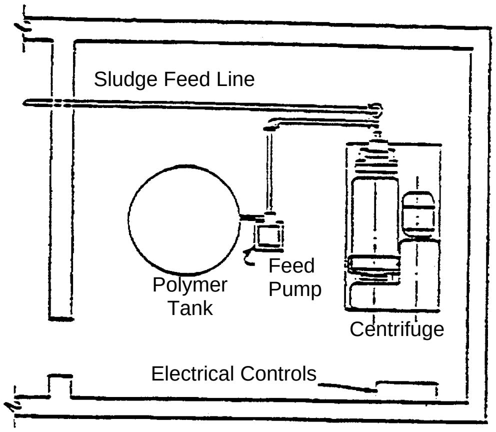
Figure 4-18. Belt filter press—example of system layout (U.S. EPA, 1979a).

4.8 Air Emissions Control

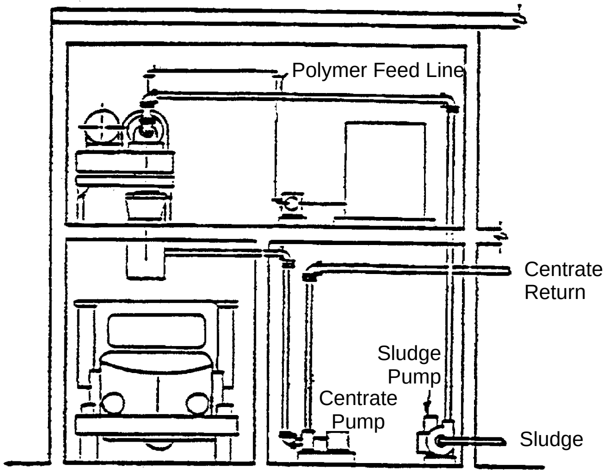
Along with traditional residuals generated by the physical/chemical treatment of water, some treatment processes can generate gaseous residuals that can be released to the atmosphere. Gaseous residuals form two groups: emissions intentionally generated as a byproduct of a treatment process, and emissions generated by the accidental release of a gaseous chemical used in water treatment.

4.8.1 Gaseous Residual Byproducts

The treatment of potable water can generate various types of gaseous residuals, from the simple release of hydrogen sulfide caused by aerating sulfur water, to the exhaust of various volatile and synthetic organic compounds from a stripping tower treating contaminated ground water. Other gaseous byproducts include the ozone-tainted air exhausting from an ozone contactor at a modern WTP, and radon released from contaminated source water.



Plan View

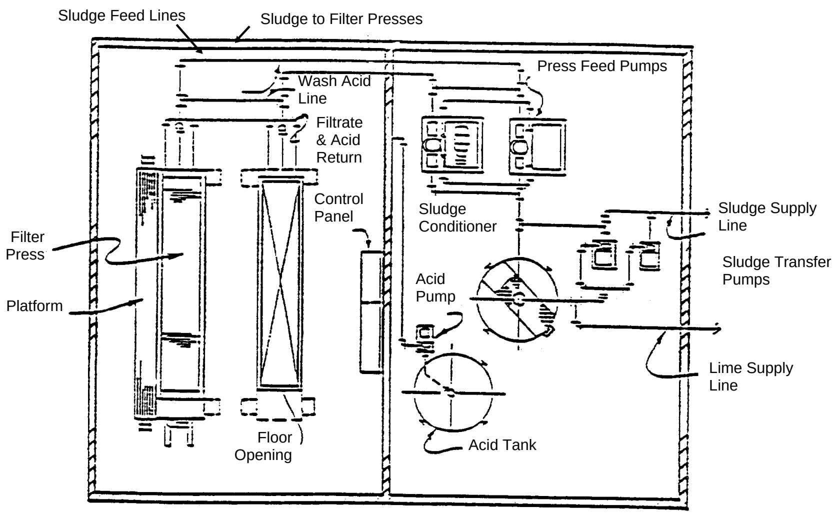


Section View

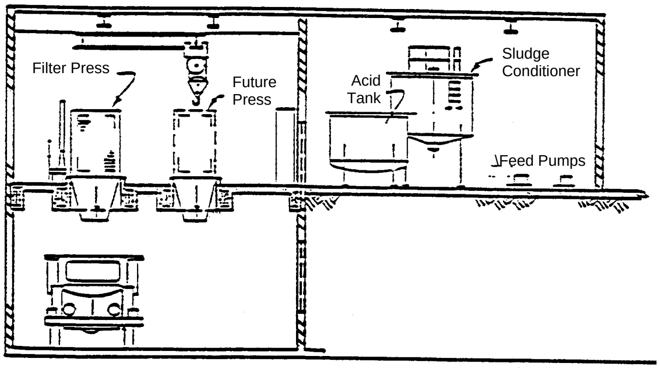
Figure 4-19. Centrifuge—example of system layout (U.S. EPA, 1979a).

4.8.1.1 Treatment of Inorganic Gaseous Emissions

The release of naturally occurring inorganic gases found in ground water (i.e., hydrogen sulfide) to the atmosphere is not an environmental concern unless it exceeds a 1 part per million (ppm) concentration. As urban development continues to encroach upon existing water treatment facilities, the concentration of gases in the exhaust from aeration and stripping processes may ultimately need to be reduced for aesthetic reasons, namely odor control. Aeration is a process that mixes air and water, normally by injecting air into the bottom of a tank of water; spraying water into the air; or allowing water to aerate by cascading over an irregular surface like an artificial stream bed. Stripping is a highly controlled form of aeration engineered for the optimum removal of a specific gas at a maximum concentration by spraying water into a variable counter-current flow of air through a stripping chamber.



Plan View



Elevation View

Figure 4-20. Filter press—example of system layout (U.S. EPA, 1979a).

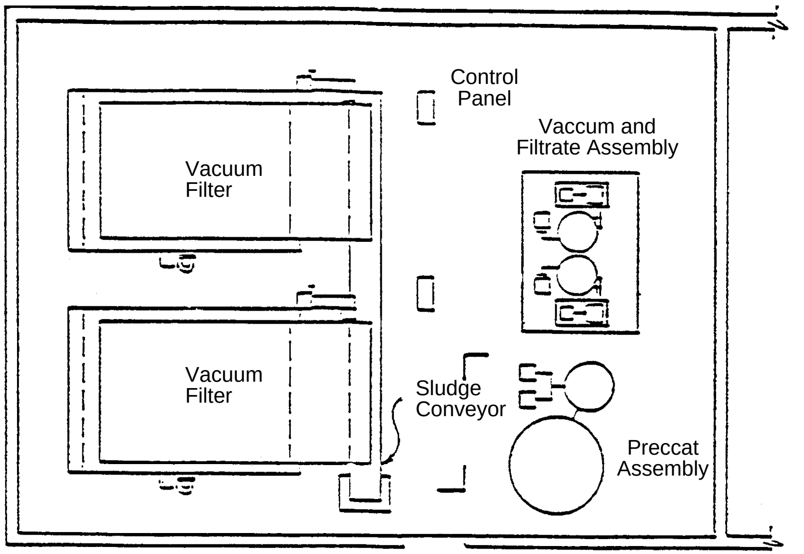
In cases where aeration is used, concentrations of hydrogen sulfide gas in the exhaust are below the 1 ppm threshold and are expelled directly into the atmosphere. If the gas concentrations in exhaust from an aeration process need to be controlled, the exhaust must be collected and delivered to a scrubber. Collection is normally achieved by installing a cover over the aeration process and using an exhaust fan to deliver the air to a scrubber system. When stripping is used, the exhaust gas is likely to exceed the 1 ppm concentration and would probably be directed to a scrubber system.

In air pollution control, a scrubber is an air cleaning process in which a contaminant gas is absorbed into a scrubbing liquid. The most common method of scrubbing is accomplished with a packed tower, a vertical chamber filled with various types of packing materials (see Figure 4-22). Process exhaust air is discharged into the bottom of the chamber and flows upward around the packing materials while scrubbing liquid trickles down through the packing in a counter-current flow pattern. The resultant sinuous pathway for the air and liquid provides an extended exposure of the air to the liquid and allows the gas to transfer from a gaseous state into a solution state and react with the scrubber liquid.

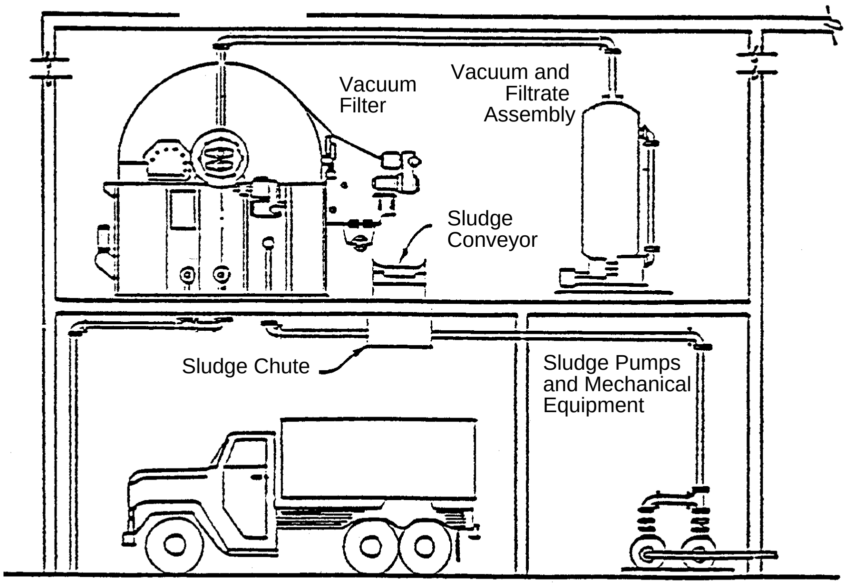
In the scrubbing of hydrogen sulfide, the scrubbing liquid is typically sodium hydroxide (caustic soda), which ultimately is converted to sodium sulfate through the reaction with hydrogen sulfide. To maintain scrubber efficiency, the scrubbing liquid is periodically replaced when it can no longer absorb the target gas. The sodium sulfate liquid is typically discharged directly into a sewer, as it is a nonhazardous residual. Other disposal alternatives must be evaluated in the event that a sewer is not available.

4.8.1.2 Treatment of Volatile Organic Gaseous Emissions

In many industrialized areas of the world, ground water has been contaminated by the careless disposal of various manmade organic chemicals. Most of these organic chemicals are very toxic and must be removed from the water before it is suitable for potable use. A group of these chemicals is called volatile organic chemicals or



Plan View



Elevation View

Figure 4-21. Vacuum filter—example of system layout (U.S. EPA, 1979a).

VOCs (e.g., trichloroethylene, known as TCE) because they can be easily removed from the water by violent agitation using a stripping chamber similar to that described above for removing inorganic gases.

Stripping chambers are similar to packed scrubbing towers except they have large blowers that can be regulated to vary the amount of air being forced into the chamber. Controlling the air flow rate relative to water flow rate regulates the stripping of the VOC from the water, which flows counter-current to the air flow. Depending on the concentration of the VOC being stripped and the local air quality standards, the stripper exhaust may be discharged directly into the atmosphere. Under these conditions, the volume of air forced through the stripper can effectively diffuse the concentration of the organic to trace levels.

In cases where the stripper has unacceptable concentrations of VOCs in the exhaust, it is discharged into an afterburner that incinerates the organic constituents. An afterburner is essentially a combustion chamber with an external fuel source that heats the air flowing through the chamber to a temperature sufficient to ignite the volatile organic gases. The size and temperature requirements of an afterburner depend on the specific organic chemical and the volume of air into which it is being vented.

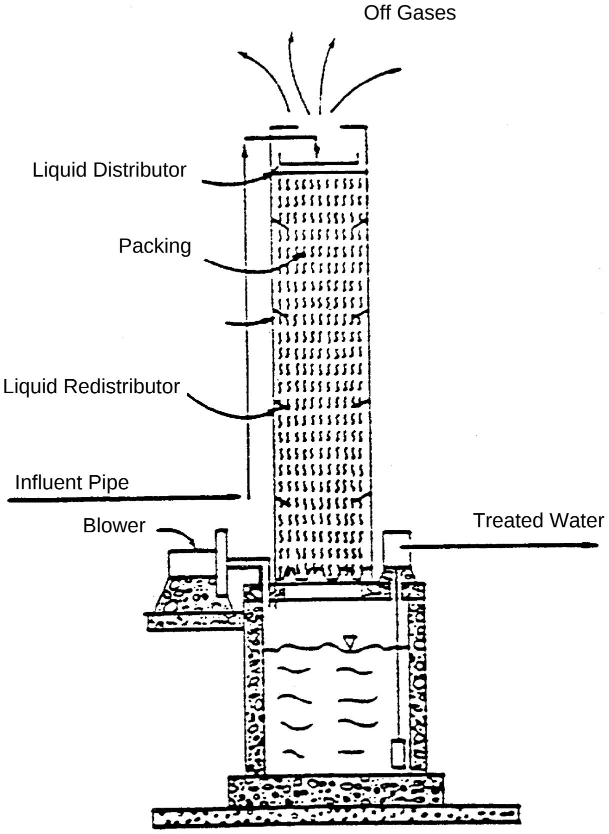


Figure 4-22. Packed tower (ASCE/AWWA, 1990).

4.8.1.3 Treatment of Nonvolatile Organic Gaseous Emission

A counterpart to the VOCs found in contaminated ground water is the nonvolatile organics group. These organic compounds are more difficult to strip, and their acceptable concentration in the exhaust is so limited that stripping with an afterburner is not a viable removal method. In these cases, the chemicals are removed using liquid-phase adsorption with granular activated carbon (GAC). Depending on the organic compound and the acceptable atmospheric discharge limits, the GAC can be regenerated or must be disposed of as a hazardous solid waste, as is the case with GAC containing dibromochloropropane (DBCP). Regeneration of GAC can be used for some nonvolatile compounds, but the regeneration exhaust is usually cycled through an afterburner for neutralization by incineration.

4.8.1.4 Treatment of Ozone-Contaminated Air

The use of ozone in potable water treatment has become more common within the water industry and may become even more popular as an alternative disinfectant. Although ozone is not the panacea to the pending Disinfectant/Disinfection Byproduct (D/DBP) Rule, it may be the only method that can truly deactivate infectious organisms like *Cryptosporidium*. When used in potable water treatment, its discharge into the atmosphere must be prevented.

Ozone is typically generated in an air flowstream or a pure oxygen flowstream discharged at least 22 feet below the water surface in a completely enclosed ozone contact chamber. Although the half-life of ozone is very short, trace concentrations in the basin exhaust are common. Two of the processes that are commonly used to eliminate the potential release of ozone into the atmosphere are thermal and catalytic destruction.

Thermal ozone destruction systems rely on the rapid decay of ozone at elevated temperatures. By heating the contactor off-gas to approximately 300 to 350 degrees Celsius for 5 seconds, the ozone is converted back to oxygen. The air is heated by a heat exchanger supplied with heat from electrically powered heating coils. As thermal pollution is also a concern in many places, the heated exhaust can be passed through an air-to-air heat exchanger, where heat is transferred to incoming air and thereby kept out of the atmosphere.

Catalytic ozone destruction units have only been in use since 1990. Although the true composition of the catalysts is considered proprietary by their manufacturers, in all designs the ozone is destroyed by electron-level exchanges and manipulations. Because the catalysts are subject to severe damage if exposed to moisture, they are usually combined with an air heater, which can also be used to regenerate the catalyst. The key advantage of the catalytic process is that it has the lowest energy costs of the two processes.

4.8.2 Accidental Release of a Gaseous Treatment Chemical

The oldest and most common water treatment process is chlorination, which, until recently, had been exclusively accomplished using compressed chlorine gas delivered and stored in cylindrical pressure vessels. The water industry has learned to handle the toxic chlorine gas with great care, but accidents do occur, usually resulting in the release of chlorine gas into the atmosphere.

The potentially deadly impact of the release of large amounts of chlorine has prompted many changes during the past 6 years in the applicable codes and laws regulating its use. In response to the increased regulatory constraints, some water utilities have modified their treatment process to reduce or eliminate the use of chlorine, and have implemented systems to provide neutralization of the chlorine in case of a leak. In some communities, public awareness and fear have forced the utilities to use a liquid form of chlorine (sodium hypochlorite/bleach). The liquid form is safer and essentially has the same disinfection capacity as the gas, but the liquid chlorine is much more expensive to transport and adds sodium to the finished water.

Since the use of chlorine is expected to continue into the future, it is reasonable to expect existing water utilities to construct chlorine containment and neutralization systems as required by the current building codes. The code requires that the accidental release of chlorine must be contained and neutralized using a treatment system such that the chlorine gas exiting the treatment exhaust does not exceed 15 ppm. The neutralization of chlorine requires the use of a scrubber system that employs caustic soda as the scrubber liquid. A chlorine scrubber operates essentially the same way as that used for inorganic gas emissions control, with one exception—the motive force for the extraction of the chlorine from the containment structure commonly uses a venturi with a caustic soda spray nozzle discharging into the throat of the venturi. The exhausted caustic soda becomes sodium hypochlorite (bleach), which can be disposed of through beneficial reuse as a disinfectant.

Coincidentally, the gaseous chemicals ammonia and sulfur dioxide are also commonly used in potable water treatment and can be neutralized by the same system as is used for chlorine.

Chapter 5
Direct Discharge of Water Treatment Plant Residuals to Surface Waters

Water treatment residuals are primarily produced through the coagulation, filtration, and oxidation of surface waters to remove turbidity, color, bacteria, algae, organic compounds and, frequently, iron and/or manganese. One type of residual is generated by the aluminum and iron salts that are commonly used for coagulation. A second type of residual results from the lime, sodium hydroxide, and/or soda ash added to reduce calcium and magnesium and soften the water.

Historically, direct discharge of water treatment effluent to surface waters has been the most commonly practiced disposal method. A 1984 water utility survey indicated that approximately 50 percent of the total residuals generated by 429 utilities (at least 548,820 metric tons) were pumped directly to surface waters (AWWA, 1986). Most of these residuals were alum sludges. Recently, this disposal method has been questioned because of concerns about possible risk to public health and aquatic life.

5.1 Theory

The chemistry of aluminum and iron in water are similar to one another; however, iron species are less soluble than aluminum species are over a wider pH range. Because of the potentially adverse effects of aluminum in the environment, more information is available on the discharge of alum residuals to surface waters than is information on residuals produced from the addition of iron salts. Consequently, this section will focus on alum residuals.

In the United States, aluminum in surface waters is primarily in solid form, with approximately 31 percent of the aluminum being dissolved (Andelman, 1973). Aluminum is abundant in the environment, yet in surface waters above pH 4.0, its concentration is normally less than 0.1 mg/L. Understanding the chemistry of aluminum is important to understanding the impact of aluminum on aquatic organisms.

5.1.1 Chemical Interactions

Aluminum is amphoteric—soluble in acidic and basic solutions, but quite insoluble at neutrality. The trivalent state, Al^{+3}, is the only naturally occurring oxidation state found in solutions and solids. The aqueous chemistry of aluminum is extremely complex; the molecular form, or species and concentration of each species depend upon pH, complexes with ligands, and, to a lesser extent, on temperature and reaction time (Campbell et al., 1983). Aluminum sulfate, used as the primary coagulant in most water treatment systems, dissociates in aqueous solutions, and Al^{+3} bonds with water molecules, hydroxide ions, other inorganic ions, and organic ions or molecules. When mixed thoroughly with turbid waters at pH levels from 4.0 to 8.5, new compounds form, especially with phosphates and organics, which are very insoluble and which, therefore, precipitate (U.S. EPA, 1988a; Driscoll and Schecher, 1988). Other aluminum coordination compounds are very soluble and only precipitate upon oversaturation.

While temperature and inorganic and organic ligand concentration are important factors affecting aluminum solubility, aluminum solubility is largely controlled by pH (Baker, 1982; Driscoll, 1985). The optimum pH for coagulation using aluminum salts generally ranges from 4.6 to 8.0. The presence of anions such as sulfate affects the solubility of aluminum salts and can move the optimum pH for coagulation to 4.0 (Hundt, 1986). Furthermore, the use of other chemicals in the water treatment scheme will affect the solubility and speciation of aluminum. Fluoride addition after alum coagulation causes increased aluminum solubility and an increase in inorganic Al^{+3} species concentration (Driscoll et al., 1987).

The source of aluminum ions in an aqueous environment is important. An additive can affect parameters important to solubility, such as pH and temperature. For example, if a loosely bound aluminum ion enters natural water from a sludge or desorbs from an organic clay particle, the resulting solubility of aluminum in that water will change as an indirect effect of the additive (Driscoll, 1985). Speciation is also dependent on the aluminum source, as it is influenced by the binding capability of ligands with aluminum ions. Tightly bound crystals of aluminum hydroxides will solubilize slowly over a period of months (Smith and Hem, 1972). Aluminum also may be held tightly within an organic complex and not solubilize readily.

The hydrated trivalent ion is the predominant form of aluminum at pH levels below 4.0. Between pH 5 and 6, the predominant hydrolysis products are $Al(OH)^{2+}$ and $Al(OH)_2$+, while the solid $Al(OH)_3$ is most prevalent between pH 5.2 and 8.8 (Martell and Motekaitis, 1989). The soluble species $Al(OH)_4$- is the predominant species above pH 9, and is the only species present above pH 10 (Martell and Motekaitis, 1989). Polymeric aluminum hydroxides appear between pH 4.7 and 10.5, and increase in size until they are transformed into colloidal particles of amorphous $Al(OH)_3$, which crystallizes to gibbsite in acidic waters (Brusewitz, 1984). Within the pH range of 5 to 6, aluminum bonds with phosphate and the resulting compound is removed from solution. Because phosphate is a necessary nutrient in ecological systems, this immobilization of both aluminum and phosphate may result in depleted nutrient states in surface waters (Brusewitz, 1984). The concentrations of aluminum in water vary with pH levels and the humic-derived acid content of the water. Even at neutral pH levels, higher aluminum levels have been found in lakes with a high humic acid content (Brusewitz, 1984; DHHS, 1990).

5.1.2 Toxicity

A key concern regarding the direct discharge of aluminum residuals to waterways is aluminum toxicity in the aquatic environment. When aluminum is mobilized in lakes and streams, it may be toxic to aquatic life. It is difficult to generalize about the environmental impacts of water treatment residuals discharges on receiving waterways, because such impacts are inherently dependent on an array of physical, chemical, and biological stream parameters. The characteristics of each case must be individually assessed to evaluate the potential threat of water treatment plant (WTP) residuals on the water and sediment qualities of receiving streams.

The aluminum species concentration causing toxicity depends on water chemistry, the organism being affected, and the effect monitored. The aqueous chemistry of aluminum has been difficult to study for several reasons. First, its form and, therefore, its effects vary with pH. Second, aluminum ions tend to undergo complex hydrolytic reactions with many ligands of differing chemistries, forming salts of differing solubilities. Additionally, aluminum is abundant in the environment, making it difficult to isolate the effects of residuals aluminum from those of ambient aluminum. Lastly, aluminum has varying solubility rates (i.e., weak complexes dissolve quickly and strongly bound complexes dissolve slowly) (George et al., 1991).

5.1.2.1 Aquatic Biota

Burrows (1977) presents toxicity data for several freshwater species of fish, invertebrates, bacteria, and algae. Studies have shown that under low pH conditions (pH less than 6), inorganic aluminum can be toxic to aquatic organisms. Major fish kills in southern Norway were attributed to the release of aluminum from soil by acid precipitation (Muniz and Leivestad, 1980). Furthermore, inorganic aluminum concentrations less than 400 μg/L had significant inhibitory effects on brook trout (Driscoll et al., 1980) and white suckers (Baker and Schofield, 1982).

Mobilization of aluminum in Adirondack lakes (pH of 4.3 to 5) may be the primary factor limiting trout survival there (Schofield and Trojnar, 1980). Freeman and Everhart (1971, 1973) studied the effects of aluminum hydroxide and aluminate (pH greater than 7.5) on rainbow trout and discovered that the aluminate ion is acutely toxic to trout at or above levels of 0.5 mg/L. In addition, freshly precipitated aluminum hydroxide can cause chronic injury in fish. Hall et al. (1985) conducted in-stream studies on the episodic effect of aluminum addition. They discovered that episodic additions of aluminum can have significant biological, physical, and chemical consequences in dilute, acidic surface waters.

An increase in aluminum under acidic conditions causes the disruption of the ion regulatory mechanism and, thus, the subsequent loss of tissue ions in various aquatic biota. Species of fish (Buckler et al., 1987; Muniz and Leivestad, 1980; Wood and McDonald, 1987), daphnids (Havas, 1985), and immature aquatic insects (Havas and Hutchinson, 1983; Witters et al., 1984) have experienced net losses of sodium (Na^+) and chloride ions (Cl^-) under certain conditions in the presence of aluminum. Ionic imbalances in fish resulting from aluminum exposure are usually accompanied by low pH and alkalinity levels in surrounding waters (Muniz and Leivestad, 1980).

Aluminum may also cause antagonistic effects to fish when pH is less than 5 (Brown, 1983; Evans et al., 1988; Havas, 1985; Hunn et al., 1987; Wood and McDonald, 1987). Baker and Schofield (1982) concluded that the initial step toward aluminum toxicity involved the binding of aluminum hydroxide to the gill surfaces.

Respiratory blockage in fish is another effect of increased aluminum concentrations. Fish may experience symptoms such as elevated ventilation frequency, increased standard oxygen uptake rate, and a fall in dorsal aortic oxygen tension (Baker, 1982; Muniz and Leivestad, 1980). This effect is normally observed at circumneutral pH where aluminum exists as a colloidal species (Malate, 1986; Ramamoorthy, 1988), or in the presence of high calcium concentrations, where the effect on the ion regulatory mechanism is masked (McDonald et al., 1983; Muniz and Leivestad, 1980; Wood and McDonald, 1987).

Aluminum may exert indirect effects on aquatic organisms as well. The release of copper (Cu^{2+}) from an organic chelator due to competitive replacement by aluminum was toxic to *Scenedesmus* (Rueter et al., 1987). In addition, aluminum's high affinity for phosphate and other compounds could remove nutrients from the environment and thereby inhibit productivity (Zarini et al., 1983; Sanville et al., 1976; Funk et al., 1977).

George et al. (1991) evaluated the toxicity of water extracts from the discharged alum residuals of ten North American WTPs. The receiving water samples from each plant were adjusted to three pH levels for testing purposes. Extracts from these samples were subjected to a battery of toxicity tests, including the *Selenastrum capricornutum* growth test, *Ceriodaphnia dubia* survival and reproduction test, the fathead minnow survival and growth test, a protozoan mortality test, and the Microtox test. *S. capricornutum* was more sensitive than any other test organism to the sludge extracts. Algal growth inhibition was observed at pH 5 and generally not at circumneutral pH. Dissolution of aluminum was observed in acidic and basic solutions; aluminum was only slightly soluble in circumneutral solutions. Alum sludge extracts were toxic to *S. capricornutum* at all receiving water pH levels when the receiving water hardness was less than 35 mg $CaCO_3$/L.

5.1.2.2 Benthic Macroinvertebrates

Potential threats to benthic communities are magnified when direct discharge occurs in quiescent bodies of water. When discharging WTP residuals to a low-velocity languid stream or lake, mass balance equations (see Section 5.2.1.3) do not apply. In this instance, a sludge deposit made up of discharge effluent will accumulate in the locus of the point of discharge. This poses a potential threat to any benthic communities at, or in close proximity to, the discharge point. A major concern with direct discharge is that residuals solids will cover the bottom sediments, damaging the periphyton community. Sludge deposits on sediments in receiving waters may limit carbon sources on which macroinvertebrates feed, and limit the available oxygen required for respiration. Areas of lesser flow with more clay substrate provide a more suitable benthic habitat than a sandy river channel does.

Historically, studies on the impacts of WTP effluent on benthic communities in large- to mid-sized rivers have found little evidence of related environmental degradation. A study by Evans et al. (1979) on the impact of waste discharges on the Vermillion River from a water works employing alum coagulation/filtration treatment techniques, concluded that the influence of the waste discharges on macroinvertebrates was imperceptible. A later study by Evans et al. (1982) on waste discharges from an alum coagulation/rapid sand filter plant on the Mississippi River, found no marked environmental degradation to the river, as measured by sediment size distribution and benthic macroinvertebrate abundance and diversity.

A study on the influence of residuals discharge on Crooked Creek in Illinois (Lin and Green, 1987) concluded that stream sediments immediately downstream of the discharge outfall showed an increase in chemical concentrations, a change in particle size distribution, and a shift in the diversity and abundance of macroinvertebrates. The macroinvertebrate biotic index (MBI), used by Illinois to indicate water quality on the basis of the type of benthic life present, showed no change at the sample stations immediately upstream and downstream of the water plant's discharge (Illinois, 1987).

Studies on the effects of alum sludge on benthic communities provide additional information on aluminum toxicity. Aluminum hydroxide has been shown to be toxic to *Tanytarsus dissimilis* (a representative of the chirocomidae), at concentrations of 80, 240, and 480 mg/L (Lamb and Bailey, 1981). A heavy floc layer on a lake bottom could inhibit deposited eggs from reaching the sediment and the surface.

An assessment of the effects of alum sludge discharges into coastal streams (Roberts and Diaz, 1985) shows a depression in the productivity of phytoplankton during an alum discharge event. This was attributed to the high level of suspended solids associated with the sludge discharge. The researchers concluded that "this turbidity effect in itself would argue for the cessation of sludge discharge even in the absence of toxic effects" (Cornwell et al., 1987). At plants where coagulation sludge is allowed to accumulate in settling basins for several months and is then discharged over short periods of time, a substantial increase in total suspended solids (TSS) and turbidity has been observed in the plant's receiving waters. Continuous withdrawal may minimize this problem (Illinois, 1987).

5.1.2.3 Nonaluminum Substances

Residuals discharged from lime softening WTPs may not adversely affect aquatic organisms due to the protection provided by high calcium and alkalinity levels. Toxicity testing, however, of residuals extract obtained at pH 8.3 from the Florence Water Treatment Plant (a lime softening facility), Omaha, Nebraska, indicated growth inhibition of *S. capricornutum* only in 50 and 100 percent extract solutions (George et al., 1995). Residuals extracts obtained at pH 6.0 also inhibited *S. capricornutum* growth in 12.5 percent extract solution plus 87.5 percent Missouri River water.

Most substances discharged into surface waters from WTPs have established toxicity levels and are state regulated. In-stream guidelines and standards are presented in Table 5-1. After characterizing the waste

Table 5-1. Possible In-Stream Water Quality Guidelines and Standards (Cornwell et al., 1987)

	Guidelines			Example Standards
	Aquatic Life Chronic Criteria			
	Fresh µg/L	Salt µg/L	Human Health[a]	Stream Used for Potable Water
Arsenic (dissolved)	72	63	2.2 ng/L	0.05 mg/L
Barium				1.0 mg/L
Beryllium	130		3.7 ng/L	
Cadmium	$e^{1.16\ (\ln(\text{hardness}))-3.841}$	12	10.0 µg/L	0.01 mg/L
Chloride				250 mg/L
Chromium	7.2	54		
hexavalent, dissolved				
trivalent, active	$e^{0.819\ (\ln(\text{hardness}))+.537}$		170 µg/L	
total				0.05 mg/L
Copper	2.0	4(2y) 23(A)		1.0 mg/L
Cyanide, free	4.2	0.57	20.0 µg/L	
Fluoride				1.4 mg/L
Hydrogen sulfide	2.0	2.0		
Iron				
total	1,000			
soluble				0.3 mg/L
Lead	$e^{1.34\ (\ln(\text{hardness}))-5.245}$	8.6	50 µg/L	0.05 mg/L
Manganese				
total		100		
soluble				0.05 mg/L
Mercury	0.00057	0.1	146 ng/L	0.002 mg/L
Nickel (total)	$e^{0.76\ (\ln(\text{hardness}))+1.06}$	7.1	13.4 µg/L	
Nitrate (as N)				10 mg/L
Phenol	1.0	1.0	3,500 µg/L	0.001 mg/L
Selenium	35	54	10 µg/L	0.01 mg/L
Silver	$0.01e^{1.72\ (\ln(\text{hardness}))-6.52}$	0.023	50 µg/L	0.05 mg/L
Sulfate				250 mg/L
TDS				500 mg/L
Zinc	47	58	5,000 µg/L	5.0 mg/L
Aldrin	0.03	0.003	0.074 ng/L	
Chloride	0.0043	0.004	0.46 ng/L	
Endrin	0.0023	0.0023	1.0 µg/L	0.0002 mg/L
Heptachlor	0.0038	0.0036	0.28 ng/L	
Lindane	0.08	0.0016		0.004 mg/L
Methoxychlor	0.03	0.03		0.10 mg/L
Toxaphene	0.013	0.0007	0.71 ng/L	0.005 mg/L
DDT	0.001	0.001	0.024 ng/L	0.1 mg/L
Chloroform	1,240		0.19 µg/L	
Radioactivity ($Ra^{226+228}$)				5 pCi/L
Gross alpha particle activity (excluding radon and uranium)				15 pCi/L

[a] Values given are the ambient water quality criteria for protection of human health for noncarcinogens, and for carcinogens the value is the risk of one additional case of cancer in 1,000,000 persons.

stream quality and flow rate, a mass balance calculation (see Section 5.2.1.3) should be conducted to determine possible toxicity of certain substances.

5.1.2.4 Summary

Based on the findings of research on aluminum toxicity, the following recommendations should be followed when disposal to surface waters is being considered for WTP residuals:

- Toxicity tests should be conducted on a variety of representative aquatic organisms from the intended receiving waters to fully determine potential toxic effects to the ecosystem.
- Alum sludges should not be discharged to acid streams (pH less than 6); discharge into water with circumneutral pH levels, however, should have minimal effects on primary production, and is acceptable.
- Caution should be taken in discharging alum sludges to soft receiving waters (i.e., hardness less than 50 mg $CaCO_3$/L).
- An environmental assessment should be undertaken for any sludge disposal to surface waters. Factors such as receiving water use, sediment structure, water chemistry, system hydrology, and receiving water biology need to be assessed to ascertain if discharge of alum sludges into a specific surface water will have a detrimental effect on the ecosystem.

5.2 Applications

5.2.1 Stream Hydraulics

The primary sources for residuals discharged to streams typically include clarifier underflow sludge, filter backwash wastes, and sedimentation basin sludge. These residuals are generated at varying rates. For example, filter backwash wastes are generated at high flow rates over short periods of time and may require equalization before discharge; sedimentation basin sludge is discharged more frequently according to periodic basin cleaning, unless the basins are equipped with mechanical sludge removal.

The impact that discharge of drinking water treatment residuals has on a stream depends upon the frequency of the discharge (ranging from daily to annually), the variability of the stream flow, and the load discharged. Under conditions of high velocity flow, more frequent controlled discharge is possible because the residuals will be suspended in the flow and dispersed downstream, diluting and minimizing the potential toxicity of the residuals. As previously described, under conditions of low velocity flow, high frequencies of discharge can significantly affect the aquatic ecology of the stream because the residuals will deposit in the vicinity of the outfall. This type of deposit can be subject to scouring during periods of high flow velocities.

5.2.1.1 Stream Characteristics and Flow Analysis

When considering the effects of a point source discharge on a stream, the principal stream characteristics of interest include geometry (width and depth relations), velocity, and flow. Stream flow, Q, is a function of the average stream velocity, U, and cross-sectional area, A, as:

$$Q = U(A) \qquad \text{(Eq. 5-1)}$$

Stream flow rates can be determined by several methods (Thomann and Mueller, 1987):

- Directly measuring cross-sectional area and average flow velocity (as with a current meter).
- Tracking of markers or dyes over time.
- Obtaining stage-discharge relations from gauging stations (operated by the U.S. Geological Survey along most major rivers).

Given the concept of dilution, it is intuitive that the effects of residuals discharges are most critical during low stream flow periods. Consideration of minimum flow involves an analysis of a stream's flow frequency. Generally, most design work uses the minimum average 7-day flow expected to occur once every 10 years. Minimum design flow is calculated using the following equation:

$$\text{minimum design flow} = a\ Q\ b \qquad \text{(Eq. 5-2)}$$

where
a = number of days used in the average
Q = stream flow (see Eq. 5-1)
b = interval in years over which the flow is expected to occur

The minimum design flow, therefore, is expressed as:

$$\text{minimum design flow} = 7(Q)(10) \qquad \text{(Eq. 5-3)}$$

Determinations of flow frequency can be made from plotting a stream's flow data and the percent of the time that the stream will have a flow less than or equal to a specific flow rate (Figure 5-1). From a flow frequency analysis of the Schuylkill River in Philadelphia, Pennsylvania, for instance, the discharge of a periodic effluent could be timed to coincide with the river's high flow periods to minimize effects of the discharge on the receiving stream.

5.2.1.2 Length to Mixing

The concentration and flow of a pollutant into a stream whose flow and other hydraulic properties are defined, can be approximated (Thomann and Mueller, 1987). This requires the assumption that water quality variables

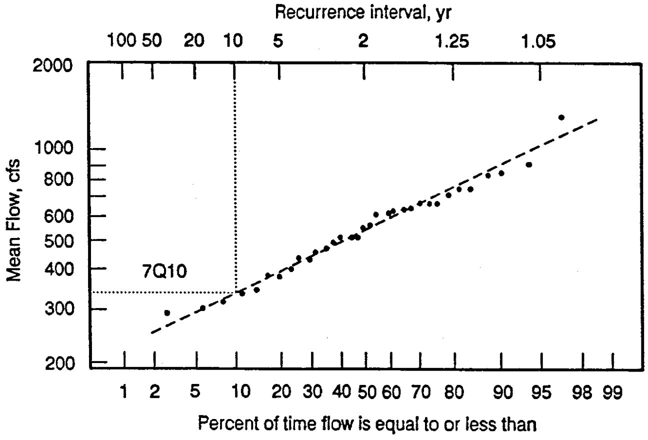


Figure 5-1. Flow frequency analysis of the Schuylkill River, Philadelphia, PA; minimum 7-day average flow values, 1932-1964 (after Thomann and Mueller, 1987).

are homogeneous laterally, across the stream, and vertically, with depth. In addition, it must be assumed that the stream flow involves only advective flow—i.e., there is no mixing in the longitudinal, or downstream, direction. True plug flow in streams is never attained, but these assumptions are valid for a simple steady-state analysis. Effluent discharge from a point source will result in a plume that will gradually spread across the stream and extend over the entire depth. The estimated distance from point of discharge to the point of complete mixing of effluent in the stream depends on whether the discharge enters the stream from the side bank or from midstream. For a side bank discharge, the length from the source to the zone where the discharge is laterally well mixed, L_m (in feet), is approximated by:

$$L_m = 2.6\ U\ B^2/H \qquad \text{(Eq. 5-4)}$$

where
U = average stream velocity in feet per second
B = average stream width in feet
H = average stream depth in feet

For a midstream discharge, the mixing length is approximated by:

$$L_m = 1.3\ U\ B^2/H \qquad \text{(Eq. 5-5)}$$

5.2.1.3 Flow and Mass Balance Calculations

Flow and mass balance calculations can be used to estimate pollutant concentrations downstream from an effluent discharge. To simplify mass balance calculations for point source effluent discharge to a stream, assume that effluent mixing in the stream occurs instantaneously and that flow occurs only through advective processes—i.e., there are no dead zones downstream to cause flow dispersion, for example.

Figure 5-2 shows two scenarios for water supply and effluent discharge sources for a WTP, where

Q_u = upstream flow
Q_i = influent plant flow
Q_e = effluent plant flow
Q_d = downstream flow
C_u = upstream pollutant concentration
C_e = effluent pollutant concentration
C_d = downstream pollutant concentration

The flow variables are in units of volume per time and the concentration measurements are in units of mass per volume. This results in units of mass per time, or mass flux, in the mass balance calculations.

In the first scenario (common supply/discharge source) depicted in Figure 5-2, the flow balance equation is:

$$Q_d = Q_u - Q_i + Q_e \qquad \text{(Eq. 5-6)}$$

The mass balance equation is:

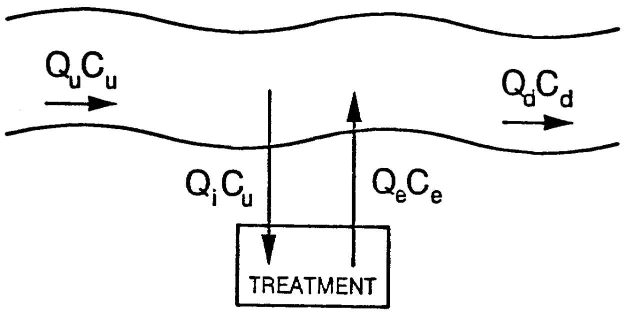
$$Q_dC_d = Q_uC_u - Q_iC_u + Q_eC_e \qquad \text{(Eq. 5-7)}$$

Typically, the upstream concentration of the pollutant is omitted because it is much less than that of the effluent or downstream concentration. The mass balance equation is then reduced to:

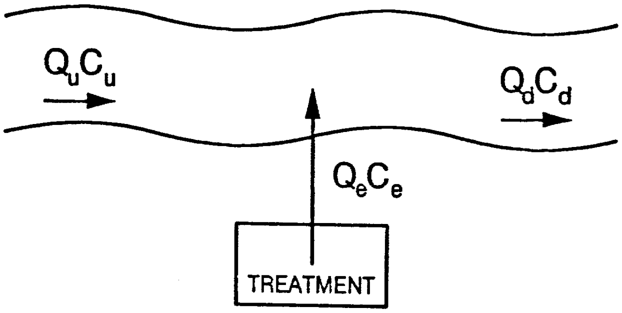
$$Q_dC_d = Q_eC_e \qquad \text{(Eq. 5-8)}$$

or

$$C_d = (Q_eC_e)/Q_d \qquad \text{(Eq. 5-9)}$$



Common Supply/Discharge Source



Different Supply/Discharge Sources

Figure 5-2. Scenarios for mass balance calculations.

In the second scenario for water supply and effluent discharge sources, the flow balance equation is:

$$Q_d = Q_u + Q_e \qquad \text{(Eq. 5-10)}$$

The mass balance equation is:

$$Q_dC_d = Q_uC_u + Q_eC_e \qquad \text{(Eq. 5-11)}$$

Again, assuming the upstream concentration is insignificant, the mass balance equation is reduced to the same form as for the first scenario:

$$C_d = (Q_eC_e)/Q_d \qquad \text{(Eq. 5-12)}$$

If the effluent stream is continuous, the pollutant is conservative (its concentration does not degrade with time or flow), and the river is assumed to flow as an ideal plug flow reactor, then the downstream concentration, C_d, will be constant for all downstream distances. Pollutant concentrations from WTP effluents, however, will decrease downstream from the discharge point through advective flow processes in the river, and chemical reactions, bacterial degradations, or particulate settling of the pollutant in the water column. Typically, the concentration decreases occur according to first order reaction kinetics. The concentration of a substance downstream is then calculated as:

$$C_d = C_m[(-kX)/U] \qquad \text{(Eq. 5-13)}$$

where

C_d = downstream pollutant concentration
C_m = mixed effluent and upstream pollutant concentration
k = first order reaction constant
X = distance downstream from the discharge point
U = river velocity

The change in pollutant concentration with downstream flow is shown in Figure 5-3.

5.2.1.4 Water Quality Controls

Three factors influence the downstream fate of a pollutant: effluent flow rate, stream flow rate, and the decay rate of the pollutant (Thomann and Mueller, 1987). By modifying one of these variables, the downstream water quality can be improved. Figure 5-4 shows the effects of three techniques for controlling downstream water quality: decreasing effluent flow rate (a), making use of increased upstream flow rate (b), and increasing decay rate, k (c). With the first technique, if the effluent flow rate is decreased and the effluent concentration remains constant, the mixed pollutant concentration at the discharge point and the downstream concentration will be reduced.

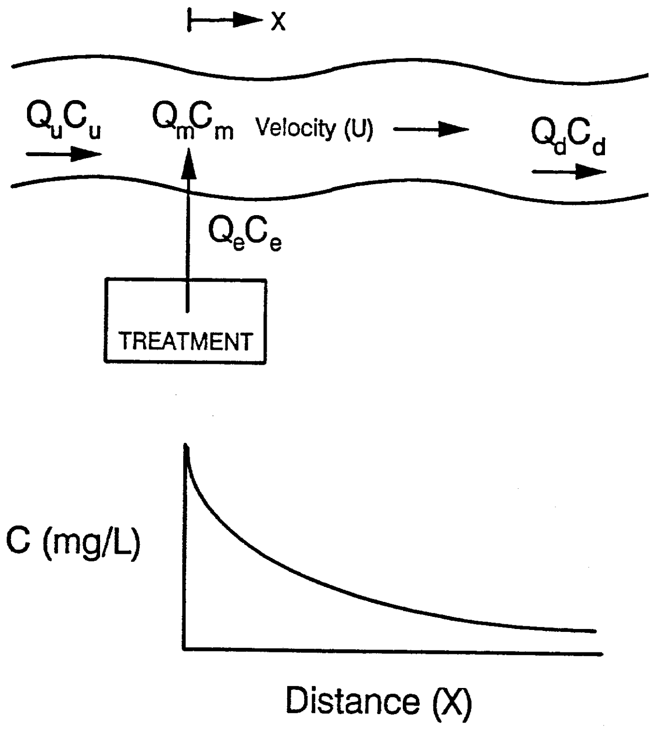


Figure 5-3. Decay of a nonconservative pollutant.

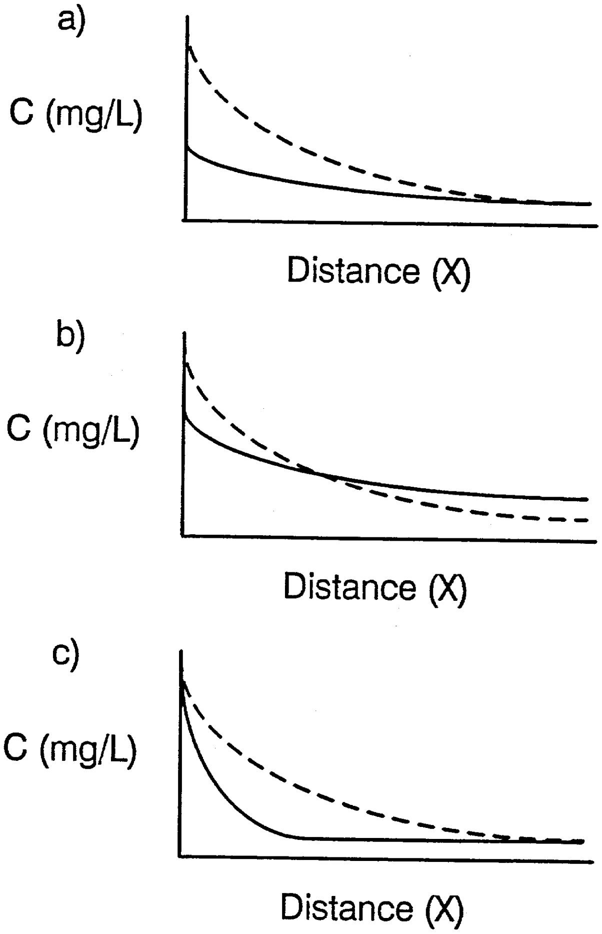
The second technique involves discharging during increased upstream flow rates, as from a storm event. In this case, the discharged effluent will mix with a greater flow of water in the stream and pollutant concentrations near the discharge point will decrease. The disadvantage of this technique is that high stream flow rates will carry pollutant concentrations farther downstream than usual.

The third technique requires more sophisticated methods to modify the pollutant properties so that its decay rate increases in the stream. The pollutant concentrations at the discharge point are unaffected by this technique, but the increased decay rate results in rapid decreases in downstream pollutant concentrations.

5.2.2 Available Transport and Chemical Models and Their Application

Numerous computer models are used to predict the transport and fate of pollutants in natural systems. The EPA Center for Exposure Assessment Modeling in Athens, Georgia, provides and supports public domain software that simulates air, water, and soil processes, enabling the user to make risk-based decisions concerning environmental protection (Ambrose and Barnwell, 1989). The software is available to the user at no cost, but considerable technical expertise is needed to successfully use this software for problem-solving.

MINTEQ, for example, is one of six geochemical-equilibrium computer models routinely used to quantitatively evaluate the equilibrium behavior of inorganic and organic constituents in various chemical environments (U.S. EPA, 1991a; Bassett and Melchoir, 1990; U.S.



a) decreased effluent flow rate;
b) increased upstream flow rate;
c) increased decay rate
Note: dashed lines represent initial conditions and solid lines represent improved conditions.

Figure 5-4. Control techniques for improving downstream water quality.

EPA, 1984a). MINTEQ describes the chemical conditions and reactions that control leaching. It then calculates the fate (or mass distribution) of dissolved and solid phases of a pollutant under equilibrium conditions in dilute aqueous systems. The model considers ion speciation, adsorption, redox, gas-phase equilibria, and precipitation and dissolution reactions in its calculations. To obtain accurate information from the model, users must be familiar with these processes and techniques and be able to quantify them for a specific pollutant and river system. There are currently no user-friendly aqueous chemical equilibrium models that are sophisticated enough to make the necessary accurate downstream water quality predictions.

5.3 Examples

An investigation of the adverse effects of alum sludge on benthic invertebrates was conducted in 1991 by Tennessee Technological University (TTU) for the AWWA Research Foundation (George et al., 1991). Three of the four following examples were summarized from this research report. The fourth example was obtained from a report to the City of Phoenix, Arizona, evaluating the discharge of three WTPs to the Salt River Project canal system (HDR, 1995).

5.3.1 California Plant, Cincinnati Water Works, Cincinnati, Ohio

5.3.1.1 Description

The California Plant in Cincinnati, Ohio, treats drinking water at a rate of 618 m^3/min. The WTP consists of primary coagulation-sedimentation, offstream storage reservoirs, secondary coagulation-sedimentation, sand filtration, and disinfection facilities (Figure 5-5). Primary sedimentation is achieved using a 460 m^3/min facility equipped with plate settlers to separate solids that are continuously returned to the Ohio River. This process is responsible for the bulk of the solids removal conducted at the facility. The water from the storage ponds is pumped to the secondary sedimentation facility, which delivers a low turbidity water to the sand filters. The storage reservoirs provide emergency capacity during hazardous spill conditions that might occur in the Ohio River system.

Aluminum sulfate and polymer are added to the raw water. Ferric sulfate, powdered activated carbon (PAC), and polymer are used intermittently. When needed, PAC can be applied to the raw water; after the primary sedimentation basins and before pumping the water to the storage reservoirs; and at the rapid-mix basin before the secondary sedimentation basins. The plant can prechlorinate the raw water; add chlorine at the rapid-mix basin between the reservoirs and the secondary sedimentation facility; chlorinate between the secondary sedimentation basins and the sand filters; and postchlorinate between the filters and the clearwell. Ferric sulfate, lime, and soda ash also can be added at the rapid-mix basin prior to the secondary sedimentation basins. The quantities of chemicals used at the California Plant from July 1988 through June 1989 are presented in Table 5-2.

Waste solids from the secondary sedimentation facility and the backwash water are transported to the primary sedimentation facility, which has plate settlers. An average of 5 m^3/min of the wasted sludge (0 to 4 percent solids) from the primary sedimentation basins is discharged continuously to the Ohio River.

Data on the chemical composition of the Cincinnati sludge and of the Ohio River at the point of discharge are presented in Table 5-3. The Ohio River samples tested by George et al. (1991) were moderately hard (123 to 155 mg $CaCO_3$/L); alkalinity ranged from 53 to 67 mg $CaCO_3$/L; and total aluminum concentrations

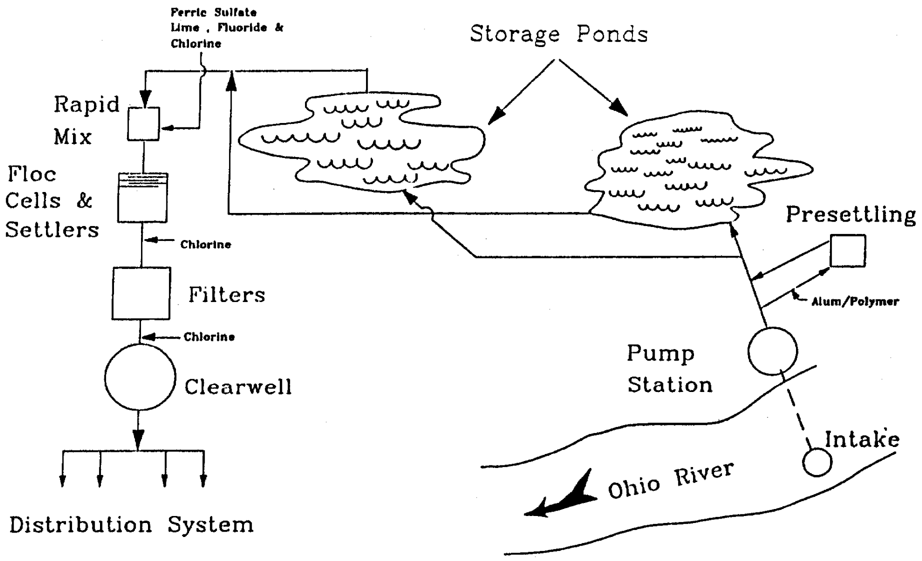


Figure 5-5. Flow schematic of California Plant, Cincinnati Water Works, Cincinnati, OH (George et al., 1991).

Table 5-2. Average Daily Chemical Use at California Plant, Cincinnati Water Works, Cincinnati, OH (George et al., 1991)

Month	Alum (kg/d)	Polymer (kg/d)	PAC (kg/d)	Ferric Sulfate (kg/d)	Lime (kg/d)	Fluoride (kg/d)	Chlorine (kg/d)
1988							
July	6,882	0	970	0	6,729	652	2,970
August	6,313	0	6,628	877/15[a]	8,133	666	3,453
September	4,948	0	2,302/27[a]	766/13[a]	5,938	445	2,585
October	4,886	0	0	742/18[a]	4,695	425	1,876
November	4,895	207	0	646/5[a]	3,895	363	1,688
December	6,648	142	0	656/16[a]	4,501	398	1,546
1989							
January	8,849	0	0	662/19[a]	4,751	405	1,426
February	7,470	986	0	965/13[a]	4,695	452	1,223
March	6,346	625	1,404	638/18[a]	4,171	429	1,110
April	6,864	405	3,103	0	4,156	465	1,132
May	7,555	574	1,986	618/16[a]	4,540	466	1,134
June	4,886	531	0	757/25[a]	4,702	554	1,806
Mean[b]	6,379	496	2,731	733	5,076	477	1,829

were less than 0.02 to 0.56 mg/L. The waste stream from the WTP contained little aluminum (1.69 to 24.1 mg/L), and suspended solids varied from 170 to 4,272 mg/L. The Ohio River provided tremendous dilution of the waste stream from the plant (2 x 10^{-5} m^3 sludge/m^3 of river flow).

5.3.1.2 Toxicity

Samples of discharged sludge and receiving water were taken for three different pH levels. Toxicity tests indicated that none of the sludge extracts produced a toxic response in a protozoan *(Tetrahymena)* mortality test, Microtox toxicity test, *Ceriodaphnia* toxicity test, or fathead minnow *(Pimephales)* toxicity test. At pH 5, however, the sludge extract was toxic to *S. capricornutum* (Table 5-4) at a 50 percent and 100 percent extract exposure. No adverse response was observed at pH 7.4 and 8.

5.3.1.3 Benthic Organisms

At the time it was studied, the California Plant discharged alum sludge to the Ohio River at a point just above the confluence with the Little Miami River. PAC was added as needed for taste and odor problems.

Table 5-3. Chemical Composition of Sludge and Ohio River Water Sampled at California Plant, Cincinnati Water Works, Cincinnati, OH, September 21, 1988, December 18, 1988, and January 10, 1989 (George et al., 1991)

	Sludge			Ohio River		
Parameter	9/21/88	12/18/88	1/10/89	9/21/88	12/18/88	1/10/89
Aluminum (mg/L)	24.1	23.6	1.69	<0.02	<0.02	0.56
Iron (mg/L)	12.0	13.4	0.84	0.16	<0.005	0.56
Manganese (mg/L)	3.59	<0.005	0.056	0.048	<0.005	0.22
Calcium (mg/L)	124.0	43	34	35	40	34
Magnesium (mg/L)	20.3	12.5	9.4	13.5	10.0	9.2
Total organic carbon (mg/L)	84	2.3	21	2.5	2.1	6.4
Alkalinity (mg $CaCO_3$/L)	1,122	56	53	54	53	67
Total suspended solids (mg/L)	4,272	170	3,656	10.2	5.4	92
pH	7.8	7.0		7.4	7.6	

Table 5-4. *S. Capricornutum* Test Results on Alum Sludge Extracts From California Plant, Cincinnati Water Works, Cincinnati, OH (George et al., 1991)

Sample Site pH	Percent Filtrate	Monomeric Aluminum Concentration (mg M-Al/L)	Percent Inhibition	Percent Stimulation	EC_{50}
pH 5	12.5	<0.04	NE		—[a]
	25	0.05	NE		—
	50	0.10	35		—
	100	0.20	70		—
pH 7.4	12.5	<0.04	NE		—
	25	<0.04	NE		—
	50	0.07	NE		—
	100	0.14	NE		—
pH 8	12.5	0.05	NE		—
	25	0.10	NE		—
	50	0.19		50	—
	100	0.38	NE		—

[a] Insufficient data to compute EC_{50}.

Key

NE = No statistically determined effect.

EC_{50} = Effective concentration producing a 50 percent reduction in growth rate compared with the growth rate of control populations.

In January, March, and May 1989, water and sediment samples were collected from the Ohio River to determine the impact of the sludge discharge on the benthic communities. During January, aluminum concentrations in the water column immediately above the sediments ranged from 1.63 to 4.67 mg/L, with the highest concentration occurring at the point of discharge. Sediment aluminum concentration (Figure 5-6) was also greatly elevated at the point of discharge (23,560 mg/kg) and gradually decreased to 1,882 mg/kg of dry sediment as the sludge was dispersed downstream.

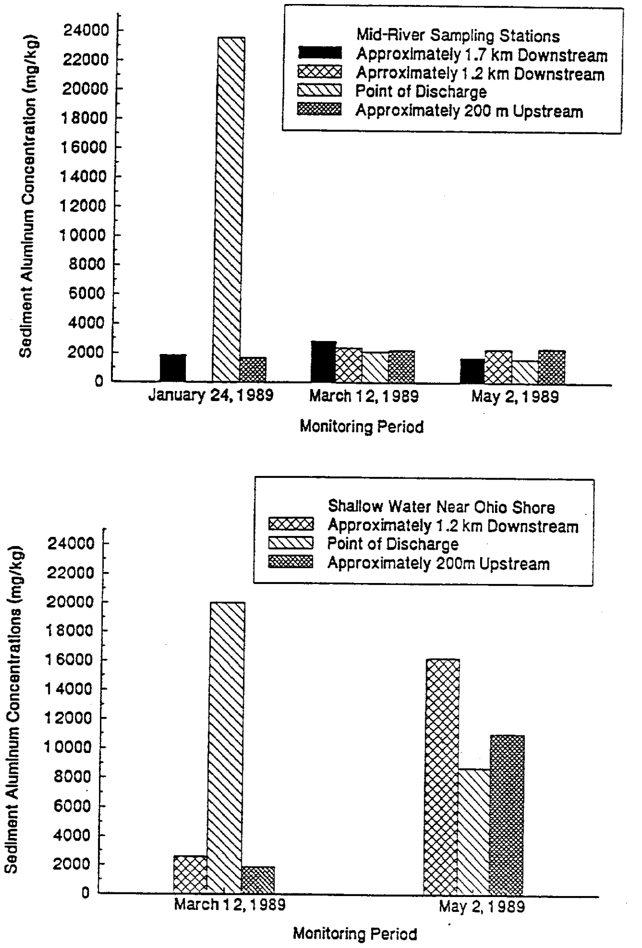


Figure 5-6. Sediment aluminum concentration (dry weight) from Ohio River, Cincinnati, OH (George et al., 1991).

Aluminum concentrations in the water column immediately above the sediment collected in March ranged from 3.92 to 8.96 mg/L. The two highest concentrations of aluminum were found at 1,700 meters downstream (8.69 mg/L) and 1,158 meters downstream (8.96 mg/L), and were approximately twice the aluminum concentrations measured at the upstream stations. Discharged alum sludge floc may have been entrained in the water currents. The greatest sediment concentrations of aluminum observed in March were collected near the point of discharge. Concentrations of aluminum in the sediment ranged from 2,100 to 20,000 mg/kg dry weight. Increased concentrations of aluminum at the point of discharge (20,000 mg/L) were expected, due to settling of suspended solids as the stream velocity decreases toward the bank of the river. Aluminum concentrations in the Ohio River during May remained fairly constant, ranging from 0.94 to 1.27 mg/L. In May, aluminum concentrations in the sediment were highest near the shore with less than 2,500 mg/kg measured in the deeper channel.

The California Plant commonly added ferric sulfate, along with increased amounts of aluminum sulfate, to the raw water because of high turbidity following rain events. The mean iron concentration in January was 1.79 mg/L (range 0.79 to 4.14 mg/L); it increased to 6.52 mg/L (range 2.55 to 7.90 mg/L) in March. By May (sampling period #3), the mean iron concentration dropped to 1.92 mg/L (range 1.56 to 2.49 mg/L). Aluminum concentrations were also higher in March, increasing to a mean of 5.69 mg/L (range 3.92 to 8.69 mg/L) from a mean of 2.57 mg/L (range 1.63 to 4.67 mg/L) observed on the previous sampling trip in January. The alum sludge discharge, more dense with the addition of PAC, apparently settled out quickly near the point of discharge in January. Greater flow throughout the remainder of the study period transported sediment deposits downstream from the point of discharge.

Benthic macroinvertebrate data from the same sampling location are presented in Table 5-5. Benthic samples were not collected in May because of high flow velocities. Heavy scouring of the sandy bottom during high flow was observed by Chisholm and Downs (1978), who found damage to benthic communities resulting from the relatively high flow velocities. All composited benthic macroinvertebrate samples collected contained fewer than 100 organisms. As the accuracy of the species diversity indices is diminished when the samples contain fewer than 100 organisms (U.S. EPA, 1983), no diversity indices were calculated. Dipterans (*Chironomidae*) and oligochaetes were the dominant taxa collected in these samples.

Table 5-5. Benthic Macroinvertebrates Collected From Site CO on the Ohio River, Cincinnati, OH, 1989 (George et al., 1991)

Distance From Point of Discharge	Number of Taxa	Organism Density (No. Organisms/m^2)
Sampling Period 1, January 24-25, 1989		
1,661 m downstream	3	113
1,158 m downstream	7	646
0 (point of discharge)	—[a]	—[a]
200 m upstream	4	1,227
Sampling Period 2, March 12-13, 1989		
1,661 m downstream	—[a]	—[a]
1,158 m downstream	5	330
0 (point of discharge)	5	174
200 m upstream	7	322
Sampling Period 3, no sample		

[a] Zero macroinvertebrates found.

No organisms were collected at the point of discharge in January; this phenomenon was probably due to sludge accumulations over the substrates. The temporary nature of these sludge deposits was evidenced by the presence of a benthic community sampled at the point of discharge in March. At the same time, no organisms were found in the sample taken 1,661 m downstream from the point of discharge. The lack of organisms at this site immediately below the confluence of the Little Miami and the Ohio Rivers was probably due to high flow conditions in both water bodies and apparent scouring of the sandy bottom. Low numbers of taxa and organisms at all stations indicate stress to the benthos in the study area. Lack of suitable substrate is believed to be a major contributing factor to the low species richness numbers of organisms in this reach of the river.

In summary, the effects of alum sludge discharged from the California Plant into the Ohio River could not be separated from ambient impacts within the system. High concentrations of sludge components in the study area appear to be transitory in both the water column and sediment. Benthic macroinvertebrate communities appear to exhibit transient impacts at the point of discharge that were overshadowed by high-flow scouring from storm events and lack of suitable habitat.

5.3.2 Ralph D. Bollman Water Treatment Plant, Contra Costa Water District, California

5.3.2.1 Description

The Ralph D. Bollman Water Treatment Plant, Contra Costa Water District, California, treats water from the Sacramento–San Joaquin River delta in northern California. Flow from the river delta is pumped into Mallard Reservoir (capacity 13.8 million m^3), where it is held until it is treated. The design capacity of the WTP is approximately 237 m^3/min, and the average production is 81 m^3/min. Water treatment consists of prechlorination; alum coagulation; flocculation; sedimentation; chlorination prior to filtration; high-rate, dual-media filtration; ammoniation; and pH control (Figure 5-7). At the time of

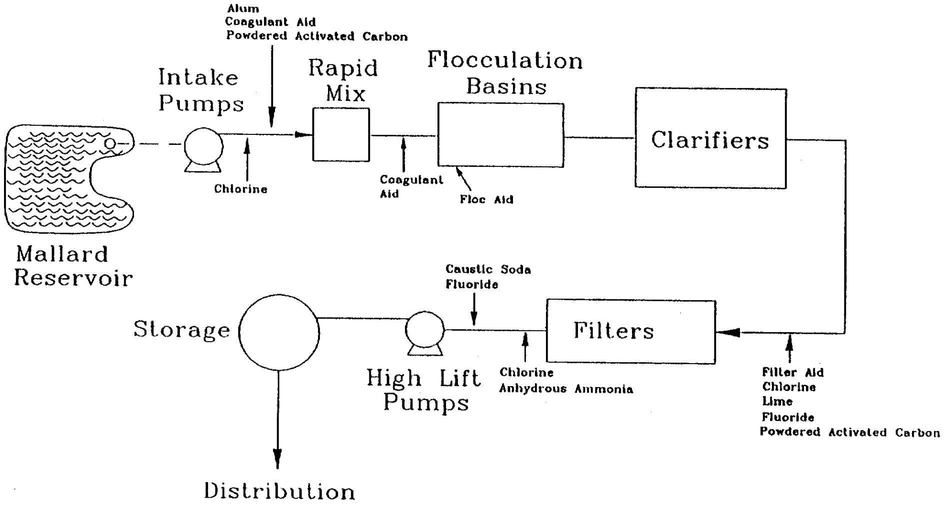


Figure 5-7. Flow schematic of Ralph D. Bollman WTP, Contra Costa Water District, CA (George et al., 1991).

this study, PAC was used to control taste and odor problems. From November 1986 to December 1988, the average dosage of PAC was 3.5 mg/L. The 1988 dosage totaled approximately 146 metric tons. Also in 1988, an average of 1,816 metric tons of alum and 182 to 227 metric tons of lime were added to the water.

Solids residuals from the sedimentation basins were discharged daily to an embayment of Mallard Reservoir for a 15-month period (October 1987 through December 1988). Filters were backwashed approximately once every 50 hours and the water was wasted to the reservoir. During 1988, the backwash water was approximately 1 percent of the water treated. Approximately 1,504 metric tons of solids were discharged into Mallard Reservoir. The water district did not waste solids from the sedimentation basins to Mallard Reservoir after December 1988.

Sludge samples were collected from Mallard Reservoir in October 1988 and February 1989 and contained from 803 to 1,450 mg/L aluminum and 196 to 319 mg/L iron. Total suspended solids concentration in the sludge was 13,733 mg/L. Mallard Reservoir water contained 0.36 to 0.81 mg/L aluminum and was moderately hard (approximately 126 to 143 mg $CaCO_3$/L). Chloride concentrations were high (159 to 217 mg/L) due to salt water intrusion into the Sacramento–San Joaquin River delta.

5.3.2.2 Toxicity

Sludge extracts obtained with Mallard Reservoir water, which was pH adjusted, were not toxic to bacteria (Microtox assay), the protozoan *Tetrahymena ceriodaphnia*, or fathead minnow. Sludge extracts obtained with pH 5 reservoir water, however, were toxic to *S. capricornutum* (Table 5-6). An EC_{50} (the effective concentration producing 50 percent reduction in growth rate compared with the growth rate of control populations) of 22.5 percent was computed. Although the alum sludge extract obtained at pH 7.5 stimulated algal growth at the lower extract concentrations (i.e., 12.5 percent and 50 percent), no effect occurred at any concentration at pH 8.5.

Table 5-6. *S. Capricornutum* Test Results on Alum Sludge Extracts From Ralph D. Bollman Water Treatment Plant, Contra Costa, CA (George et al., 1991)

Sample Site pH	Percent Filtrate	Monomeric Aluminum Concentration (mg M-Al/L)	Percent Inhibition	Percent Stimulation	EC_{50}
pH 5	12.5	<0.04	37		22.5%
	25	0.05	54		—[a]
	50	0.10	60		—[a]
	100	0.20	100		—[a]
pH 7.5	12.5	<0.04		29	—[a]
	25	<0.04	NE		—[a]
	50	0.05		43	—[a]
	100	0.09	NE		—[a]
pH 8.5	12.5	0.07	NE		—[a]
	25	0.14	NE		—[a]
	50	0.27	NE		—[a]
	100	0.54	NE		—[a]

[a] Insufficient data to compute EC_{50}.

Key

NE = No statistically determined effect.

EC_{50} = Effective concentration producing a 50 percent reduction in growth rate compared with the growth rate of control populations.

5.3.2.3 Benthic Organisms

The Ralph D. Bollman WTP discharged wasted solids daily into Mallard Reservoir from October 1987 through December 1988. Taste and odor problems with the finished product water were controlled by the addition of PAC, and the waste carbon was discharged to the reservoir with the alum sludge. Sludge discharge to Mallard Reservoir ended in December 1988.

Water and sediment samples were collected from Mallard Reservoir at three stations during February 1989. Station 1 was located at the point of discharge, and Stations 2 and 3 were located in the opposite corners of the reservoir, outside the mixing zone. At the point of discharge, aluminum concentration in the water column above the sediments was 9.40 mg/L and averaged 0.36 mg/L within the bulk of the reservoir. The iron concentration was 2.80 mg/L at the point of discharge while an average iron concentration of 0.15 mg/L existed in the water column above the sediments in most of the impoundment.

Aluminum concentrations in the sediment (Figure 5-8) were much higher at discharge point (102,000 mg/kg dry weight) than at other locations within the reservoir (averaging 42,640 mg/kg). A thick layer of alum sludge containing PAC appeared to be distributed over much of the bottom surface of Mallard Reservoir.

Oligochaetes and amphipods dominated the benthic macroinvertebrate communities at all stations. The point of discharge had the highest number of taxa and also the highest species diversity (Table 5-7). At the point of discharge, eight taxa were identified and a Shannon species diversity of 1.90 was computed. The other monitoring stations within the reservoir had five taxa with a diversity of 1.16, and four taxa and a diversity of 0.62. Species diversity and evenness values were lower at Station 2 (500 organisms/m^2) and Station 3 (1,049 organisms/m^2), while the estimated number of organisms per square meter, i.e., the density at these stations was greater than at the sludge discharge location (113 organisms/m^2).

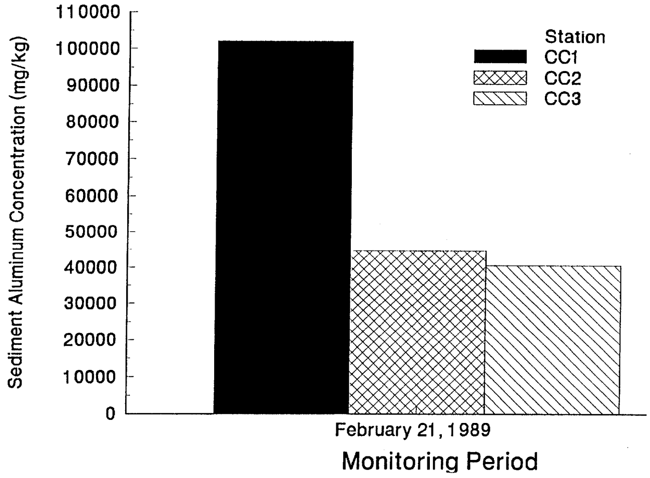


Figure 5-8. Sediment aluminum concentration from Mallard Reservoir, Concord, CA (George et al., 1991).

Impacted communities and stations usually exhibit a reduced Shannon diversity value (Washington, 1984), and values less than 3 indicate moderate to heavy pollution (Goodnight, 1973). Margalef species richness values were also highest at the discharge point (1.03) and lowest at Station 3 (0.030). The major difference between the benthic communities of the three sites was the number of oligochaetes collected. Oligochaetes composed 15 percent of the sample collected at Station 1, the point of discharge; 57 percent at Station 2; and 13 percent at Station 3. Fine particle, consolidated sediments apparently favored oligochaete assemblages at the expense of aquatic insects (Pennak, 1989).

The low density of organisms at the point of discharge (113 organisms/m^2) was indicative of moderate stress on the benthic macroinvertebrate community and was characteristic of a system subjected to toxic metal wastes (Cairns et al., 1971; Young et al., 1975). Relatively high numbers of oligochaetes in the sample from Station 2, along with a relatively low species diversity and evenness value, also indicate a probable negative impact on the ecological integrity of the benthic community (Washington, 1984; Cairns, 1977; Brinkhurst et al., 1968). The Simpson heterogeneity index indicated greatest species heterogeneity at the point of discharge (0.84), where the highest number of taxa (eight) were collected. Lowest heterogeneity (0.32) and lowest number of taxa (four) were found at Station 3.

In summary, data from the water and sediment field samples taken at three stations at Mallard Reservoir indicate moderate stress on benthic community structure, possibly resulting from alum sludge discharge and habitat constraints. High concentrations of alum were present in the water column and sediment of all stations, with the highest concentration found at the point of discharge. Assessment of benthic macroinvertebrate

Table 5-7. Benthic Macroinvertebrates Collected From Mallard Reservoir at Site CC, Contra Costa Water District, Concord, CA, February 21, 1989 (George et al., 1991)

Station	Number of Taxa	Organism Density (No. Organisms/m^2)	Shannon Diversity Index	Pielou Evenness Index	Margalef Richness Index	Simpson Heterogeneity Index
1	8	113	1.90	0.91	1.03	0.84
2	5	500	1.16	0.72	0.45	0.63
3	4	1,049	0.62	0.45	0.30	0.32

data supports the observation of a stressed community at all stations sampled.

5.3.3 Mobile Water Treatment Plant, Mobile, Alabama

5.3.3.1 Description

The Mobile Water Treatment Plant has a capacity of 158 m^3/min and an average daily drinking water production of 105 m^3/min. The raw water treated by the plant is taken from the Big Creek Lake, which is a 1,457 hectare (ha) reservoir. The water from the reservoir is very aggressive (Aggressive Index less than 10), with an average calcium concentration of only 2.9 mg $CaCO_3$/L, a bicarbonate concentration of 4 mg $CaCO_3$/L, and a pH of 6.2. Water is pumped from Big Creek Lake to a ground-level storage basin from which water is pumped to the plant. As water is transported from the storage basin to the plant, it is disinfected by chlorine dioxide. At the plant the water pH is raised by lime addition. Water treatment includes coagulation by alum (average dose 24.8 mg/L), flocculation, sedimentation, rapid sand filtration and postchlorination (Figure 5-9). Zinc orthophosphate is added for corrosion control. The average mass loading of aluminum to the system has been about 3.8 metric tons/day, and the average daily lime addition has been 2.4 metric tons.

Settled solids from the sedimentation basins are wasted daily for approximately 2.5 hours each morning at a rate of 13 m^3/min. The filters are backwashed over a 3-hour period every afternoon at a rate of approximately 2.1 m^3/min. The waste discharge goes into a small drainage ditch that enters Three Mile Creek, which flows through the city of Mobile, Alabama. At the time of the study, Three Mile Creek water was soft (approximately 15 mg $CaCO_3$/L hardness) and had low alkalinity, ranging from 12 to 20 mg $CaCO_3$/L. Aluminum in the Three Mile Creek water samples ranged from less than 0.02 to 0.19 mg/L.

5.3.3.2 Toxicity

The protozoan mortality test using *Tetrahymena* indicated no toxic response of the organism to extracts from the Mobile Water Treatment Plant waste sludge. Using the Microtox assay, however, sludge extracts had a 15-minute EC_{50} equal to 85 percent of the filtered sludge extract at pH 9. The predominant aluminum species at pH 9 was most likely the aluminate ion. The Microtox test showed no toxic effects to any of the remaining alum sludge extracts. Mobile's sludge extracts were significantly inhibitory to *S. capricornutum* at all concentrations of each pH level (Table 5-8).

5.3.3.3 Benthic Organisms

The brownish alum sludge discharge was clearly visible for approximately 400 meters downstream of the point of entry before mixing with the receiving water. During water and sediment monitoring in January 1989, sulfate concentrations in Three Mile Creek ranged from 2.7 mg/L upstream from the point of discharge to 18.7 mg/L at the point of discharge. TSS also followed this trend in January when concentrations ranged from 3.2 mg/L upstream (Station 4) to 185 mg/L at point of discharge (Station 3). Likewise, aluminum concentrations ranged from 1.21 mg/L at Station 4 to 5.26 mg/L at Station 3 in January.

While the aluminum concentration upstream from the point of discharge ranged from 1.21 (January) to 0.02

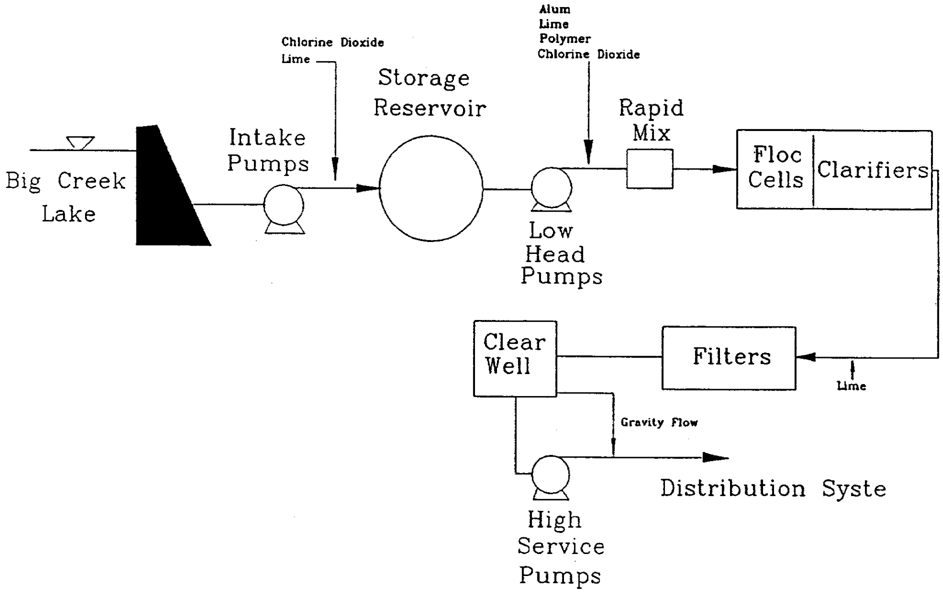


Figure 5-9. Flow schematic of Mobile WTP, Mobile, AL (George et al., 1991).

Table 5-8. *S. Capricornutum* Test Results on Alum Sludge Extracts From the Mobile, AL, Water Treatment Plant (George et al., 1991)

Sample Site pH	Percent Filtrate	Monomeric Aluminum Concentration (mg M-Al/L)	Percent Inhibition	EC_{50}
pH 6.7	12.5	0.04	18	37%
	25	0.08	27	—[a]
	50	0.17	62	—
	100	0.34	92	—
pH 8	12.5	0.04	30	19%
	25	0.08	63	—
	50	0.17	95	—
	100	0.34	99	—
pH 9	12.5	1.34	99.6	—
	25	2.69	100	—
	50	5.38	100	—
	100	10.76	100	—

[a] Insufficient data to compute EC_{50}.

Key

EC_{50} = Effective concentration producing a 50 percent reduction in growth rate compared with the growth rate of control populations.

mg/L (February), the downstream mean aluminum concentration in January was 12.6 mg/L, decreasing to 0.05 mg/L in February, and increasing to 1.09 mg/L in March. Downstream mean TSS concentrations were also highest in January 78.2 mg/L, dropping to 5.2 mg/L in February. Iron exhibited a similar trend, as downstream mean concentration was 2.59 mg/L in January, 0.30 mg/L in February, and 0.96 mg/L in March.

The data gathered suggest the possibility that deposition of aluminum from the plant's alum sludge occurred in sediment downstream from the point of discharge (Figure 5-10). Aluminum concentrations in the sediment at the downstream Station 2 (2,208 mg/kg) were higher than the concentrations found at the point of discharge (1,236 mg/kg).

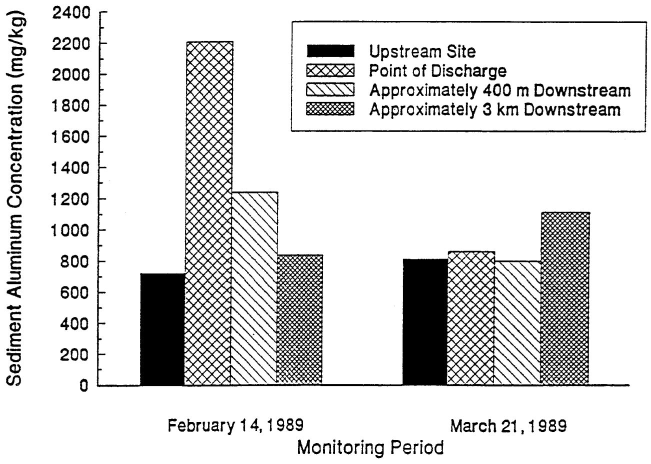


Figure 5-10. Sediment aluminum concentration from Three Mile Creek, Mobile, AL (George et al., 1991).

Benthic macroinvertebrate data are presented in Table 5-9. Oligochaetes and dipterans *(Chironomidae)* were the dominant taxa collected in Three Mile Creek, in addition to trichopterans *(Cheumatopsyche* sp.). The upstream monitoring station had the greatest number of taxa and the greatest number of organisms for two of the three sampling events. In February, the density of organisms ranged from 4 organisms/m^2 to 3,606 organisms/m^2 at Station 4. By March, the number of organisms per square meter ranged from 41 at the point of discharge to 3,641 at the upstream reference station. In January, the lowest number of organisms was recorded at the point of discharge. The alum sludge discharged by the Mobile Water Treatment Plant apparently had an adverse effect on the benthic macroinvertebrate density of Three Mile Creek at and below the point of discharge. A study by Lin (1989) also found an impact to benthic macroinvertebrates from alum sludge discharge.

In summary, the alum sludge discharged by the Mobile Water Treatment Plant appeared to have an adverse effect on the benthic macroinvertebrate community of the receiving stream at and below the point of discharge. Moderate recovery of the macroinvertebrate community appeared to have occurred approximately 1,200 meters below the point of discharge.

Table 5-9. Benthic Macroinvertebrates Collected From Three Mile Creek, Mobile, AL, 1989 (George et al., 1991)

Station	Number of Taxa	Organism Density (no. organisms/m^2)
Sampling Period #1 - January 19, 1989		
1 - ~3 km downstream location from point of discharge	6	1,570
2 - ~0.5 km downstream from point of discharge	6	54
3 - point of discharge	6	14
4 - upstream monitoring station	11	517
Sampling Period #2 - February 14, 1989		
1	9	529
2	8	57
3	22	4
4	21	3,606
Sampling Period #3 - March 21, 1989		
1	12	335
2	3	167
3	5	41
4	15	3,641

Note: Samples were collected using a Surber sampler and consisted of five replicates per station. The numbers of taxa and organisms are the summations of the numbers found in each replicate sample for each site.

5.3.4 City of Phoenix Utility, Phoenix, Arizona

5.3.4.1 Description

The Val Vista, Deer Valley, and Squaw Peak WTPs, which are owned and operated by the City of Phoenix, Arizona, directly discharge their waste residual solids into the Salt River Project (SRP) canal system (Figure 5-11). The SRP also serves as the raw water source for these WTPs. EPA has determined that discharges to the SRP canals, considered waters of the United States, are subject to NPDES permitting requirements (HDR, 1995).

Water in the SRP is a combination of the Salt and Verde Rivers. Furthermore, the SRP canal system is interconnected with water from the Central Arizona Project (CAP). The quality of raw water treated by the WTP varies according to the proportion of water received from each of the aforementioned sources. Table 5-10 presents the average raw water quality for each of these three sources.

Table 5-11 shows the flow rates and average turbidity data for each facility. Even though Deer Valley and Squaw Peak WTPs obtain raw water from the same source (the Arizona Canal), the average raw water turbidities at the Deer Valley WTP were higher than turbidity levels treated by the Squaw Peak plant. The higher raw water turbidity at the Deer Valley WTP has been attributed to the residuals solids discharged into the canal by the Squaw Peak WTP.

The three WTPs each use conventional treatment, employing grit removal, presedimentation, chemical addition, rapid mixing, flocculation, final sedimentation, filtration, and disinfection. Direct discharge of residuals solids into SRP was an acceptable practice at the time the facilities were constructed and, therefore, no solids handling facilities were constructed at any of the WTPs. The following residuals solids were in waste streams discharged to the canal system from the following water treatment processes:

- Raw water solids (usually without chemical addition) from grit and presedimentation basins.
- Alum residual solids from final sedimentation basins.
- Wasted washwater from filter backwashing and filter-to-waste flows (i.e., filtered water immediately after a filter is backwashed when turbidities exceed drinking water standards).
- Miscellaneous waste streams from leaking valves, basin and reservoir overflows, surface drainage, and sample sink return flows (no chemical addition except for chlorine residuals).

Currently, residuals solids from the three WTPs are being discharged directly into the canals without equali-

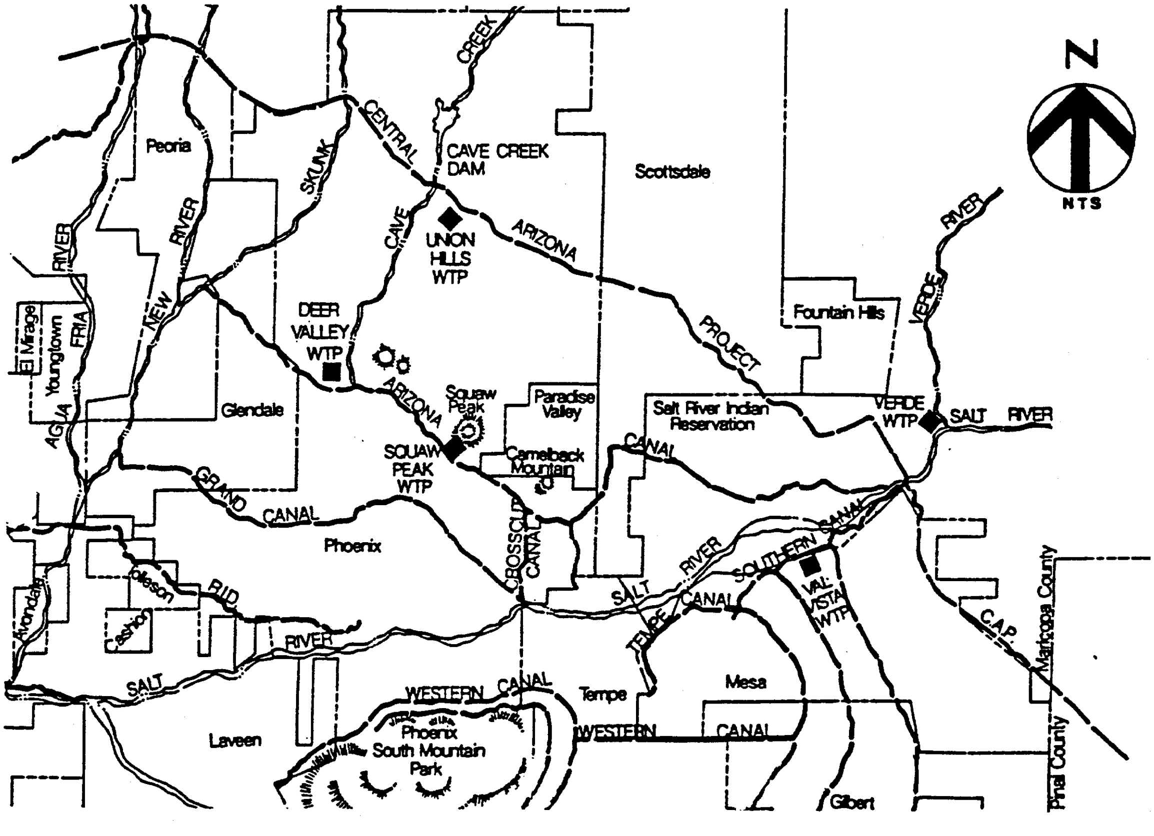


Figure 5-11. Location of City of Phoenix WTPs, Phoenix, AZ (HDR, 1995).

Table 5-10. Typical Canal Source Water Characteristics (HDR, 1995)

Parameter (mg/L unless noted otherwise)	Verde River Below Bartlett Dam, SRP Canal[a]	Salt River Below Stewart Mountain Dam, SRP Canal[a]	Colorado River Below Parker Dam, CAP Canal[a]
Arsenic	0.013	0.004	0.003
Barium	0.1	0.1	0.7[b]
Bicarbonate	235	160	155
Boron	0.19	0.19	0.19[b]
Cadmium	0.006	0.007	0.001
Calcium	43	50	82
Carbonate	2	0	0
Chloride	19	235	91
Chromium	0.004	0.002	0.006[b]
Copper	0.009	0.009	0.006[b]
Fecal coliform (cfu/mL)[f]	13	7	4
Fluoride	0.3	0.4	0.4
Hardness (as $CaCO_3$)	212	180	330
Hardness	19	49	203
Iron	0.19	0.19	0.13
Lead	0.07	0.07	0.01
Magnesium	26	14	30
Manganese	0.09	0.06	0.02
Mercury	0.00026	0.00004	0.00003
Nitrate (as N)	0.4	0.1	0.1[c]
pH (units)	8.0	7.7	8.0
Phosphorus	0.21	0.22	0.02
Potassium	3.4	5.8	5.1
Selenium	0.00006	0.00003	0.00003
Specific conductance	472	1,138	1,083
Silver	0.00001	0.0017	0.0001[b]
Sodium	30	161	104
Sulfate	53	51	294
Total dissolved solids	280	5,007	721
Total organic carbon	3.5[d]	3.5[d]	5.2[e]
Turbidity (NTU)[g]	58	11	4[c]
Zinc	0.04	0.033	0.02

[a] National Stream Quality Accounting Network (NASQAN) Period of Record, March 1975 to September 1979. Information on this water source is provided because CAP water can be released to the Salt River and be carried in the SRP canal.
[b] Period of Record, Water Year 1979 only.
[c] Combined nitrate-nitrite.
[d] Water Quality Master Plan.
[e] Tucson Water, Period of Record: October 1986 to August 1987.
[f] cfu/mL: colony forming units per milliliter.
[g] Turbidities can range from as low as 1 NTU to as high as 500+ NTUs.

zation or treatment. Characteristics of the waste streams from each of the WTPs are presented in Tables 5-12, 5-13, and 5-14. The solids mass loading discharged from each treatment process was computed based on the flows and total suspended solids data (Table 5-15). The Phoenix Water Service Department funded a study to determine the effects of these residuals solids on the hydraulics, water quality, and biological communities of the SRP canal system (HDR, 1995).

5.3.4.2 Effects of Discharged Residuals Solids on Canal Hydraulics and Sediment Load

Solids discharged by the Val Vista WTP to the South Canal are permitted through an agreement between the City and SRP. The City pays SRP's costs for dredging sediment from a specific length of canal downstream from each discharge point. These portions of the canal are drained and dried annually. During this time, the sediments are removed and placed on the canal banks to dry. In 1995, the annual cost to the City ranged from $15,000 to $50,000. If the discharged solids settle and are retained in the canal, the resulting sediments may affect the canal hydraulics.

The U.S. Army Corps of Engineers' HEC-2 computer models of canal physical and hydraulic characteristics were used to determine the impact of residuals solids on the SRP canal system. Computer simulations were compared with information from the SRP on water level profiles corresponding to specific flow rates in the canal. The SRP also included water surface elevations corresponding to design capacities within the Arizona Canal (19.8 m^3/sec at Station 38+862 to Station 73+963) and South Canal (41.1 m^3/sec at Station 0+000 to Station 25+700). The velocities in the canal system varied slightly but were generally about 94 cm/sec. The HEC-2 hydraulic analysis compared favorably with independently measured flow and cross-section data.

HDR (1995) evaluated the settling characteristics of the discharged solids to determine if they would settle and be retained in the canal system. According to Table 5-16, 81.4 percent of the solids measured were finer than a #200 sieve. Using the U.S. Army Corps of Engineers' HEC-6 computer program, transport of solids in the canals was assessed. HEC-6 uses the same input scheme as HEC-2 to describe the hydraulic and geometric characteristics of the canals. In addition, grain size distribution data (Table 5-16) and total volume of solids entering the canal are entered into the program. The model calculates the volume of each grain size that will settle out at each canal cross section. The canals being evaluated are divided into reaches and the model predicts the solids mass load for each of the reaches (Table 5-17).

Computer results indicate that the material finer than a #200 sieve (approximately 81 percent of the particles) would be transported through the canal system. These results were based on the assumption of maintaining a water velocity of 58 cm/sec or greater. Localized areas exist—eddies caused by intakes, delivery and dispersion structures, control structures, bridge piers, and

Table 5-11. Summary of Plant Flows and Turbidity Data (HDR, 1995)

	Average		Maximum		Peak	
WTP	Flow (m^3/min)	Turbidity (NTU)	Flow (m^3/min)	Turbidity (NTU)	Flow (m^3/min)	Turbidity (NTU)
Val Vista	315	11	368	20	368	40
Squaw Peak	184	10	368	20	368	40
Deer Valley	210	20	315	40	315	40

Table 5-12. Val Vista WTP Discharge Stream Characteristics (HDR, 1995)

	Discharge Stream					
Parameter	Raw Water	Grit Basin Discharge	Presed. Basin Blowdown	Final Sed. Basin Blowdown	Filter Waste Washwater	Leakage and Drains
Operational frequency	Continuous	Intermittent	1 hr/day	1–4 hr/day	4 times/day	Continuous
Flow (m^3/min)	315	0.68	0.63	3.4	7.9	5.0
Turibidity (NTU)	11	N/A	N/A	N/A	N/A	N/A
Total suspended solids (mg/L)	20	16	1,600	1,400	55	3
Total aluminum (mg/L)	<0.05	N/A	19	1,800	165	0.010

Note:
1. To be reduced under current plant modifications.
2. Storm drainage not included.

Table 5-13. Squaw Peak WTP Discharge Stream Characteristics (HDR, 1995)

	Discharge Stream				
Parameter	Raw Water	Presed. Basin Blowdown	Final Sed. Basin Blowdown	Filter Waste Washwater	Leakage
Operational frequency	Continuous	3 times/day	0.5 hr/day	3 times/day	Continuous
Flow (m^3/min)	184	1.6	1.3	1.3	1.3
Turbidity (NTU)	10	N/A	N/A	N/A	N/A
Total suspended solids (mg/L)	30	130	3,900	360	N/A
pH	7.8	N/A	N/A	N/A	N/A
Dissolved aluminum (mg/L)	0.2	0.2	0.2	0.6	N/A

Table 5-14. Deer Valley WTP Discharge Stream Characteristics (HDR, 1995)

	Discharge Stream					
Parameter	Raw Water	Grit Basin Discharge	Presed. Basin Blowdown	Final Sed. Basin Blowdown	Filter Waste Washwater	Leakage
Operational frequency	Continuous	3 times/year	3 times/week	2 times/week	6 times/day	Continuous
Flow	210 m^3/min	757 m^3/ discharge	2,200 m^3/ discharge	1,500 m^3/ discharge	2.4 m^3/min	5.3 m^3/min
Turbidity (NTU)	20	>1,000	>1,000	>1,000	71.25	4.4
Total suspended solids (mg/L)	39	4,900	21,000	9,400	264	14.5
Settleable solids (mg/L)	0.16	18	290	310	35	0.27
pH	8.4		7.65	7.83	7.45	N/A
Temperature (°C)	24		24	26	28	N/A
Total dissolved solids (mg/L)	732	760	801	813	731	740
Total aluminum (mg/L)	1.27	240	769	704	45.3	0.91

Table 5-15. Existing Discharge Quantities (HDR, 1995)

		Val Vista WTP			Squaw Peak WTP			Deer Valley WTP		
Discharge Stream	Plant Flow Cond	Flow (m^3/min)	Dur. (min)	Solids (kg/day)	Flow (m^3/min)	Dur. (min)	Solids (kg/day)	Flow (m^3/min)	Dur. (min)	Solids (kg/day)
Grit basin drain	Ave.	0.68	Cont.	27						
	Max.	0.68	Cont.	50	No grit basin drain			Intermittent discharge[a]		
	Peak	0.68	Cont.	100						
Presediment basin blowdown	Ave.	0.68	120	24,500	1.6	60	181	0.37	19	13,000
	Max.	0.63	120	4,500	1.6	60	726	0.37	20	44,000
	Peak	7.6	120	9,100	1.6	60	1,450	0.37	30	44,000
Final sediment basin blowdown	Ave.	3.4	120	12,700	1.3	60	10,400	0.47	31	3,900
	Max.	3.4	120	24,500	1.3	60	31,000	0.68	35	14,000
	Peak	6.6	120	48,100	2.6	60	62,000	0.68	35	14,000
Filter waste washwater	Ave.	12.6	8	1,700	1.3	10[b]	1,000	2.4	20[b]	900
	Max.	12.6	30[b]	3,300	5.3	15[b]	4,400	5.8	20[b]	3,200
	Peak	19.7	30[b]	6,400	10.0	15[b]	8,600	5.8	20[b]	3,200
Leakage and drains	Ave.	0.18	Cont.	1.4	Included in final sediment basin blowdown			5.3	cont.	15
	Max.	0.18	Cont.	2.3		5.3	Cont.			15
	Peak	0.18	Cont.	5.4		5.3	Cont.			15
Totals	Ave.	17.6		17,200	4.2		12,000	8.5		18,000
	Max.	17.6		32,200	8.1		36,000	12.1		61,000
	Peak	33.9		63,500	14.2		73,000	12.1		61,000

[a] Grit basin discharges are estimated to be 760 m^3 per event with a solids content of 3,600 kg. This discharge will be eliminated in upcoming modifications.

[b] Data obtained through telephone conversations with plant managers.

Table 5-16. Grain Size Distribution for WTP Residuals (HDR, 1995)

Sieve Size	Retained (%)	Passing (%)
#4	0	100
#8	2	98
#10	1	97
#16	4	93
#30	5	88
#40	2	86
#50	1	85
#100	2	83
#200	1	82
Pass #200		81.4

Table 5-17. Estimated Amount of Solids Deposited in the Canal System, Phoenix, AZ (HDR, 1995)

	Suspended Solids Load			
WTP	Entering Reach (mt/day)	Existing Reach (mt/day)	Solids Deposited in Reach (mt/year)	Length of Canal Reach (m)
Squaw Peak	24.5	22.4	762	10,700
Deer Valley	17.7	17.3	132	3,500
Val Vista	17.0	12.2	73	7,800

mossy areas—where the stream velocity may fall below 58 cm/sec and increased deposition might occur.

The City of Phoenix investigated the concentration of suspended solids in the Southern Canal following discharge of waste residuals solids from the Val Vista WTP. The sample collected upstream from the point of discharge was a grab sample, whereas the remaining samples were time-weighted composite samples. The City discovered that the TSS concentration at the point of discharge in the canal ranged from 200 mg/L (presedimentation blowdown) to 1,600 mg/L (final sedimentation blowdown). At 25 m downstream from the discharge point, 50 percent of the solids had been removed, and at 100 m downstream at least 70 percent of the solids had been removed from the water column. Analysis of suspended solids in water samples collected 500 m downstream indicated a TSS reduction of 78 to 98 percent and TSS levels 3 to 45 times the concentration measured upstream of the point of discharge.

5.3.4.3 Impact on Aquatic Organisms

The Arizona Game and Fish Department electrofished the Arizona Canal between October 1992 and April 1993. The results of the electrofishing are presented in Table 5-18. *Ctenopharyngodon idellus* (white amur or grass carp) has been used for the control of aquatic vegetation and, therefore, is probably the most economically important species.

Indigenous planktonic invertebrates were assumed to have originated from the Salt River reservoir system. Analysis of zooplankton in Canyon Lake by McNatt (1977) revealed that at least 15 cladoceran, 5 copepod, and 1 rotifer species may inhabit the rivers and canals. Among the cladocera identified were *Bosmina longirostris*, *Daphnia parvula*, *Daphnia pulex*, *Daphnia galeata*, *Daphnia ambigua*, *Diaphanosoma leuchtenbergianum*, *Ceriodaphnia lacustris*, *Chydorus sphaericus*, and *Ilyocryptus* sp. Copepod species found include *Paracyclops fimbriatus*, *Acanthocyclops vernalis*, *Diacyclops bicuspidatus*, and *Diaptomis siciliodes*. *Asplanchna priodonta* was the identified rotifer. In the Salt River system benthic communities, several species have been identified: *Chironomus*, *Cryptochironomus*, *Polypedium*, *Procladius*, *Tanypus*, two tubificid worms (*Branchiura* and *Limnodrilus*), the Asiatic clam (*Corbicula malinensis*), a crayfish (*Oroconectes virilus*) and two sponge species. HDR (1995) did not conduct a study of the phytoplankton present in the canal system. Based on another analysis of water in the SRP reservoirs (Olsen, 1975), however, HDR inferred that a reasonable representation of flora in the canals would include *Cyclotella*, *Stephanodiscus*, *Synedra*, *Spirulina*, *Anabaenopsis*, *Dinobryon*, *Glenodinium*, *Phacotus*, *Closteriopsis*, *Tetrastrum*, *Tetraedron*, and *Ankistrodesmus*.

The primary pollutants found in the WTP wastestreams that may adversely affect aquatic organisms were arsenic, barium, cadmium, chromium, mercury, selenium, and trihalomethanes. Barium is only toxic to freshwater aquatic life at levels greater than 50 mg/L (U.S. EPA, 1986c) and was not viewed as a problem by the consultants. Concentrations of the remaining pollutants in the discharged waste streams are presented in Table 5-19. An evaluation of the pollutants in the City of Phoenix's waste streams revealed that 85 to 98 percent of the metals discharged were in particulate form and would ultimately settle once the turbulence was reduced downstream (HDR, 1995). HDR (1995) used a simple mathematical model to estimate the concentration of pollutants in the water column downstream from the discharge points. The assumptions were: 1) complete mixing of the pollutants occurred in the volume of water passing the

Table 5-18. Total Number of Fish Caught by Electrofishing, Arizona Canal, Phoenix, AZ, October 15, 1992, to April 30, 1993 (HDR, 1995)

Common	Site #1	Site #2	Site #3	Site #5	Site #7	Total
Bluegill	10	0	0	0	0	10
Carp	1	1	1	1	1	5
Channel	5	35	1	2	20	63
Desert sucker	35	98	474	300	91	998
Goldfish	1	0	0	0	0	1
Largemouth	20	7	3	7	0	37
Mosquitofish	0	0	0	0	7	7
Rainbow trout	0	1	0	1	0	2
Red shiner	225	223	48	40	33	599
Roundtail	0	2	15	1	5	23
Smallmouth	1	0	0	0	0	1
Sonora sucker	82	157	467	571	518	1,795
Threadfin	579	97	29	62	33	800
White amur	97	4	15	12	5	133
Yellow bass	11	4	0	3	2	20
Yellow	2	14	3	0	0	19

Site 1: From 67th Ave. downstream to the Skunk Creek drain gates.
Site 2: From the corner of 43rd Ave. and Peoria, downstream to the corner of 51st Ave. and Cactus.
Site 3: From the 19th Ave. bridge downstream to the Black Canyon freeway bridge, which lies between Peoria and Dunlap.
Site 5: From the 68th Street bridge downstream to the water control structure located near the 56th Street bridge.
Site 7: From Pima Road bridge downstream to Hayden Road, which lies between Indian Bend Road and McDonald Drive.

discharge point by the end of a 24-hour period; and 2) no volume reduction was made to compensate for sedimentation. Using data in Table 5-19 to compute daily pollutant mass loadings, average and maximum daily pollutant contributions to the receiving stream were computed for typical flows in the Arizona Canal (213 m^3/sec) and Southern Canal (442 m^3/sec). HDR (1995) also computed the pollutant concentration at a low flow condition of 25 percent of the typical flow. The results of these computations are presented in Tables 5-20 and 5-21.

Under maximum mass discharge conditions and typical canal flow, the computed aluminum concentration in the receiving water ranged from 1 mg/L (Val Vista WTP) to 45 mg/L (Deer Valley WTP). Estimated aluminum concentration during low flow would be 179 mg/L below the Deer Valley point of discharge. Deer Valley's residuals solids would produce the highest concentration of metals in the canal water column (see Tables 5-20 and 5-21). The estimated increase in arsenic, cadmium, chromium, and selenium in the water downstream from Squaw Peak WTP would be less than 1 μg/L.

Metal Toxicity

Environmental factors may affect aluminum toxicity to aquatic organisms. High calcium concentrations protect against the effect of aluminum on the ion regulatory mechanism of rainbow trout (McDonald et al., 1983; Muniz and Leivestad, 1980), brown trout (Wood and McDonald, 1987) and *Daphnia magna* (Havas, 1985). Furthermore, natural organic compounds can complex with aluminum and protect aquatic organisms (Birchall et al., 1989; Driscoll et al., 1980; Karlson-Norrgren et al., 1986). Increases in hardness and alkalinity reduce the

Table 5-19. Estimated WTP Discharge Stream Pollutant Concentrations (HDR, 1995)

WTP	Total Discharge (m^3/min)	TSS (mg/L)	Al (mg/L)	As (mg/L)	Cd (mg/L)	Cr (mg/L)	Hg (mg/L)	Se (mg/L)	THM (mg/L)
Squaw Peak									
Presedimentation									
Average	0.05	8,400	0.2	0.015	0.005	0.01	0.0002	0.005	0.100
Maximum	0.18	9,400	0.2	0.015	0.005	0.01	0.0002	0.005	0.100
Final sedimentation									
Average	2.79	5,000	840	0.005	0.005	0.01	0.0002	0.005	0.136
Maximum	10.75	5,000	406	0.005	0.005	0.01	0.0002	0.005	0.136
Filter back wash									
Average	4.73	190	8	0.010	0.005	0.01	0.0002	0.005	0.100
Maximum	18.92	280	11	0.010	0.005	0.01	0.0002	0.005	0.100
Deer Valley									
Presedimentation									
Average	0.03	14,400	0.2	0.428	0.036	0.30	0.0045	0.005	0.026
Maximum	0.16	9,400	0.2	0.428	0.036	0.30	0.0045	0.005	0.026
Final sedimentation									
Average	2.37	4,700	560	0.478	0.028	0.34	0.0012	0.029	0.126
Maximum	9.12	4,700	560	0.478	0.028	0.34	0.0012	0.029	0.126
Filter back wash									
Average	4.21	170	14	0.056	0.005	0.01	0.0002	0.005	0.061
Maximum	15.77	280	14	0.056	0.005	0.01	0.0002	0.005	0.061
Val Vista									
Presedimentation									
Average	0.05	7,800	0.2	0.005	0.005	0.01	0.0002	0.005	0.100
Maximum	0.16	7,800	0.2	0.005	0.005	0.01	0.0002	0.005	0.100
Final sedimentation									
Average	2.68	4,800	214	1.170	0.005	0.07	0.0002	0.005	0.220
Maximum	9.59	5,000	300	1.600	0.005	0.10	0.0002	0.005	0.300
Filter back wash									
Average	8.67	100	5	0.005	0.005	0.01	0.0002	0.005	0.100
Maximum	23.13	150	16	0.005	0.005	0.01	0.0002	0.005	0.100

Table 5-20. Daily Resultant Pollutant Concentration Contribution to Canal at Typical Flow, Phoenix, AZ (HDR, 1995)

				Resultant Concentration Contribution (µg/L)					
Squaw Peak	**Canal Flow (m^3/s)**	**TSS (mg/L)**	**Al (mg/L)**	**As**	**Cd**	**Cr**	**Hg**	**Se**	**THM**
Average	213	13	2	0.05	0.03	0.06	0.001	0.03	0.75
Maximum	213	51	4	0.20	0.12	0.25	0.005	0.12	2.83

				Resultant Concentration Contribution (µg/L)					
Deer Valley	**Canal Flow (m^3/s)**	**TSS (mg/L)**	**Al (mg/L)**	**As**	**Cd**	**Cr**	**Hg**	**Se**	**THM**
Average	213	10	1	1.20	0.1	7.2	0.03	0.8	4.7
Maximum	213	41	45	46	2.9	28	0.13	2.9	18

				Resultant Concentration Contribution (µg/L)					
Val Vista	**Canal Flow (m^3/s)**	**TSS (mg/L)**	**Al (mg/L)**	**As**	**Cd**	**Cr**	**Hg**	**Se**	**THM**
Average	442	6	0	2	0.0	0.1	0.001	0.0	0.6
Maximum	442	21	1	6.3	0.1	0.4	0.003	0.1	2.1

Table 5-21. Daily Resultant Pollutant Concentration Contribution to Canal at 25% of Typical Flow, Phoenix, AZ (HDR, 1995)

				Resultant Concentration Contribution (µg/L)					
Squaw Peak	**Canal Flow (m^3/s)**	**TSS (mg/L)**	**Al (mg/L)**	**As**	**Cd**	**Cr**	**Hg**	**Se**	**THM**
Average	53	51	8	0.20	0.12	0.26	0.005	0.12	3
Maximum	53	204	16	0.80	0.50	1.00	0.020	0.50	11

				Resultant Concentration Contribution (µg/L)					
Deer Valley	**Canal Flow (m^3/s)**	**TSS (mg/L)**	**Al (mg/L)**	**As**	**Cd**	**Cr**	**Hg**	**Se**	**THM**
Average	53	41	5.0	4.6	0.3	29	0.11	3.1	18
Maximum	53	164	179	186	12	111	0.5	12	72

				Resultant Concentration Contribution (µg/L)					
Val Vista	**Canal Flow (m^3/s)**	**TSS (mg/L)**	**Al (mg/L)**	**As**	**Cd**	**Cr**	**Hg**	**Se**	**THM**
Average	110	23	1.0	7.1	0.09	0.38	0.004	0.09	2.4
Maximum	110	86	5.3	25.3	0.27	1.76	0.011	0.26	8.5

degree of toxicity of alum sludge in water (George et al., 1991). HDR (1995) concluded that with the neutral pH levels, high alkalinity (130 to 185 mg $CaCO_3$/L) and hardness (180 to 212 mg $CaCO_3$/L) of the receiving water, aluminum toxicity to aquatic organism in canal water should be minimized or eliminated.

Table 5-22 presents acute arsenic toxicity data for several aquatic organisms. Estimated arsenic levels in the canal water column downstream from a WTP discharge were less than 0.2 mg/L. Arsenic does not appear to bioaccumulate in fish (U.S. EPA, 1986c). Based on the low bioaccumulation potential, short half-life of arsenic, and the toxicity data, HDR (1995) concluded that arsenic toxicity should not be a problem with aquatic organisms.

Cadmium can bioaccumulate and cause damage to fish vital organs (Kay, 1986). Acute exposure to cadmium can result in respiratory failure caused by damage to the gills. Data on toxicity of cadmium to aquatic organisms are presented in Table 5-23. As water hardness, alkalinity, salinity, temperature, and organic matter increase, the toxicity of cadmium to aquatic organisms decreases (Sorensen, 1991).

Hardness has a similar effect on chromium toxicity to aquatic organisms (Mance, 1987; U.S. EPA, 1986c).

Table 5-22. Toxicity of Arsenic to Freshwater Organisms (HDR, 1995)

Species	Life Stage	Hardness (mg/L)	pH (SU)	Concentration (mg/L)	Duration (days)	Effect
Oncorhyncus mykiss	Eggs	—	—	0.1	180	No effect migration
	Adult	—	—	13.3	6	LC_{50}
	2 months	13.2	8.3	1	21	Reduced growth
	Parr	385	8	13.2–19.4	6	LC_{50}
	Adult	45	7.3	<0.09	28	No mortality
Ictalurus lacustris	Adult	—	—	3,050	4	LC_{50}
Lepomis macrochirus	Immature	310	7.6	0.2–0.7	112	Growth unaffected
	Adult	310	7.6	0.02–2.3	112	Growth/survival unaffected
Micropterus dolomieu	Fingerling	—	—	900	4	LC_{50}
Pimephales promelas	Fry	47	7.2	5–19	1–4	No mortality
	Egg	47	7.8	<16.5	29	No effect hatching
	Egg	47	7.8	<2.13	29	No effect survival/growth
	4 cm	48	7.2	135	4	LC_{50}
Crustacea						
Bosmina longirostris	1 day	120	6.8	0.85	4	EC_{50}
Cyclops (mixed spp)	Nauplii	139	7.6–8.8	1.6–10	14	<20% mortality
Daphnia magna		—	—	0.8–31	1–4	LC_{50}
	1 day	47	7.2–8.1	1.32	28	50% mortality, no young, growth reduced 10%
	1 day	47	7.2–8.1	0.6–1.8	1–4	LC_{50}
Daphnia pulex	1 day	120	6.1–7.8	49.6	2	EC_{50}
Gammarus pseudolimnaeus	Adult	47	7.2–8.1	0.8–2.0	2–4	LC_{50}
	Adult	44	6.9–7.3	0.09–0.96	14	10–100% mortality
Mollusca						
Aplexa hyorumpn	Adult	49.5	7.4–7.7	24.5	4	LC_{50}
Helisoma companulata	Adult	44	6.9–7.3	0.85–0.97	28	No mortality
Insecta						
Pieronarcys dorsata	Larva	44	6.9–7.3	0.85–0.97	28	No mortality
Tenytarsus dissimilis	Larva	47	7.2–7.7	97	2	LC_{50}
Rotifera						
Philodina roseola		—	—	5.0–18	4	LC_{50}
Benthic fauna (mixed)		—	0.5–3.0	112		50–90% reduction in numbers

Key
LC_{50} = Lethal concentration of substance causing mortality in 50 percent of the population exposed to the substance.

Chromium is more toxic in soft water. Chronic chromium toxicity to *D. magna* was reported at 66 µg/L in soft water, whereas 44 µg/L inhibited reproduction in hard water. Fish are more susceptible to inhibitory effects of chromium at low pH. Table 5-24 presents data on toxicity of chromium to freshwater organisms.

Sorensen (1991) states that maximum selenium levels for fish should not exceed 3 to 8 µg/L. Inorganic selenate has been shown to be acutely toxic to aquatic organisms at levels as low as 760 µg/L (U.S. EPA, 1986c). HDR (1995) indicated that no chronic toxicity data were available on selenium effects on freshwater organisms. Toxicity effects of selenium vary according to environmental conditions such as differences in oxygen tension, habitat, food availability, trophic level of fish, chemical form, fish age, dietary consumption, population composition and competition, presence of other metals, and thermal gradient (Sorensen, 1991). A summary of selenium toxicity data is presented in Table 5-25.

Table 5-23. Toxicity of Cadmium to Freshwater Organisms (HDR, 1995)

Species	Life Stage	Hardness (mg/L)	pH (SU)	Concentration (mg/L)	Duration (days)	Effect
Oncorhyncus mykiss	Eggs	80	7.9	>0.124	13	No effect migration
	Alevins	80	7.9	>0.124	27	LC_{50}
	Fry	80	7.2	0.091–0.677	2	Reduced growth
	Juvenile	125	7.8	0.01–0.03	7–10	LC_{50}
	Adult	82	7.8	0.006–0.40	4–14	No mortality
Castostomus commersoni	Adult	18	6.3	0.95–6.7	1–4	LC_{50}
	Embryo	—	—	>0.012		Reduced growth
Cyprinus carpio	Adult	55	8	0.24–0.45	1–4	LC_{50}
Lepomis macrochirus	Yearling	200	7.2	>0.08	330	90% survival
	Yearling	200	7.2	0.757–2.14	270	100% mortality
	Egg/Larvae	200	7.2	0.031–0.80	330	No effect
	Adult	200	7.2	>0.757	6	No spawning
	Larvae	200	7.2	0.031	60	18% mortality
	Larvae	200	7.2	0.08	30	70% mortality
Pimephales promelas	Immature	201	7.2	<0.057	30	No effect survival/growth
	Immature	201	7.2	0.068–0.150	270	LC_{50}
	Fry	204	7.6	<0.057	30	No effect survival/growth
	Eggs	204	7.6	<0.027	180	Hatchability
	Eggs	204	7.6	<0.037	180	Spawning unaffected
Crustacea						
Cyclops abyssorum	Adult	50	7.2	3.8	2	LC_{50}
Daphnia magna	1 day	130	6.95	0.005–0.008	2–4	LC_{50}
	Mature	—	7.8–8.6	>0.5	1–3	100% mortality, <25°C
	Female	—	7.8–8.6	<0.025	1–3	No mortality, @ 30°C
Daphnia pulex	1 day	106	8.5–8.6	0.115	1	LC_{50}
				0.015	70	Brood size reduced
				0.005	58	Survival reduced
Eudiaptomus padanus	Adult	50	7.2	0.55	2	LC_{50}
Gammarus pulex	Adult	40	—	0.12–0.68	2–4	LC_{50}
Cyclops (mixed sp.)	Nauplii	139	7.6–8.8	0.04	14	100% survival
				0.016	14	54% mortality
				0.4	14	73% mortality
Annelida						
Tubifex tubifex	Adult	224	8.5	450	1	LC_{50}, 28°C
		224	7	320	2	LC_{50}, 10°C
Limnodrilus hoffmeisteri	Adult	224	7	0.2–0.55	4	LC_{50}
Mollusca						
Physa integra	Adult	46	7.1–7.7	0.02–0.114	7–21	LC_{50}
Lymnaea stagnalis	Adult	soft	—	0.11–0.33	7–14	LC_{50}
Insecta						
Ephemerella subvaria	Larva	54	7	2	4	LC_{50}
Ephemerella spp.	Larva	46	7.1–7.7	0.085	21	No mortality
Hydropsyche betteni	Larva	46	7.1–7.7	0.238	28	No mortality
Chironomus tendipes	Larva	220	7	25	2	LC_{50}

Key
LC_{50} = Lethal concentration of substance causing mortality in 50 percent of the population exposed to the substance.

Table 5-24. Toxicity of Chromium to Freshwater Organisms (HDR, 1995)

Species	Life Stage	Hardness (mg/L)	pH (SU)	Concentration (mg/L)	Duration (days)	Effect
Oncorhyncus mykiss	Eggs	80	7.8	0.02–2.0	9	Unaffected
	Juvenile	80	7.8	2	224	50–100% mortality
	Alevins	80	7.8	0.02–2.0	224	Unaffected growth/survival
	Alevins	80	7.8	0.2–2.0	224	50–100% mortality
	Fry	70	7.8	0.013	60	Reduced growth
	Fry	70	—	0.017	60	94% mortality
	Adult	70	—	0.077–0.170	60	Unaffected growth/survival
	Adult	—	—	8.88–32	4	LC_{50}
	4–9 months	70	—	7.0–180	4	LC_{50}
	Fingerling	80	7.8	2.0–50	4	30–70% mortality
Castostomus commersoni	Egg/Fry	35	—	>0.29	60	Reduced growth
Cyprinus carpio	Adult	55	8	14.3–21.2	1–4	LC_{50}
Lepomis macrochirus	Egg/Fry	35	—	1.122	60	Reduced growth
	Adult	35	—	<0.522	60	No effect
Pimephales promelas	Immature	209	7.5	27–58	25	No effect survival/growth
	11 weeks	209	7.5	18.0–140	1–11	LC_{50}
	Juvenile	209	7.5	1–3	400	No effect spawning/hatching
	Adult	270	7.6	38–61	2–7	LC_{50}
Crustacea						
Cyclops abyssorum	Adult	50	7.2	10	2	LC_{50}
Daphnia magna	1 day	50	7.8	1.8	1	LC_{50}
Eudiaptomus padanus	Adult	50	7.2	10.1	2	LC_{50}
	Adult	50	7.2	0.55	2	LC_{50}
Gammarus pseudolimnaeus	Adult	45	—	0.067	4	LC_{50}
Gammarus sp.	Adult	50	—	3.2	4	LC_{50}
Annelida						
Nais sp.	—	50	—	9.3	4	LC_{50}
Mollusca						
Lymnaea emarginata	Adult	154	—	34.8	2	LC_{50}
Physa integra	Adult	154	—	0.66	2	LC_{50}
Insecta						
Ephemerella subvaria	Larva	50	6	2	4	LC_{50}
Hydropsyche betteni	Larva	42	—	64	4	LC_{50}
Tanytarsus dissimilis	—	44	—	59.9	4	LC_{50}
Rotifera						
Philodina acuticornis	—	25	—	3	4	LC_{50}
	—	81	—	15	4	LC_{50}

Key
LC_{50} = Lethal concentration of substance causing mortality in 50 percent of the population exposed to the substance.

Mercury (Hg) can accumulate in tissue; concentrations as high as 27.8 mg/L have been measured in fish muscle (Sorensen, 1991). In general, mercury levels greater than 1 µg/L have been toxic to freshwater organisms (Table 5-26). Chronic mercury toxicity to fathead minnows in their early life stage has occurred at 0.26 µg/L. The estimated mercury levels in the canal water column were less than 1 µg/L and were generally less than 0.2 µg/L, except for Deer Valley during minimum flow conditions (0.5 µg Hg/L) (see Table 5-21).

Table 5-25. Toxicity of Selenium to Freshwater Organisms (HDR, 1995)

Species	Life Stage	Hardness (mg/L)	pH (SU)	Concentration (mg/L)	Duration (days)	Effect
Oncorhyncus mykiss	Juvenile	135	7.9	5.41	4	LC$_{50}$ daily feed
	Uneyed eggs			0.47 and less	42	No mortality
	Fingerlings			0.47 and less	42	No effect Mortality/growth
	Adult	36		12.5	4	LC$_{50}$
Carassius auratus	Juvenile	148	7.6	36.6	4	LC$_{50}$
				8.8	14	LC$_{50}$
Cyprinus carpio	3–26 g	—	7.2–8.2	35	4	LC$_{50}$
Ictalurus punctatus	Juvenile	140	7.9	19.1	4	LC$_{50}$
Lepomis macrochirus	Juvenile	150	7.75	30.7	7	LC$_{50}$
				17.6	14	LC$_{50}$
Micropterus salmoides	Juvenile	25 & 200	6.0 or 8.7	0.01	120	No mortality
Pimephales promelas	Juvenile	151	7.8	7.3	4	LC$_{50}$
				2.9	7	LC$_{50}$
Crustacea						
Daphnia magna	1 day	140	8.2	1.87	7	LC$_{50}$
				1.5	32	Reproduction unaffected
				1	32	Growth unaffected
Daphnia pulex	1 day	46	7.4	0.6	28	No effect
	Adult	46	7.4	1.374	2	LC$_{50}$

Key

LC$_{50}$ = Lethal concentration of substance causing mortality in 50 percent of the population exposed to the substance.

Combinations of dissolved metals in the aquatic environment may have antagonistic, synergistic, and additive effects on organisms (Esernick et al., 1991; Goss and Wood, 1988; Spehar and Fiandt, 1986; Mukhopadhayay and Konar, 1985; Kaverkamp et al., 1983; Kaviraj and Konar, 1983; Suffern, 1981; Westerman and Birge, 1978). These effects will vary with respect to environmental conditions and exposed indigenous organisms.

The State of Arizona and EPA acute and chronic water quality standards for heavy metals to protect aquatic life are presented in Table 5-27. Based on these standards and the source water quality data presented in Table 5-10, cadmium (6 to 7 µg/L) and mercury (0.04 to 0.26 µg/L) levels in the Verde and Salt Rivers exceed chronic standards. Contribution of cadmium (12 µg/L) from the Deer Valley WTP may cause the levels in the canal to exceed state and federal acute toxicity standards, 5.6 µg/L and 3.9 µg/L, respectively.

Suspended Solids

HDR (1995) states that sedimentation of residuals solids may affect benthic communities, algae, aquatic macrophytes, and suitability of fish spawning habitat. The European Fisheries Advisory Commission states that salmonids will not spawn in areas of high solids deposition (EIFAC, 1969). Siltation shortly after spawning may affect the proper transfer of oxygen and carbon dioxide between the egg and adjacent water and, thus, result in mortality (EIFAC, 1969; U.S. EPA, 1986c).

Deposition of solids can also smother benthic organisms (Gammon, 1970; Tebo, 1965; George et al., 1991). Furthermore, alum sludge sediments may adversely affect benthic macroinvertebrates by limiting the availability of carbon (George et al., 1991). Deposition of solids can also reduce species diversity of the benthic organisms (Mackenthun, 1973; George et al., 1991). As previously mentioned, the deposition of residuals solids depends on stream velocity. George et al. (1991) found that deposition rates could be reduced and, therefore, the detrimental effects of alum sludge deposits to benthic communities could be minimized by discharging residuals solids during periods of high stream velocities.

HDR (1995) concludes that any effects of WTP residuals deposition on fish spawning would be considered negligible to the City, since the white amur, the most economically valuable species, is sterile and will not spawn anyway. Arizona's surface waters must comply with the state Water Quality Standards for Navigable Waters, which states that "navigable waters shall be free from pollutants in amounts or combinations that settle to form deposits that adversely affect aquatic life, impair recreational uses, or are unsightly." Discharge of residuals

Table 5-26. Toxicity of Mercury to Freshwater Organisms (HDR, 1995)

Species	Life Stage	Hardness (mg/L)	pH (SU)	Concentration (mg/L)	Duration (days)	Effect
Oncorhyncus mykiss	2 month	82–132	6.4–8.3	0.33	4	LC_{50}
Carassius auratus	Adult	100	—	0.35	2	LC_{50}
Channa punctatus	55–g	160	7.4	1.8	4	LC_{50}
Cyprinus carpio	Adult	55	8	0.18	4	LC_{50}
Lepomis gibbosus	Adult	55	8	0.3	4	LC_{50}
Lepomis macrochirus	0.6 g	46	7.1–7.3	0.16	4	LC_{50}
Pimephales promelas	3.2–4.2 cm	40–48	7.2–7.9	1–Jan	4	LC_{50}
Tilapia mossambica	Adult	115	8.5	1	14	LC_{50}
Annelida						
Limnodrilus hoffmeisteri	Adult	5	7	0.15–0.50	4	LC_{50}
Tubifex tubifex	Adult	5	7	0.14–0.27	4	LC_{50}
Planaria						
Dugesia dorotocephala	Adult	—	—	0.2	10	no mortality
Crustacea						
Cyclops (mixed sp.)	Nauplii	139	7.6–8.8	<0.032	14	100% survival
Cyclops abyssorum	Adult	50	7.2	2.2	2	LC_{50}
Eudiaptomus padanus	Adult	50	7.2	0.85	2	LC_{50}
Mollusca						
Aplexa hypnorum	Adult	50	7.2–7.4	0.37	4	LC_{50}
Insecta						
Chironomus tendipes	Larva	220	7	64	1	LC_{50}
Ephemerella subvaria	Larva	42	7.6	2	4	LC_{50}
Hydropsyche betteni	Larva	42	7.6	2	4	LC_{50}

Key
LC_{50} = Lethal concentration of substance causing mortality in 50 percent of the population exposed to the substance.

Table 5-27. Maximum Contaminant Levels (μg/L) Cold Water Fishery (HDR, 1995)

	Acute Standard		Chronic Standard	
Contaminant	State	EPA	State	EPA
Arsenic (total)	360	360	190	190
Cadmium (total)	5.6[a]	3.9	1.5	1.1
Chromium III (total)	2,261	1,700	269	210
Mercury (dissolved)	2.4	2.4	0.01	0.012
Selenium (dissolved)	20.0	260	2.0	35
TTHM	NNS	11,000	NNS	NNS

[a] Value for warm water fishery is 34.6.
Key
NNS = No numerical standard.

solids may violate state surface water quality standards by reducing benthic community populations and diversity, and destroying suitable spawning beds (HDR, 1995).

5.3.4.4 Conclusions

After evaluating the available data, HDR (1995) concludes that no adverse effects to aquatic organisms are anticipated from metals (aluminum, arsenic, cadmium, chromium, selenium, and mercury) contained in WTP waste stream discharges. If low flow conditions (25 percent of typical flow) are maintained, cadmium concentrations in the canal downstream from the Deer Valley WTP may exceed state and EPA water quality standards. The hardness, circumneutral pH, and high alkalinity of the waters in the SRP canal system would help reduce any potential inhibitory effects from metals on aquatic life. Solids deposition may affect benthic communities and eliminate spawning areas and the successful development of fish eggs. HDR (1995) further states that the impacts may not be important unless the Arizona Game and Fish Department attempt to create a sustainable urban fishery in the canals.

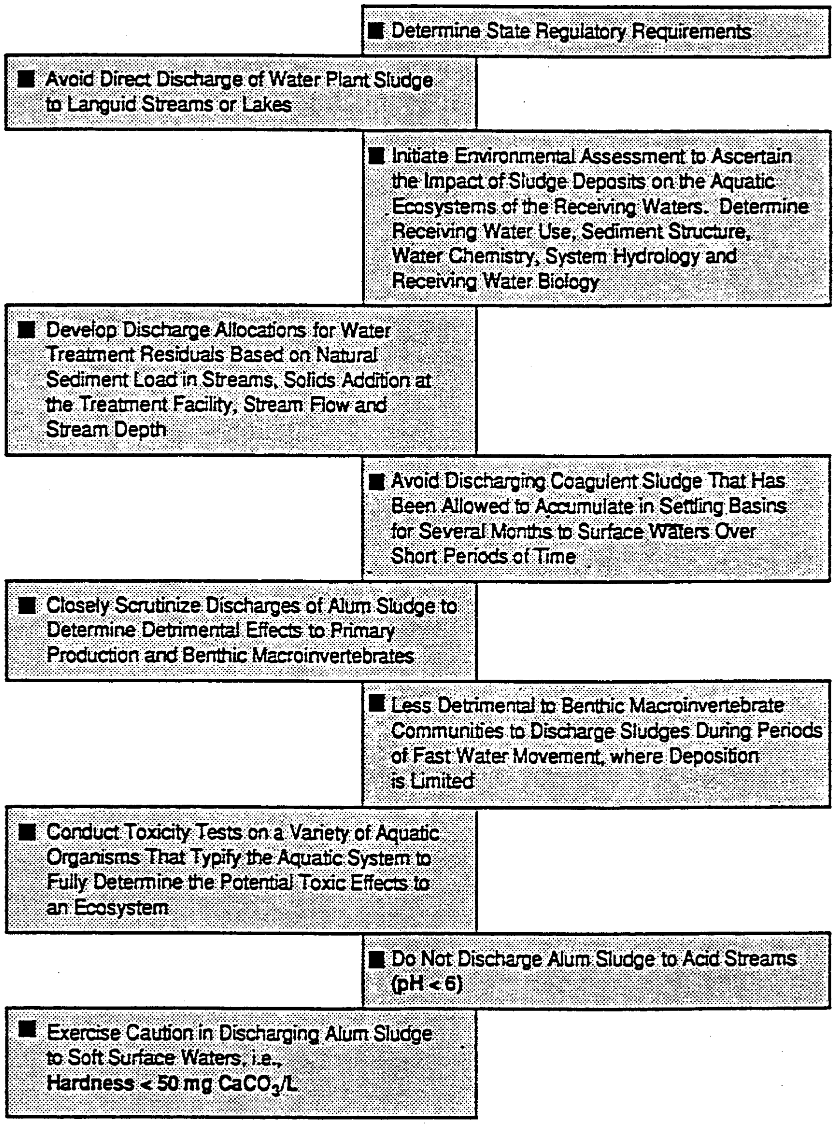


Figure 5-12. Recommended practices for direct discharge of WTP residuals to surface waters.

5.4 Recommended Practices

Limited information exists regarding the impact of direct discharge of WTP residuals solids on aquatic environments. Regulatory agencies have permitted the direct discharge of residuals to streams and impoundments because of the absence of information showing any adverse effects. Some states have taken a more conservative approach recently and have discontinued allowing discharge of residuals solids to freshwater. The first step, therefore, by a utility considering direct discharge of WTP residuals into a receiving stream is to determine its state regulatory requirements (see Figure 5-12).

The scientific literature and examples discussed in this chapter show that deposition of residuals solids can adversely affect benthic community populations and diversity. These direct consequences on benthic organisms may limit a food source for certain fish species. In addition, solids deposition may affect fish egg survival. Discharge of WTP residuals in sluggish streams or lakes should be avoided. A utility can use hydrologic models such as the U.S. Army Corps of Engineers' HEC-2 or HEC-6 models to estimate possible deposition within a receiving stream. More frequent, reduced solids mass loading from the WTP during high stream flow will possibly reduce deposition and adverse effects to downstream benthic organisms.

The receiving water chemistry can minimize toxic effects of metals in the discharged waste stream. Hardness, alkalinity, pH, dissolved oxygen, sulfate, and other water quality parameters minimize the inhibitory effects of heavy metals. Because of the potential toxicity to aquatic organisms, utilities that are evaluating discharge to soft waters (hardness less than 50 mg $CaCO_3$/L) should consider an alternate residuals disposal method. Furthermore, receiving waters with a pH less than 6 should be avoided. Discharge to these waters could result in increased solubility of metals and increased toxic effects. The chemical characteristics of the waste streams must be determined as well as the contaminant mass loadings to the receiving stream. Using information on stream hydrology and stream and waste quality, a mass balance should be conducted to estimate potential concentrations of contaminants downstream from a prospective point of discharge. Consideration of mixing zone should be included in modeling the system.

When considering the future use of a receiving water, a utility should consider whether direct discharge of its residuals will negatively affect the downstream aquatic life or quality. As part of this consideration, the utility should evaluate upstream and downstream water quality, sediments, and aquatic life to determine if the water has already been negatively affected by some previous use. To fully evaluate the toxicity of the WTPs discharged residuals, a series of toxicity tests should be conducted on a variety of aquatic organisms that are indicative of the organisms inhabiting the aquatic ecosystem. As shown in the case studies, one toxicity test may not adequately describe toxic effects.

Residuals from WTPs using aluminum salts have been the focus of research on the impacts of direct discharge of wastestreams to freshwater aquatic ecosystems. The recommended protocol presented in Figure 5-12, however, can be followed for any type of discharge. Aluminum may not be inhibiting organism growth or survival; other metals or contaminants contained in wasted residuals may be the primary toxicant. Detrimental effects of solids deposition on benthic organisms can be caused by limiting oxygen transfer to chemical toxicity. In any case, sedimentation of solids must be evaluated and anticipated. Other chemical residuals can affect benthic communities and fish spawning areas. In addition, chemical characteristics and quantities of the receiving water and the wastestreams must be determined to anticipate chemical changes to the water downstream from the points of discharge. Toxicity testing must be conducted to determine potential toxic effects on the ecosystem.

Chapter 6
Discharge to Wastewater Treatment Plants

Discharge of water treatment plant (WTP) residuals to a wastewater treatment plant (WWTP) is an attractive disposal option for water utilities. This chapter provides information on the experience of utilities using this practice, factors that should be considered, regulatory limitations, and design considerations for conveyance facilities (both gravity and pumped systems) and for the receiving WWTPs. The impacts on options for ultimate disposal of residuals from WWTPs are also considered.

6.1 Background

Many water utilities across the country discharge WTP residuals to a sewer and/or to a WWTP. These options are often economically attractive, and transfer disposal liability to the WWTP. The American Water Works Association (AWWA) Water Industry Database (AWWA/AWWARF, 1992), a survey of WTPs currently discharging to a WWTP, reveals that such plants are variable in size, and typically treat surface water supplies as opposed to ground waters. Table 6-1 summarizes the facilities surveyed.

Conventional WTPs (coagulation, sedimentation, filtration) commonly discharge filter backwash solids and/or clarification basin residuals to a sanitary sewer system for eventual treatment at a WWTP.

Several factors must be considered when evaluating the feasibility of discharging residuals to a WWTP. The interests and concerns of WWTP managers and operators are different from those of similar personnel in water utilities. Factors for a WWTP to consider are available capacity (conveyance system and treatment plant), treatment process compatibility, and final disposal requirements. Introducing water treatment residuals to a WWTP may offer some benefits in terms of process performance. On the water treatment facility side, pretreatment requirements, storage facilities, and conveyance systems must be considered. Costs and service agreement terms must also be evaluated.

Regulatory requirements, especially those related to ultimate disposal, must also be addressed. Essentially, all states allow for discharge of WTP residuals (sludge and brines, or reject waters) to a WWTP, but the specific requirements and limitations vary. Regulatory requirements imposed on WWTPs will be directly or indirectly imposed on the WTP discharging to the WWTP. Quality issues, especially levels of metals and other inorganics, are likely to be a concern.

6.2 Survey of Operating Systems

The results of a nationwide survey of WTPs currently discharging to WWTPs are shown in Appendix B.

6.3 Design Considerations and Conveyance Systems

For WTP managers and operators, disposal of WTP residuals into a sewer system or to a wastewater treatment facility is usually very attractive, cost effective, and offers significant benefits from a regulatory and operational standpoint (Weaver, 1985; Price et al., 1989; Robertson and Lin, 1981; Reh, 1978). This disposal method, however, has not been universally accepted by the wastewater community (Novak, 1989). Provided that the water treatment facility complies with the receiving wastewater utility's pretreatment and discharge requirements, the liability for proper disposal of the water treatment residuals is transferred from the water utility to the wastewater utility. Operationally, a WTP has no further responsibility for its waste residuals once they have been transferred to a wastewater utility; routine maintenance of the sewer system and/or operation of the wastewater treatment facility is the responsibility of the wastewater utility.

Costs associated with this option include the capital expenditures necessary to intercept, tie in, or transport residuals; user fees imposed by the receiving wastewater utility to recover conveyance; treatment and ultimate disposal operation and maintenance (O&M) costs; and other costs. These costs are often less than other options for directly handling WTP residuals. In some cases, the receiving WWTP benefits from increased removal of suspended solids and/or biochemical/chemical oxygen demand (BOD/COD), additional phosphorus, and H_2S conversion at the treatment plant (van Nieuwenhuyze et al., 1990).

WWTP operators, however, may be increasingly reluctant to accept WTP residuals because of

Table 6-1. Survey of Water Treatment Plants Discharging to WWTPs (AWWA/AWWARF, 1992)

Facility	Size Avg. Daily (mgd)	 Design (mgd)	Major Processes
Dunkirk Water Treatment Plant Dunkirk, NY	4	8	Coagulation, lime softening, filtration
City of Boulder WTP Boulder, CO	5.3	12	Coagulation, sedimentation, filtration
Chesterfield County WTP	10.5	12	Fe/Mn removal, coagulation, lime softening, filtration
Conestoga WTP Lancaster, PA	7	16	Coagulation, sedimentation, filtration
T.W. Moses WTP Indianapolis, IN	10	16	Coagulation, sedimentation, filtration
Texarkana WTP Texarkana, TX	11	18	Sedimentation, filtration
Franklin WTP, Charlotte-Mecklenburg Utility District	47.6	96	Coagulation, sedimentation, filtration
Vest WTP, Charlotte-Mecklenburg Utility District	18.9	24	Coagulation, sedimentation, filtration
City of Myrtle Beach WTP Myrtle Beach, NC	14	29.5	Coagulation, sedimentation, filtration, ozone
City of Greensboro WTP Greensboro, NC	31	44	Coagulation, sedimentation, filtration
Knoxville WTP Knoxville, TN	30-33	60	Coagulation, sedimentation, filtration
Erie City Water Authority Erie, PA	35-40	60-80	Coagulation, sedimentation, filtration
Chattanooga WTP Chattanooga, TN	38	72	Coagulation, sedimentation, filtration, GAC
Belmont WTP Philadelphia, PA	60	80	Coagulation, sedimentation, filtration
Queens Lane WTP Philadelphia, PA	100	100	Coagulation, sedimentation, filtration
Nottingham WTP Cleveland, OH	87.1	100	Coagulation, sedimentation, filtration
Morgan WTP Cleveland, OH	80	150	Coagulation, sedimentation, filtration
Baldwin WTP Cleveland, OH	85	165	Coagulation, sedimentation, filtration
Nashville WTP Nashville, TN	120	180	Coagulation, softening, filtration
City of Milwaukee Waterworks Milwaukee, WI	60 Winter, 120 Summer	250	Coagulation, sedimentation, filtration
F.E. Weymouth Filtration Plant LaVerne, CA	330	520	Coagulation, sedimentation, filtration
Robert A. Skinner Filtration Plant Temecula, CA	300	520	Coagulation, sedimentation, filtration
Joseph Jensen Filtration Plant Granada Hills, CA	333	550	Coagulation, sedimentation, filtration

concerns regarding National Pollutant Discharge Elimination System (NPDES) compliance and toxicity. For example, the Charlotte-Mecklenburg Utility Department will limit future sewer disposal of residuals from its Franklin WTP because this practice violates pretreatment ordinances (Appendix B). Potential limiting concerns for a WWTP when it accepts WTP residuals include increased final suspended solids, decreased effective digester capacity, overloading of primary clarifier and sludge removal systems, and overloading of dewatering operations (van Nieuwenhuyze et al., 1990).

Discharge to a sewer is possible only when water treatment and wastewater treatment facilities work together cooperatively. Information sharing is imperative to establish such a relationship. A water utility should provide information concerning residuals quality and quantity, dry solids content, pH, nutrients, heavy metals, and other relevant parameters. In addition, a wastewater utility and water utility must cooperate to determine whether positive or negative effects can be expected at the wastewater facility with respect to hydraulic conditions in the sewer lines, hydraulic and process capacity at the WWTP, sludge treatment, and ultimate disposal of WWTP biosolids.

6.3.1 Regulatory Considerations

Regulations governing discharge to WWTPs are discussed in Chapter 2. Hazardous waste regulations (federal and state) are usually not a problem, since WTP residuals are rarely, if ever, classified as hazardous waste. Radioactive waste regulations are also not a problem, since the radioactive component of WTP residuals usually is very low and results from the removal of normally occurring radioactive material (NORM) from the raw water supply. Local receiving wastewater regulations are usually driven by the need for a utility to comply with provisions of the Clean Water Act. These local regulations are imposed to reduce the risk of operational problems at the receiving plant or of violation of its discharge permits.

Discharges to sanitary sewers are subject to the U.S. Environmental Protection Agency's (EPA's) National Pretreatment Standards and possibly to more stringent pretreatment requirements imposed by the state or WWTP. The requirements imposed by a wastewater treatment facility are necessary to enable the facility to achieve compliance with its NPDES permit (including provision for sludge disposal). Pretreatment standards are typically industry or site specific. The local discharge requirements imposed by a wastewater utility may be governed by:

- Impact of residuals on the waste conveyance system (sewer, pump station, force mains, abrasion, and corrosion).
- Impact of residuals on either the liquid or solids process treatment system at the wastewater treatment facility (e.g., rate of discharge, solids concentrations, need for flow equalization).
- Concerns about biotoxicity (within the WWTP or in the plant's effluent).

The wastewater facility may refuse to accept the waste, or it may impose extremely strict local limits on the supplier's discharge (Koorse, 1993b).

Limitations often placed on WTP residuals discharged to sanitary sewers and/or to WWTPs are the result of problems perceived or experienced by the receiving wastewater treatment facilities. The data in Appendix B indicate that discharge of residuals through a force main from a WTP in Knoxville, TN, caused a high degree of solids accumulation in flocculation basins and considerable yearly maintenance. In a study of 13 WTPs operated by the American Water Works Service Company in West Virginia (EET, 1992), five of the plants discharged directly to sanitary sewers, and three plants hauled their residuals to a WWTP. Sewer systems at two locations experienced blockages that resulted in backwash water and WTP residual overflows to surface water. Only one plant is operating under a local industrial user permit, stipulating maximum heavy metal concentrations, pH and flow limitations, and monitoring requirements.

As greater concern is paid to WWTP effluent toxicity, municipalities are adopting pretreatment regulations to limit the strength or quality of discharge to sanitary sewers. Aquatic toxicity caused by aluminum in wastewater plant effluent has been widely studied. Hall and Hall (1989) report that a substantial increase in the mortality rate of *Ceriodaphnia dubia* is not observed in 100 percent alum effluent, but delayed brood release and significant reduction in reproduction are observed. Reductions in pH and dissolved oxygen concentrations, high levels of suspended solids, and, possibly, aqueous aluminum were implicated as likely causes of toxicity. In Norfolk, Virginia, WWTP effluent toxicity was tested (Tsang and Hurdle, 1991). When the greatest quantities of alum sludge were fed into the wastewater plant influent, no acute but some chronic toxicity results were measured. At lower feed rates of alum residuals to the plant, no toxic effects were noted.

Larger WTPs that discharge their residuals to a sanitary sewer or WWTP are usually regulated by local industrial user permits. Examples of this are Indianapolis, Milwaukee, and several large WTPs at the Metropolitan Water District in southern California, where a surcharge is required if heavy metal limits are exceeded (see Appendix B).

The City of Philadelphia's Queens Lane and Belmont WTPs discharge all of their residuals to sanitary sewers and operate under combined effluent discharge permits required in-house between the water and wastewater plants. The permits are administered by the Industrial Waste Unit of the Philadelphia Water Department. Limitations were placed on the discharge from these plants because of NPDES permit violations by the WWTPs. The City of Philadelphia's permits limit the discharges resulting from periodic raw water basin cleaning, routine flocculation/sedimentation tank discharges, and periodic flocculation/sedimentation tank cleaning. The user permit requires notification of discharges, sampling and monitoring, onsite detention for equalization and solids relocation, prohibition of accidental or slug discharges, and maximum 3-day discharge levels (lb/day dry weight) of residuals solids that may be discharged (AWWA/AWWARF, 1992).[1]

Municipal discharge permit ordinances are written to protect the receiving WWTPs and are industry oriented. The limitations specified are often too stringent for the discharge of WTP residuals. When the discharge of WTP residuals is allowed, discharge permit limitations may include:

- Limitations on maximum discharge rates, often during stated periods of the day.
- Maximum BOD concentration limits.
- Maximum total suspended solids (TSS) concentration limits.
- Total dry solids limits—tons/day maximum.
- Cessation of discharge upon notification during high storm flow periods.
- Prenotification of discharge occurrences.

6.3.2 Conveyance System Design Considerations

6.3.2.1 Selection of Type

Three types of conveyance systems generally are used to transport WTP residuals to a wastewater treatment facility: 1) gravity sanitary sewers, 2) pumping/force main systems, and 3) truck transport. Factors that affect the choice of system include the receiving wastewater system's availability and capacity, impacts of the residuals on the treatment plant, cost, corrosion considerations, and ultimate disposal considerations. Discharge to sanitary sewers, where available, is often the least expensive and preferred choice for a WTP. This option, however, provides the receiving wastewater utility with the least treatment flexibility—all of the WTP residuals must be processed through the wastewater liquid stream processes. If a WWTP has excess design capacity beyond its normal wastewater loading, WTP residuals received through the sanitary sewer generally can be also processed.

When problems develop because of WTP residuals, they usually occur in the liquid stream processes. Alternatives to the sanitary sewer are generally more costly: discharge by pumping/force main or truck haul directly to the wastewater treatment facility. With either of these alternatives, the discharge is controllable and can be directed to the desired liquids or solids stream process at the WWTP. The WTP residuals do not have to go through the liquid stream treatment process at all. They can either be directed through the solids processing facilities, in a combined facility or separately, or they can be combined with the wastewater treatment biosolids after the solids processing facilities for ultimate disposal.

There are other reasons for considering alternatives to the sanitary sewer for conveying WTP residuals to a wastewater plant. Koplish and Watson (1991), for example, described a situation in Allentown, Pennsylvania, where one entity is responsible for all water and wastewater functions. There the preferred, most cost-effective option for dewatering residuals (water and wastewater) is to have all equipment at a single location. This, in turn, necessitates direct transport (versus gravity sewer transport) of the alum residuals to the wastewater treatment facilities, where they are fed into a thickener prior to belt filter press dewatering.

6.3.2.2 Pretreatment Requirements

Wastewater utilities often impose requirements on a water treatment facility that govern the release of residuals to a sanitary sewer. The most common of these requirements is equalization of discharge flow. Other pretreatment requirements can include regulating the quality of the discharge, which may include pH neutralization; homogenization of the waste stream to ensure uniform concentration versus slug concentration discharges; limits on the total solids allowed to be discharged; and limitations on quality parameters such as heavy metals or components that may cause corrosion, odors, or other undesirable conditions.

Equalization

Directly discharging WTP residuals to a sewer system in a fairly continuous and uniform way may be possible if the sedimentation basins at a WTP are equipped with residuals removal mechanisms. Generally, flow equalization is required to ensure a uniform sludge flow to the sewer. Equalization facilities provide storage for quantities of waste discharge that exceed the allowable discharge to a sewer system. The storage requirements depend on the designated waste discharge schedule. Flow equalization is almost always required when sedimentation basins are being cleaned, to control wastes from ion exchange processes that are only produced during media regeneration, and before discharging filter backwash water to a sewer.

The need to control the rate of water treatment waste discharge into a sewer system was shown in the 1989 Durham study by McTigue et al. (1989). The water treatment facility examined had sedimentation basins in two groups of four basins each. Each basin had six conical hoppers in the inlet end; each hopper was approximately

---

[1] Wankoff, W. 1992. Personal communication between C.P. Houck and W. Wankoff, Water Treatment Plant Manager, City of Philadelphia Water Department. December 14.

10 feet deep and had a drain in the bottom for the removal of residuals. Normal operation of the plant consisted of flushing the residuals from each hopper daily. Half of the basin hoppers were discharged to a sanitary sewer for 2 weeks, with no significant effect on the receiving WWTP effluent quality.

Based on these results, the remaining hoppers were connected so that all hopper-flushing residuals could be sent to the wastewater plant. Only slight increases in effluent suspended solids, turbidity, and color were observed from the additional hopper flushings. Problems did occur, however, when a complete basin was cleaned and the residuals were discharged over 6 hours. A slug discharge caused severe deterioration of the wastewater utility's effluent quality in terms of turbidity, color, and suspended solids. It took approximately 6 days for the wastewater treatment facility to recover from the shock load. In addition, the inert WTP solids captured by the WWTP began to accumulate in the digester when the operators failed to compensate by increasing the amount of digested biosolids removed for ultimate disposal. The result was a deterioration in digester supernatant quality.

The need for equalization capabilities is often realized during an unanticipated experience such as a situation that developed in Philadelphia in 1988.[2] Philadelphia has three WTPs, two of which—Queens Lane WTP (100 mgd) and Belmont WTP (60 mgd)—discharge their residuals to sanitary sewers. In 1988, the Queens Lane WTP cleaned flocculation/sedimentation basins that are stacked, with chain and flight residuals collectors on the top, and no residuals collection on the bottom. A significant time had elapsed since the last basin cleaning. A slug of approximately 400 tons (dry weight) of solids was discharged into the sanitary sewer that then completely overloaded the 210 mgd Southwest Water Pollution Control Plant (then loaded at 120 mgd). Residuals from the Southwest Plant were discharged via a 5-mile force main to the Sludge Recycling Center, and so the additional 400 tons of WTP residuals were pumped as quickly as possible into the Sludge Recycling Center, overloading it as well.

The result of the poor coordination between the water and wastewater treatment plants was that two WWTPs fell out of compliance with their discharge permits. When the permits were renewed, EPA required that an internal city permit be issued by the Wastewater Department to the Water Department, regulating the WTP residual discharges. Extensive monitoring was also required.

In Philadelphia, effluent limits have been established for WTP discharges from cleaning of raw water basins, routine flocculation/sedimentation tank discharges, and periodic flocculation/sedimentation tank cleaning. Because their solids content is low, routine filter backwash discharges are not regulated by permit, even though a high volume of water is discharged. Flow, total suspended solids (TSS), aluminum, iron, and arsenic are monitored on a daily maximum and monthly average basis. TSS limitations of 132,000 lb/day (daily max) and 53,000 lb/day (monthly average) have been placed on the Queens Lane plant discharges.

Similar limits are placed on Philadelphia's Belmont plant discharges. Total discharge to the city sewers must be at a controlled rate such that no more than 10 percent of the total solids in the basin is removed per day. TSS concentrations must be below 100 ppm. Total allowed maximum flow from either plant is 18 mgd, and it must occur during the first shift only. Cleaning and discharge from both plants cannot occur simultaneously. Additionally, the Queens Lane plant is required to maintain, in good working order, an onsite lagoon to receive diversional flow streams. The need for the installation and operation of equalization facilities for waste WTP residuals in Philadelphia is obvious, given its 1988 experience.

Equalization restrictions may include limitations on the time of day of discharge, the maximum flow over a certain period, or the maximum solids discharge that is allowed. Cornwell et al. (1987) present a mass storage diagram approach that analyzes storage volume required for a given solids concentrations. The resulting equalization basin is usually sized to be capable of decanting and thickening. Either a continuous flow or batch fill and draw thickener may be designed. The capability to mix the contents of the equalization/thickener tank also may be desirable. Cornwell et al. (1987) also discuss design considerations for equalization facilities to handle backwash water or similar high-flow, short-duration discharges before their release to sanitary sewers. Sometimes, discharges of peak flows during otherwise low flow periods in the sanitary sewer may be desirable.

Quality Discharge Limitations

Limitations may be imposed on TSS or total dissolved solids (TDS), pH range, heavy metal constituents (either from a receiving water quality or residuals standpoint, or from a biotoxicity standpoint), and on corrosion causing constituents. These limitations may be in a permit that regulates discharge directly to the WWTP through a force main or truck discharge, or in a permit for discharge to a sanitary sewer. Most municipal permit limitations imposed on industrial discharge to sanitary sewer systems contain a provision that the water discharged must be in a pH range of 6 to 9. This could cause problems for lime softening WTPs that either have continuous discharge of softening residuals, or periodic cleaning and discharge of sedimentation ba-

---

[2] See footnote 1.

sins. Neutralization facilities for pH adjustment before discharge at the water treatment facility may be required.

Heavy metals in the water environment are increasingly regulated to ensure against toxicity to humans, plants, and aquatic organisms. Heavy metals are generally concentrated in the residual solids of water treatment and wastewater treatment wastes, and limitations on the ultimate disposal of those residuals, as well as on the dissolved fraction of heavy metals in the effluent, are becoming more stringent.

Under the final City of Norfolk, Virginia/Hampton Roads Sanitation District (HRSD) implementation plan to treat WTP residuals at a district wastewater plant, WTP residuals were to be pumped directly to the wastewater solids processing facility to circumvent problems both with sludge settling and effluent toxicity (Tsang and Hurdle, 1991). The heavy metal content of the water plant residuals, compared with HRSD's Industrial Waste Discharge Regulations, indicated that the concentrations of copper, chromium, lead, and zinc were close to the HRSD limits (CDM, 1989).

In Philadelphia, the occurrence and level of arsenic in the city's WTP residuals discharged to the sanitary sewer may be of concern.[3] The city's Sludge Recycling Center biosolids (received from the city's WWTPs) have an arsenic concentration of 11 to 12 mg/kg. The city's WTP residuals contribute less than 30 percent of the total arsenic in the biosolids and are responsible for no more than 2 to 3 mg/kg of the total. EPA's 40 CFR Part 503 biosolids regulations establish a maximum allowable concentration of 41 mg/kg of arsenic in bulk sewage sludge, applied to a lawn or home garden, or in sludge that is sold or given away. For land application of biosolids, the State of New Jersey has set a maximum limit of 10 mg/kg (New Jersey, 1993). The concentration of arsenic in Philadelphia's biosolids is not a problem under the EPA regulation, but under New Jersey state regulations, it may be. The city permit requires monitoring of the arsenic concentration discharged by the WTPs to the city's sanitary sewer system.

A similar situation exists in Massachusetts, where the Type 1 cadmium and nickel standards of the Massachusetts Department of Environmental Protection are substantially more stringent than the federal 503 regulations for "exceptional quality" biosolids (Donovan and Toffey, 1993). Should the arsenic levels become problematic, a higher grade aluminum sulfate could be used which generally contains lower levels of contaminants such as arsenic.

Additional quality concerns that a WTP and a WWTP receiving industrial discharge must consider are the potential corrosive nature of the WTP residuals and the potential for hydrogen sulfide (H_2S) generation. Aside from the potential odor problem, the presence of H_2S can lead to corrosion of certain construction materials in the sewer system. If the waste residuals to be discharged are septic (e.g., a lagoon cleaning operation) or contain a high sulfate content, the sanitary sewer grades are especially flat, or the sewers are constructed with concrete pipe, the potential for H_2S generation and corrosion attack should be investigated.

The discharge of WTP ferric residuals can counteract the problems of H_2S formation in sewers. Iron residuals promote H_2S binding. Iron residuals can also benefit the wastewater treatment process through partial promotion of presedimentation and partial phosphorus removal. They can also minimize problems with sulfide or H_2S formation by keeping the H_2S content in anaerobic digester gas at an acceptable level (van Nieuwenhuyze et al., 1990).

The corrosive nature of brines resulting from ion exchange (IX), reverse osmosis (RO), or activated alumina treatment of ground water is well known. Brines discharged to WWTP could be corrosive to piping systems (Snoeyink et al., 1989) and may also upset the biological balance in biological wastewater treatment systems.

A final quality concern is the amount of heavy metal impurities in the water treatment chemicals used at the WTP. The effect of contaminants on the chemical character of treated residuals can be significant (Lee et al., 1990). The data in Table 6-2 analyze sludge from a water treatment facility in Pennsylvania. Theoretical calculations show that the treatment chemicals could be responsible for 100 percent of the levels of chromium, copper, and lead detected in the residuals. A large fraction (78 percent) of the zinc concentration could also be attributed to chemical addition. At the plant represented in Table 6-2, the levels of zinc in the residuals prohibit use for agricultural land application.

6.3.2.3 Gravity Sewers

Although WTP residuals will settle in the sanitary sewer, their deposition should not occur more rapidly than sanitary solids (except residuals from lime softening and diatomaceous earth WTPs). Sanitary sewers designed with adequate slopes for conveying sanitary solids should also convey WTP residuals. Generally, a velocity of approximately 2.5 ft/sec (0.8 m/sec) or a residuals dry solids content of less than 3 percent should be maintained to prevent sedimentation of hydroxide residuals solids (van Nieuwenhuyze et al., 1990; Cornwell et al., 1987). While not reporting the percent solids of its residuals, the Indianapolis Water Company did report that its residuals were too thick for gravity flow (Appendix B). Lime residuals may have settling velocities much higher than those of metal coagulant residuals, and deposition

---

[3] See footnote 1.

Table 6-2. Treatment Chemical Analysis Range of Detected Contaminant Levels (Dixon et al., 1988)

Contaminant	Ferric[a] Chloride (mg/kg)	Liquid[b] Alum (mg/kg)	Granular Alum (mg/kg)	Percol (mg/kg)	Cat Floc-T (mg/kg)	Cat Floc-TL (mg/kg)	Hydrated Lime (mg/kg)	Pebble Lime (mg/kg)	Carbon (mg/kg)	Hydrofluo-silicic Acid (mg/L)	Caustic Soda (mg/L)	C-9 (mg/L)	C-39 (mg/kg)
Arsenic (As)	108–122	214–270	555–621	<5.0	13	14	<5.0–15.5	<5.0	<5.0	3.4–18	<0.50	27	25
Barium (Ba)	<5.0–7.4	<5.0	<50	<50	1.3	<5.0	24–83	<50	45–78	0.8–8.7	<5.0	1.7	<50
Cadmium (Cd)	<0.5–59	<0.5–7.5	<5.0	<5.0	<0.5	<0.5	<5.0	<5.0	<5.0	1.8–2.9	<0.50	4.6	<5.0
Chromium (Cr)	148–568	16–24	<5.0	<5.0	<0.5	<0.5	<5.0–24	<5.0	<5.0	<0.50–2.9	<0.50	27.5	26
Copper (Cu)	32–840	<0.2–5.3	<2.0	<2.0	<0.2	1.3	<2.0–6.1	<2.0	5.9–13.5	<0.2–0.5	<0.20	1.4	<2.0
Lead (Pb)	<0.5–332	<0.5	<5.0	<5.0	<0.5	2.8	<5.0–21	<5.0	<5.0	44–55	<0.50	52	44
Nickel (Ni)	1.5–114	<5.0	<50	<50	<5.0	2.2	<50	<50	<50	<5.0	<5.0	21	<50
Selenium (Se)	129–150	84–104	<5.0–204	<5.0	<0.5	10.7	<5.0–50	<5.0	<5.0	14.7–13.5	<0.50	39	35
Zinc (Zn)	69–7,990	<0.5–2.3	<5.0–12	<5.0	<0.5	1.6	<5.0–9.5	<5.0	<5.0–8.0	<0.5–1,418	<0.50	203,400	114,600
No. of Samples	8	6	3	1	1	1	4	1	3	4	2	1	1

[a] Solution 30 percent w/w; specific gravity = 1.346 g/mL. Contaminant concentration expressed in milligrams per kilogram (mg/kg) ferric chloride solution.

[b] Solution 8 percent w/w Al_2O_3; specific gravity = 1.33 g/mL. Contaminant concentration expressed in mg/kg liquid alum.

in sewer lines can be difficult to prevent. Robertson and Lin (1981) recommend against the disposal of softening basin residuals by direct discharge to a wastewater utility because of the adverse effects on the sewer system conveying the wastes and on the equipment and unit processes of the pollution control plant.

The capacity of the sanitary sewer system into which WTP residuals are to be discharged should be checked against the planned rate of residuals discharge and the background flow in the sanitary sewer. Tsang and Hurdle (1991) identify other considerations regarding conveyance that should also be checked:

- Ensure that there are adequate upstream dischargers to flush the water plant residuals through the sewer system.
- If there are no dischargers or too few dischargers upstream, consider a discharge of “clean” filter backwash water after the residuals are discharged.
- If dilution flows are inadequate, plan on increased maintenance at intermediate raw sewage pump stations to guard against damage from abrasive material in the water plant residuals (especially if the raw water source is a river) and from the low pH of the residuals.

6.3.2.4 Pumping/Force Mains

Considerable data are published on the pumping of water treatment residuals (e.g., Gandhi, 1992), as well as on wastewater sludges (U.S. EPA, 1979b; Metcalf & Eddy, 1991; Mulbarger et al., 1981). Relatively little information is available, however, about the differences in pumping dilute water treatment coagulant residuals (less than 3 percent solids, dry weight) versus pumping water. Dilute concentrations of coagulant residuals may be pumped using centrifugal pumps with nonclog impellers (ASCE/AWWA, 1990). An operating point should be selected to the right of the pump curve so that when the pipe begins to clog, adequate head is available to unclog it. A pump with a steep pump curve should be selected to give a large reserve head. Precautions are required to protect pumps and pipeline materials from corrosion and abrasion, especially the abrasion of pump impellers. If impeller abrasion is a serious concern, it may be possible to use pneumatic ejectors to transport WTP residuals (Foster, 1975).

In the design of a force main, the pipeline should be laid out like a sewer—with constant slope—so pockets of residuals do not solidify in the dips during periods when pumping does not occur. Air release valves should be provided at high points, but without using small orifices that can clog. The pipeline should be provided with flushing ports and cleanouts, and a drain facility at either end so that the line can be flushed without creating a problem. Pigging has been used for long reaches where cleanouts are impractical, but rodding and jetting facilities are the most cost-effective cleaning alternatives for shorter runs. The pipeline should be designed with a minimum velocity of 2 ft/sec. If valving in the pipeline is required, eccentric plug valves are best, but ball valves, butterfly valves, pinch valves, and knife gate valves also work. Using a valve that will not obstruct solids or rodding equipment is best.

EPA's *Process Design Manual for Sludge Treatment and Disposal* (U.S. EPA, 1979b) discusses the differences between pumping water and wastewater residuals. In water piping, flow is usually turbulent. Formulas for friction loss with clean water (Hazen-Williams, Darcy-Weisbach) are based on turbulent flow. Residuals flow may also be turbulent, in which case the friction loss may be roughly that of water. Residuals flow, however, is unlike clean water in that laminar flow is also common. When laminar flow occurs, the friction loss may be much greater than that for water. Residuals up to 3 percent solids concentration may have friction loss characteristics up to twice that of water under laminar flow conditions when velocities are between 2.5 and 8 ft/sec.

The thixotropic behavior of residuals flow may significantly increase friction losses in residuals pumping, especially when restarting a pipeline that has been shut down over a period of time. Head loss calculations must consider the residuals viscosity and density, or alternatively, allow some safety factor of two to three times the head loss with clean water. One rule of thumb for residuals thicker than 5 percent is to multiply the water head loss by the percent total solids. In *Pipeline Friction Losses for Wastewater Sludges,* Mulbarger et al. (1981) prepared an extensive review of the literature on wastewater residuals pumping and rheology, resulting in a methodology for predicting head loss relationships with appropriate use of safety factors. Their document provides a series of predictive head loss curves for cast or ductile iron pipelines, 4 through 20 inches in diameter. Curves are presented for routine operation and worst case (50 percent greater head loss than associated with water) scenarios.

If the range of solids to be pumped is over 3 percent (dry weight), the solids tend not to separate (especially over 6 percent) unless a super polymer or chemical (H_2O_2 or acid) is added. In this event, pumping velocity is not so important, since keeping solids in suspension is not a serious concern. Also, head loss may preclude high velocities. Positive displacement pumps are recommended (and are mandatory for solids greater than 4 percent). Cleanouts and flushing ports are mandatory. Keeping the suction line short into the pump is also important.

Communities in which WTP residuals are currently pumped include Norfolk, Virginia (37th St. WTP) (CDM, 1989), Wichita, Kansas (lime sludge), North Marin County Water District, and Novato, California (U.S. EPA, 1978a).

6.3.2.5 Truck Hauling

A primary consideration in determining whether to use truck hauling to transport liquid WTP residuals is the potential impact, perceived or real, of residuals on the WWTP liquid stream processes. When a significant impact is anticipated, the use of truck transport allows the separate introduction of WTP residuals into a process or receiving vessel at the WWTP, where negative impacts can be minimized. Distance to the WWTP and quantity of residuals, as well as the congestion of land use, often determine whether to use a pumping/force main system or truck haul. If residuals quantities are small or if distance to the WWTP exceeds 10 miles, truck haul is preferred over pumping through a force main. Additionally, if the WTP residuals have already been thickened or dewatered, truck transportation is often the only feasible method of delivering the residuals to the WWTP.

Trucking partially dewatered WTP residuals often results in solids compression because of vibration and release of free water (ASCE/AWWA, 1990). As a result, the truck bed should be sealed and watertight. Wastewater treatment facilities are well aware of the requirements for truck transport of sludges from their facilities, and the same precautions and limitations should be applied to truck transport of WTP residuals, whether dewatered, thickened, or unthickened.

6.4 Design Considerations for Wastewater Treatment Plants

Anything discharged to a WTP, including WTP residuals, has an impact on the design and operation of the facility. Careful consideration of the design and/or operation of the WWTP, however, can mitigate the effects of discharging WTP residuals to the facility.

The first issues to consider are the types and characteristics of WTP residuals to be discharged to the WWTP. These include not only the physical and chemical characteristics of the residuals, but the form in which they are to be conveyed to the WWTP (i.e., as a liquid, semi-solid, solid.

The next important factor to consider is the manner in which the residuals are to be conveyed to the WWTP. Typical conveyance systems include gravity sewers, pumping/force mains, and truck hauling. The type of conveyance system used is important because it determines where the WTP residuals can be introduced into the WWTP's processes (i.e., to the liquid and/or solids handling processes), as well as the rate at which the residuals are to be introduced (i.e., gradually over a long period of time, or as a slug dose).

The points at which WTP residuals are introduced to the WWTP for processing must be carefully considered. In certain cases, introducing residuals to the liquid treatment processes may have an adverse effect on those processes while, in other situations, it may improve the performance of the processes. Sometimes, it may be more advantageous to introduce the residuals to the WWTP's solids handling processes to prevent overloading the liquid treatment processes. Introducing WTP residuals to the solids handling processes does not necessarily eliminate all impacts on the liquid treatment processes because this option can still affect the recycled flows to the liquid treatment processes.

Obviously, the final and most important consideration in accepting WTP residuals at a WWTP is the impact on the performance of the WWTP. The performance of the unit treatment processes at the WWTP must be maintained to ensure that discharge of the final effluent to the receiving body is not adversely affected, and that disposal or beneficial reuse of the residuals from the WWTP does not become a problem.

As noted in Chapter 2, the NPDES permit governs any direct discharges to waters of the United States. The introduction of WTP residuals to a WWTP may affect two areas of the WWTP's NPDES permit: the parameters for which specific effluent limits are described, and the Whole Effluent Toxicity (WET).

Equally important to meeting effluent NPDES permit requirements is the fact that the disposal and/or reuse of residuals from a WWTP has taken on new meaning as a result of the promulgation of the 40 CFR Part 503 sewage sludge regulations (U.S. EPA, 1993c). These regulations address the disposal and beneficial reuse of sewage sludge in three general categories—land application, surface disposal, and incineration.

Constituents in WTP residuals discharged to a WWTP may affect the quality of the WWTP residuals as governed by 40 CFR Part 503, especially the concentrations of 10 heavy metals. If aluminum is included in round 2 of the 40 CFR Part 503 regulations, it definitely will present a problem for discharge of WTP residuals to WWTPs. The remaining sections of this chapter discuss design issues that must be carefully considered when evaluating whether to accept WTP residuals in the liquid or solids handling processes at a WWTP.

6.4.1 Hydraulic Loading

Hydraulic loading is generally not a factor in the acceptance of WTP residuals conveyed by gravity sewers to a WWTP. Large volumes of liquid wastes generated by WTP processes in a short period (e.g., filter backwash waters, IX regenerate waste, reject water from membrane processes), are usually equalized at the WTP prior to being discharged to a sewer. This is done to prevent overloading of the gravity sewer itself. If large volumes of liquid WTP residuals are not equalized before discharge, attenuation that takes place in the sewer generally prevents a hydraulic loading problem at the WWTP.

It is generally more important to check the hydraulic loading of WTP residuals conveyed to a WWTP through a dedicated pump/force main or by truck. The hydraulic loading on unit treatment processes at the WWTP is usually not the controlling parameter when WTP residuals are introduced to the liquid treatment process train, unless the WWTP is a small one.

The condition that most warrants concern about hydraulic loadings is when WTP residuals are discharged to the solids handling processes at a WWTP. In this case, equalization at either the WTP or WWTP is generally required.

6.4.2 Organic Loading

The organic content of WTP residuals varies widely, depending heavily on the quality of the raw water processed at the WTP (Albrecht, 1972; Calkins and Novak, 1973; Cornwell et al., 1987). The concentration of organic matter in WTP residuals, as measured by BOD or COD, is not typically in the range of that found in wastewater. Although organics loading is typically the controlling parameter for the biological unit treatment processes in both the liquid and solids handling process trains at a WWTP, the additional organics loading encountered by introducing WTP residuals generally does not have a significant impact.

6.4.3 Solids Loading

The impact of solids in WTP residuals on the WWTP is limited to two general areas:

- The performance of primary, intermediate, and final clarifiers.
- The constituents in the WWTP biosolids as related to the requirements of the 40 CFR Part 503 sewage sludge regulations.

The introduction of additional solids from WTP residuals generally does not significantly affect the ability of the WWTP to comply with its NPDES permit requirements for the final effluent (McTigue et al., 1989; Tsang and Hurdle, 1991). In several cases, alum and iron residuals from WTPs have actually improved the efficiency of primary clarification in WWTPs; in another case, these residuals increased the amount of phosphorus chemically precipitated from a WWTP (McTigue et al., 1989; Tsang and Hurdle, 1991). A common factor related to the discharge of WTP residuals to a WWTP is an increase in biosolids from the WWTP, with a corresponding decrease in the volatility of the biosolids.

In certain instances, metals present in WTP residuals can affect NPDES permit effluent limits. This situation is becoming more of a problem since most of the specific effluent requirements for metals in NPDES permits have daily limits. If a significant portion of the precipitates in the WTP residuals is in colloidal form, the solids pass through primary and/or secondary clarification at the WWTP.

Another cause of increased metals concentrations in a WWTP effluent can be the resolubilization of metals that occurs when WTP residuals are processed in a WWTP's solids handling processes, and recycled flows from solids handling are returned to the liquid process train. Precipitates and other solids in WTP residuals are fairly inert. Concern has been expressed that additional solids from WTP residuals could impair fixed film biological treatment processes, but definitive information to substantiate this concern is unavailable.

6.4.4 Toxics Loading

The potential toxicity of WTP residuals is a concern because of 1) the potential inhibitory impact on biological unit treatment processes at the WWTP, and 2) the possibility of failing a bioassay with *Ceriodaphnia dubia* or fathead minnows, resulting in noncompliance with the WET requirements.

A discussion of the potential for hazardous waste classification of WTP residuals is presented in Chapter 2. Passing the TCLP test allows a WTP residual to be classified as nonhazardous; however, it does not preclude the possibility of toxic effects on a WWTP if the WTP residual is discharged to the facility. Although organics and inorganics can be the cause of toxic effects from WTP residuals, heavy metals are most often responsible for toxicity problems at WWTPs.

Nearly all heavy metals can exert toxic effects at elevated concentrations. Heavy metals can be grouped into "essential" and "nonessential" metals, according to their importance in biological systems. The vast majority of heavy metals fall into the "essential" category. Cadmium, mercury, and lead are the most commonly found nonessential heavy metals. In assessing the potential toxicity of individual heavy metals, two important issues should be considered:

- For metals essential to biological processes, an increase in concentration can often improve biological conditions if a deficiency of metals existed initially.
- When they are present in excess, the essential metals can sometimes exert greater toxicity than the nonessential heavy metals.

Metal speciation plays a major role in the potential toxicity of a particular heavy metal. The dissolved portion, available as the free metal ion, is generally the most toxic to biological systems. This simple differentiation between the dissolved and particulate forms, however, is generally not adequate to determine toxicity effects. Heavy metals in the dissolved phase can be free metal ions, or can be tied up with inorganic or organic compounds to form complexes. Heavy metals in the particulate phase are capable of being bound to the surface of other solids, or tied up in the bulk phase of a precipitate.

6.4.5 Liquid/Solids Separation

A key impact of WTP residuals on WWTP unit processes is that additional residuals handling will be necessary. The effects of the additional residuals on clarification, digestion, dewatering, and final disposal must be considered. Generally, final biosolids volume increases as the amount of influent WTP residuals increases. An increase in the primary clarifier sludge volume is to be expected with the addition of WTP residuals to raw wastewater.

Liquid/solid separation is an important unit operation in water and wastewater treatment. The success of the overall treatment process depends greatly on a facility's ability to separate solids from the liquid stream. Liquid/solid separation can be accomplished by several unit operations. This section discusses the primary and secondary sedimentation operations, while other liquids/solids operations are discussed in Section 6.4.6.3.

When WTP residuals are discharged into a sanitary sewer system, one of the first unit processes that they encounter is primary sedimentation at a WWTP. Several studies conducted on the impact of WTP residuals on the sedimentation process have reported fairly consistent results.

Full-scale testing at the Hookers Point Sewage Treatment Plant in Tampa, Florida (Wilson et al., 1975) showed that when WTP residuals were discharged to a sanitary sewer system, a fluffy blanket layer of sludge formed at a level somewhat above the normal blanket level in WWTP primary sedimentation tanks. At times, the sludge collectors in the primary sedimentation tanks seemed incapable of collecting this fluffy material even when the sludge pumps were run at their maximum rate. Data show that at a WTP residuals dosage of up to 50 mg/L, removal of suspended solids from the primary sedimentation tanks improved. At dosages above 50 mg/L, the percentage removals may be expected to drop. Wilson et al. (1975) note that when WTP residuals were accepted on a regular basis at the plant, the primary sludge solids content dropped from 5.8 to 4.7 percent. The addition of high concentrations of alum residuals tended to produce a primary sludge that was difficult to handle. Dosages of fresh alum less than 100 mg/L or WTP residuals of 40 mg/L were reported to produce a more manageable product.

Rolan and Brown (1973) report that combined alum residuals and sewage sludge did not settle as well as each would separately. Sludge volume tests were conducted for different dosages of WTP alum residuals to a WWTP (see Figure 6-1). The addition of 200 mg/L of alum residuals approximately doubled the volume of primary clarifier sludge. It was concluded that WTP residuals in concentrations of 200 mg/L or less would have no significant adverse effects on a WWTP.

Rindt (1973) studied the effects of alum on primary sedimentation in a series of laboratory tests using laboratory-prepared alum and WTP alum residuals. He reports that WTP alum residuals caused nearly a four-fold increase in the sludge volume index (SVI) when added to wastewater samples. Turbidity and total suspended solids in the supernatant also increased in proportion to the amount of residuals disposed of and the suspended solids concentration of the residuals.

Hsu and Pipes (1973) investigated the effect of aluminum hydroxide floc on the settling characteristics of primary and secondary effluent. Settleability of bulking sludge, as measured by the SVI, may be improved significantly with the addition of aluminum hydroxide, but there will be no effect on the SVI of normal sludge. The authors attribute the decrease in the SVI of bulking sludge to the inorganic aluminum hydroxide enmeshment, which increased the density and the compatibility of the sludge.

Nelson et al. (U.S. EPA, 1978a) conducted a full-scale evaluation of alum residuals discharged via sanitary sewer to an activated sludge WWTP and found that the efficiency of primary settling decreased by approximately 10 percent. Phosphorus removal improved by approximately 12 percent, and scum removal and settling improved in secondary clarification.

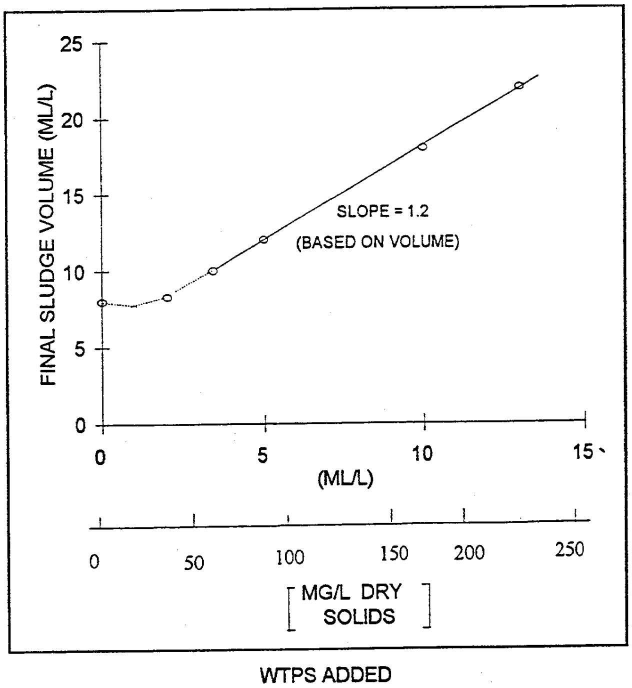


Figure 6-1. Effect of WTP sludge on the combined volume of wastewater sludge after 30 minutes of settling (Rolan and Brown, 1973).

Camp Dresser & McKee, Inc. (CDM, 1989), conducted a study of Norfolk, Virginia's, 37th Street Water Treatment Plant residuals and their subsequent treatment at the Hampton Roads Sanitation District (HRSD) Virginia Initiative Plant (VIP). To verify the potential effect that WTP residuals might have on the VIP, representative quantities of the residuals were hauled by truck to two of HRSD's smaller plants at a rate comparable to VIP receiving all of the residuals. The sludge was processed through liquids and solids processing facilities at one plant, and through liquid stream facilities at the other. The study showed the following:

- The addition of WTP residuals to the WWTP influent caused the primary sludge concentration at the WWTP to drop from 3.5 to 2.5 percent, increasing the volume of liquid primary sludge to be pumped by approximately 60 percent.
- The addition of WTP residuals to the WWTP influent decreased the efficiency of solids settling in the secondary clarifiers, resulting in the need for additional clarifier capacity or the addition of a polymer to enhance settling.
- Discharging of WTP residuals directly to the WWTP solids handling facilities resulted in minimal adverse impacts on solids processing facilities other than increased flows.

The study concluded that sanitary sewer discharge of WTP residuals to the VIP was not feasible because of the adverse effect on settling at VIP and the need to obtain an exception to the HRSD Industrial Discharge Regulations as a result. The processing of WTP residuals, transported by force main to the VIP and introduced directly to the solids processing facilities (i.e., centrifuges), appears technically feasible. The residuals will be dewatered separately by the centrifuges and then combined with the WWTP biosolids before being fed to the incinerators.

6.4.5.1 Summary

All the above studies involved WTP coagulation residuals, which constitute most of the WTP residuals produced in the United States. The findings from these studies confirm that 1) the WWTP sludge volume will generally increase, 2) phosphorus removal from the wastewater will be improved, and 3) sludge concentration in the underflow may decrease when coagulation sludge is discharged to a WWTP. Lime softening residuals settle very well and should not affect the wastewater treatment primary settled sludge volume as much as coagulation residuals do.

When evaluating the option of discharging WTP residuals to a WWTP via a sanitary sewer system, sedimentation tank capacities must be checked to ensure that they will not be overloaded by the additional solids. This precaution is especially critical for primary sedimentation, since the majority of the residuals solids will be removed here. The capacity of sludge pumps, pipes, and subsequent conveyance systems must also be examined; the removal of a larger volume of sludge at a lower solids concentration would require a larger pumping capacity and/or a higher pumping frequency.

6.4.6 In-Plant Solids Handling

The introduction of WTP residuals to a WWTP will inevitably increase the solids loading to the whole treatment train. This is especially critical for the solids handling operations and treatment processes such as sludge digestion, thickening, and dewatering.

6.4.6.1 Aerobic Digestion

Aerobic digestion is a sludge stabilization process used primarily in small treatment facilities. The process is similar to the activated sludge process. Sludge in the digester is aerated to oxidize the organic substrate, and as the available substrate is depleted, the microorganisms begin to consume their own protoplasm through endogenous respiration.

Important design criteria for aerobic digesters include hydraulic detention time, solids loading, oxygen requirement, and energy requirements for mixing. Both volatile solids loading together with total solids loading should be considered. Because aerobic digestion is similar to the activated sludge process, the effect of WTP residuals on aerobic digestion should be somewhat similar to the impacts of the residuals on activated sludge systems. More data are available on the impact of WTP residuals on activated sludge, than on aerobic digestion. Consequently, some of the following discussion will be on activated sludge systems.

To accommodate WTP residuals, an aerobic digestion system must have the physical capacity to accept the additional solids loading that residuals bring. Although the volatile solids contribution from the residuals is expected to be low and should not significantly affect the overall volatile solids loading to the digester, the impact of the residuals on hydraulic and solids retention time, as well as the overall reduction in volatile solids concentration, must be considered.

Toxic compounds present in WTP residuals can adversely affect the biological processes of wastewater treatment facilities. Dissolved solids present in a liquid phase waste may thwart the biological process, depending on their form and/or concentration. Predischarge equalization techniques may alleviate toxicity problems, as a continuous discharge of suitable dilution will enhance the ability of the microorganisms present in the biological process to adapt and adjust to the presence of any inorganic ion. If the dosing of WTP residuals is equalized so that surges do not occur, and the dose is kept below 150 to 200 mg/L, no direct effect on the activated sludge process is likely to take place, although downstream process effects or solids handling process effects are possible (van Nieuwenhuyze et al., 1990).

Several investigations have found no evidence that alum hinders the substrate removal properties of the activated sludge process. Nelson et al. (U.S. EPA, 1978a) reported no change in the efficiencies of COD and BOD removals when alum sludge was discharged into a wastewater treatment plant. In a series of laboratory-scale studies, Voorhees (1974) studied the effects of alum sludge on the activated sludge process and reported that no significant effects on either COD or TOC removal were observed at alum dosage of up to 100 mg/L. Voorhees observed that aluminum hydroxide sludge did not decrease SVI, and that the settling characteristics were not adversely affected in the secondary clarifier. When a shock load of alum sludge was introduced to the system, the COD and TOC removals were decreased. The system was able, however, to recover to the same COD and TOC removal efficiencies before the shock load.

Despite several findings reporting the apparent lack of impact of WTP residuals on biological treatment processes, the metal contents of these residuals do pose some concerns. Reid et al. (1968) presented data to show that metallic ions such as hexavalent chromium, cadmium, copper, nickel, aluminum, and silver can hinder biological processes.

Hsu and Pipes (1973) reported that doses of aluminum hydroxide in the range of 10 to 300 mg/L as aluminum to an aeration tank caused a significant increase in total phosphorus removal, no change on the nitrification, and a slight increase in COD removal. Hsu and Pipes stated that the most significant adverse effect on the biological treatment process was the increased volume of sludge produced. As a result, the sludge withdrawal rate and sludge return rate would have to be controlled very carefully to ensure adequate supply of active biological sludge mass to the aeration tank.

Anderson and Hammer (1973) studied the effects of alum addition on activated sludge biota and found that the bacteria were able to synthesize soluble organics in a chemical-biological sludge process and that BOD removal was not affected. Higher life forms such as protozoa, however, found the environment under high alum additions too adverse for normal existence.

The level of salt concentration in WTP residuals is extremely important to the proper performance of the biological processes at a WWTP. Introducing WTP residuals with high sodium chloride concentrations, such as may occur from the undiluted discharge of IX waste, could cause considerable stress to biological organisms. As a result, COD of the biological reactor effluent could increase after the salt addition. Threshold concentrations of toxic ions in the biological process are the level above which a decrease in the COD removal efficiency of the biological process occurs. If precautions for equalization and dilution of WTP residuals prior to discharge are adhered to, then brine discharges are unlikely to affect the receiving wastewater plant detrimentally (Cornwell et al., 1987).

6.4.6.2 Anaerobic Digestion

Anaerobic digestion is one of the oldest processes used to stabilize sludges. It involves the decomposition of organic and inorganic matter in the absence of oxygen. Important design criteria for anaerobic digestion include hydraulic and solids retention times, volatile solids loading, mixing energy, gas production, and volatile solids reduction.

The additional volume and dry weight of sludge and the decrease in percent of volatile matter must be considered when determining the sizing and performance parameters of digesters. A 1989 study (Foley et al., 1989) of the East Bay Municipal Utility District (EBMUD) in San Francisco, California, examined the effects of alum residuals on a digester by placing the alum residuals directly into the digester. The performance of digester 11 receiving alum residuals directly was compared with digesters operating under “normal” procedures (i.e., treating a mixture of primary and WTP residuals introduced into the influent).

Table 6-3. Comparison of Digester 11 With Background Digesters (Foley et al., 1989)

Parameter	Digester 11 (mg/L)	Background Digesters (mg/L)
Aluminum	15,000	7,200
Cadmium	16	18
Chromium	140	140
Copper	390	400
Iron	16,000	17,000
Lead	180	160
Nickel	41	41
Silver	16	23
Zinc	900	960
pH	7.7	7.6
TS	2.3	2.5
VS	1.4	1.6
TS/VS	58	61

The results of this part of the study are presented in Table 6-3. The alum residuals that were fed directly to digester 11 had an average total solids (TS) percent of 2.5 and an average volatile solids of 1 percent TS. No data were presented on the rate of gas production versus volatile solids reduction. There was a significant increase in aluminum concentration in the digester receiving the alum residuals compared with the digesters operating under “normal” procedures. This needs to be considered relative to ultimate disposal options. All other parameters were essentially equal.

Because softening residuals can cause lime deposition, discharging softening residuals to a WWTP may adversely affect the performance of the anaerobic digester. If softening residuals are added directly into the anaerobic digester, the temperature can fall because of the additional volume of inert material. Additional heating may be required in this event.

A study in Durham, North Carolina (McTigue et al., 1989) investigated the impact of controlled alum residuals discharge to a sanitary sewer on the physical facilities at a WWTP. The study showed an insignificant effect on flow rate through the wastewater treatment system. The digested solids, however, increased 46 percent with the addition of WTP residuals. The loadings on the biosolids drying beds increased by the same percentage, from 9.8 lb/ft^2/yr to 14.3 lb/ft^2/yr, with the addition of these residuals. The retention time and surface loading rate associated with other processes were not significantly affected, with the exception of the volume of sludge collected and handled in the primary clarifier, which was estimated to increase by 59 percent, from approximately 3.91 to 6.2 tons per day (TPD). The retention time in the aerobic digesters decreased from 34.7 to 26.7 days, a 23 percent reduction, when WTP residuals were introduced. The impact on the wastewater treatment facility was significant, particularly in the retention time of the aerobic digesters, increased pumping needed for the primary clarifier sludge, and the additional drying bed capacity required.

Rindt (1973) used a laboratory-scale study to evaluate the impact of alum residuals on anaerobic digestion. He found that gas production and TOC remained largely unaffected. Total solids in the digester increased because of the additional alum residuals load. The pH in the digester was also depressed slightly by the alum residuals. The percentage of volatile solids in the digester decreased because of the increased inorganic input. Inorganic phosphates in the digester were almost entirely precipitated by the aluminum present.

These results were confirmed by Reed (1975), who used laboratory-scale anaerobic digesters to study the effects of both reagent grade alum (aluminum sulfate) and WTP residuals (aluminum hydroxide) on the digestion process. Under comparable aluminum concentrations, Reed found that digestion was adversely affected in the digester where feed sludge contained aluminum sulfate, but not in the digester where feed sludge contained aluminum hydroxide residuals. Inhibition was attributed to the high sulfide concentration associated with the reagent grade alum.

Barth et al. (1965) studied the effect of metals on biological processes and found that for chromium, nickel, and zinc, an influent metals concentration of 10 mg/L—for any one or combination of the three—did not affect digestion. Copper continuously present at 10 mg/L caused failure of combined sludge digestion.

Wilson et al. (1975) did not observe any negative impacts at the anaerobic digesters in Tampa, Florida, except that the solids content of the biosolids discharged to the drying beds was reduced by a factor of approximately two as a result of the WTP residuals addition.

Nelson et al. (U.S. EPA, 1978a) observed an increase in digester gas production beyond that generated by the increased load of volatile solids applied. Nelson et al. speculated that this was due to easier mixing of the digester contents as a result of WTP residuals being added. Hsu and Pipes (1973) reported that aluminum hydroxide retarded the digestion process.

Emig (1979) conducted a full-scale test employing alum and polymer to treat phosphorus at a WWTP. He reported a drop in digester gas production, accompanied by a drop in digester pH (from approximately 7.2 to 6.0) after the addition of alum. This change led to a digester upset at alum dosage of approximately 400 mg/L. Alum addition had reduced the alkalinity of the digesting sludge and increased the volatile acid levels. The upset was caused by the depression of the alkaline wastewater pH.

These studies show that unless alum residuals are introduced at a very high rate, anaerobic digesters are unlikely to be significantly affected, provided the digester has adequate capacity to accommodate the increased solids loading. In assessing digester capacity, the reduced volatile solids concentration of the residuals stream must be taken into account.

6.4.6.3 Thickening/Dewatering

Thickening and dewatering are unit operations that separate liquid from solids. In its unthickened condition (e.g., underflow withdrawal from a clarifier), a WTP residuals/WWTP sludge blend normally has a solids concentration in the range of 2 to 6 percent total solids. Thickening can reduce the water content to the range of 9 to 15 percent total solids. In this range, the thickened residual still behaves like a liquid but has the consistency and physical properties of chocolate pudding. Dewatering of thickened residuals will result in a cake with a solids content of 18 to 40 percent.

Thickening and dewatering can be accomplished by natural or mechanical means. Examples of thickening operations are gravity thickening, flotation thickening, centrifuge thickening, and rotating drum thickening. Common dewatering operations include sand or paved drying beds, dewatering centrifuges, belt filter presses, and recessed chamber filter presses. All thickening and dewatering equipment is sized according to solids loading. The introduction of WTP residuals may cause overloading of these facilities, depending on the quantity of residuals introduced and the existing WWTP loadings relative to the plant's design capacity.

Hsu and Pipes (1973) reported that the dewaterability of wastewater biosolids, as measured by the specific resistance to filtration, was improved by an increase in the dosage of aluminum hydroxide. Nelson et al. (U.S. EPA, 1978a) found that the concentrate quality and cake solids content of the centrifuge were not significantly changed by the introduction of WTP residuals. They noted that the yield from the centrifuge increased from 600 to 800 lb/hr for biosolids, to about 900 lb/hr for the alum residuals biosolids mixture. A study in Norfolk, Virginia (CDM, 1989), showed that the settling characteristics of waste activated sludge as measured by the sludge volume index was negatively affected by the addition of alum residuals. No significant impacts on centrifuge dewatering in terms of cake solids, polymer dosage, and centrate recovery were reported.

In almost every case, the dewaterability of the wastewater biosolids, as measured by the specific filtration resistance, was shown to improve with the addition of WTP residuals (van Nieuwenhuyze et al., 1990). A study by McTigue et al. (1989) investigated how the use of WTP residuals changed sludge handling characteristics and affected WWTP performance. A pilot column evaluation was conducted to determine the effect of WTP residuals on drainage characteristics of the WWTP biosolids. These tests were necessary to determine the impact of the WTP residuals on the WWTP's ultimate dewatering process (i.e., sand drying beds). McTigue et al. (1989) found that the addition of WTP residuals had little influence on the drainage characteristics of the aerobically digested activated sludge. The addition, however, of residuals to aerobically digested activated sludge caused the biosolids to settle in the bed and produce a clear water layer suitable for decant, thus improving the dewatering process. The dewatering tests on the combined anaerobically digested primary sludge and WTP residuals were very similar. The major difference was that the anaerobically digested primary sludge drained faster than the aerobically digested activated sludge and also contained some clarified surface water available for decant.

In Allentown, Pennsylvania, a pilot test was conducted to determine the effects of dewatering alum residuals either separately or combined with biosolids using a recessed chamber filter press (Koplish and Watson, 1991). The dewatering results indicated that the highest cake solids of 46.8 percent were achieved by dewatering alum residuals alone, although a cycle time of 3 hours was required. In comparison, wastewater biosolids were dewatered to 34.3 percent cake at a cycle time of 1 hour. When they were mixed at different proportions, the resultant cake solids were between 34.3 to 46.8 percent; the blend having higher portions of alum residuals resulted in the higher cake solids.

With the success of the pilot tests, the City of Allentown has proceeded to install full-scale facilities for accepting and dewatering a combination of biosolids and WTP residuals at its WWTP. The operation is termed the "Centralized Dewatering and Co-Disposal (CDCD)" concept, and is described by Koplish et al. (1995). Dewatering of a 50-50 blend is accomplished at the WWTP by high-solids belt filter presses, and the combined operation is saving the city at least a quarter of a million dollars annually. At a 50-50 blend, feed solids in January and February 1995 were 2.6 percent; cake solids were 25.6 percent. The WTP residuals are tank-trucked to the WWTP, where they are discharged to a blended residuals holding tank and mixed with biosolids. The operation has been so successful that the city is now considering accepting third party liquid residuals for processing, which could represent a new revenue source for the city.

A pilot study was conducted in Wilmington, North Carolina, employing a belt filter press.[4] It was found that alum residuals dewatered alone attained a maximum cake solids content of 23 to 27 percent. Wastewater biosolids were dewatered to 15 to 18 percent solids. When alum residuals and wastewater biosolids were blended, the dewatered cake had a solids concentration of 17 to 20 percent. The results were consistent with bench-scale tests conducted earlier on the same sludges. The reported test results followed a pattern similar to the Allentown study.

Vandermeyden and Potter (1991) reported on full-scale sludge dewatering tests for alum WTP residuals using both centrifuge and belt filter press equipment. The tests yielded favorable results in terms of cake solids concentration, polymer dose, and centrate/filtrate quality. The dewatering test results are summarized in Table 6-4 and indicate that cake solids concentrations typically ranged from 20 to 27 percent, polymer dose from 5 to 70 lb/ton of solids, and overall solids capture around 98 percent.

Table 6-4. Alum Treatment Plant Sludge Dewatering Test Results (Vandermeyden and Potter, 1991)

Parameter	Belt Filter Press	Centrifuge
Loading rate (lb/hr)	700–800	560–750
Feed solids (%)	2.5	2.5
Polymer dose (lb/hr)	20–23	25–27
Solids capture (%)	98	98

Although available information has generally shown that WWTP biosolids dewaterability is not negatively impacted by alum residuals, gravity settling of the wastewater sludge may be affected. The ability to reduce the water content of sludge depends on many factors, including the physical, chemical, and biological characteristics of the sludge, and the unit operations employed to thicken or dewater it. To date, the science of dewatering has not advanced to the point when sludge dewaterability can be predicted precisely without actual testing. Data that are useful in determining sludge dewaterability, such as particle size distribution and zeta potential, are seldom available.

If the impacts of cake solids are critical to downstream processing, the effect of WTP residuals on sludge thickening and dewatering should be fully investigated through pilot- or full-scale testing before the co-processing of WTP residuals and WWTP biosolids is implemented.

6.5 Ultimate Disposal of Wastewater Treatment Plant Biosolids

Several different ultimate disposal options are available to WWTPs. Before the advent of the Clean Water Act (CWA), one common disposal option was direct discharge to a nearby surface water body. Current regulations limit this as a viable option for WWTPs. The regulatory climate further requires WWTPs to develop, evaluate, and implement alternative ultimate disposal methods. Options include land application, incineration, and composting, all of which could be considered beneficial uses. The regulations governing beneficial reuse options have only recently been established by some primacy agencies; other primacy agencies are still developing regulations. The promulgation of 40 CFR Part 503 will further these options as guidelines and regulations are established.

Quality, quantity, and cost are key factors in determining which disposal options a WWTP employs. The regulatory requirements for WWTP biosolids, established by the primary agencies responsible for monitoring ultimate disposal, are applied to WTPs that discharge to WWTPs. Regulatory requirements and technical issues

---

[4] City of Wilmington. 1991. Personal communication.

relating to these disposal options are addressed in the following sections.

6.5.1 Direct Discharge

In the past, direct discharge was commonly used to dispose of WTP residuals. Protection of aquatic resources and maintenance or improvement of water quality conditions has resulted recently in more stringent regulation. As a result of the CWA, regulation of this option has increased the cost and concern over liability to the WTP. The regulatory requirements are primarily associated with the Federal Ambient Water Quality Criteria, established through the CWA to protect aquatic life in marine and freshwater environments. Regulations of particular interest to the water treatment industry are those that establish acceptable levels of metals, suspended solids, and water treatment–related contaminants (see Chapters 2 and 5).

For a WWTP to consider receiving WTP residuals, anticipated levels of metals and other contaminants would have to be determined to ensure compliance with NPDES permits limits.

6.5.2 Land Application

Land application, as it applies to disposal of WTP residuals, is discussed in Chapter 8. The same quality restrictions and load limitations that apply to land application WTP residuals apply to wastewater biosolids; however, biosolids have the added benefit of higher organic or nutrient content. Higher organic content, nitrogen, and phosphorus levels are beneficial if land application of the biosolids is associated with agricultural uses.

The addition of alum WTP residuals to WWTP biosolids could reduce the level of available phosphorus and create concerns over aluminum toxicity. The addition of residuals from a water softening WTP, or the addition of alum residuals conditioned by lime for dewatering could prove to be very beneficial to a joint land application program with biosolids. Examples of each of these two approaches are reported by Parsons and Waldrip (1995) and by Assadian and Fenn (1995).

Ionic aluminum is toxic to many plants, including agricultural crops (Elliot and Singer, 1988). This detrimental effect could limit the land application process for biosolids containing alum WTP residuals. A WWTP manager currently land applying biosolids must therefore be watchful of additional aluminum in the biosolids to ensure that regulatory limits are not exceeded. Recent studies, however, performed by Grabarek and Krug (1989) show that large doses of alum sludge do not necessarily produce aluminum toxicity (Elliot et al., 1988).

As discussed previously, one benefit of land applying WWTP biosolids is the contribution of organic matter and nutrients. The addition of aluminum or iron hydroxides could result in a strong fixation of available phosphorus (PO_4^{-3}), thus making it unavailable for vegetation. This does not preclude the addition of WTP residuals, but careful consideration must be given to relative quantities and qualities of any additives, as well as beneficial reuse objectives. If land spreading for nonagricultural purposes is the ultimate disposal method, concerns over aluminum toxicity and available phosphorus are minimized. Pilot-scale testing and chemical analysis can be conducted to assess viability.

The City of Boulder, Colorado, has undertaken an extensive evaluation of the benefits and risks of coapplication of WTP residuals and WWTP biosolids (Harberg and Heppler, 1993). They reported that the beneficial effects of direct addition of WTP residuals to WWTP biosolids depend on soil conditions, characteristics of the residuals, and application rates. If not properly managed, direct application of WTP residuals could cause detrimental plant growth and other environmental effects. An evaluation of the literature on the effects of WTP residuals on crop growth response suggested that land coapplication of WTP residuals and WWTP biosolids for beneficial use was a viable way to recycle WTP residuals, and did not reduce the plant-available phosphorus in soils. In fact, the WWTP biosolids would help supplement the phosphorus available in the soil. A proper combination of the two materials could also provide the nitrogen needed by the plants without reducing the phosphorus availability in the soil. The City of Boulder study is specifically evaluating coapplication of an aluminum-based WTP residual and a WWTP biosolid to dry land winter wheat. Based on crop yield, the WTP residuals did not significantly affect yields, and no difference between control and applied plots was found.

6.5.3 Incineration

Incineration provides an effective method for reducing volume and creating a relatively inert material. Depending on the initial metals levels in the biosolids, leaching of metals from the ash residue may be a concern.

The trend in biosolids disposal is increasingly in favor of beneficial reuse options. Incineration is becoming a less desirable option because of its relatively higher costs, as well as recent concerns and regulations on air emissions.

The addition of a WTP residual into the processes of a WWTP currently incinerating biosolids could create some problems. Because the organic content of WTP residuals is generally lower than that of WWTP biosolids, the thermal destruction process may be hindered. The presence and contribution of iron and aluminum, as well as other coagulant contaminants, must also be considered relative to facility operations, air emissions, and leaching from ash residue.

6.5.4 Composting

Composting is a biological process that converts biosolids into a stable humic material that could be land applied (Tsang and Hurdle, 1991). Composting (including co-composting) of WWTP biosolids, considered a beneficial reuse option, has been increasing as a disposal practice in the United States (Bowen et al., 1991). The problems associated with composting WWTP biosolids are generally related to the presence of pathogens and odors. The addition of WTP residuals to the WWTP waste stream would not compound these problems, and could act to dilute them. The resulting lower volatile solids content of the biosolids, however, could inhibit the degradation process (Tsang and Hurdle, 1991) by reducing the level of biological activity.

Studies conducted in Greenwich, Connecticut, and Myrtle Beach, South Carolina, indicate that if managed properly, composting WTP residuals and WWTP biosolids could be an attractive alternative.

Vandermeyden and Potter (1991) reported on full-scale tests of joint composting of alum residuals and wastewater biosolids. A 25-percent mix ratio (residuals/biosolids) appeared to be the maximum ratio for attaining proper composting temperatures for pathogen destruction and for maintaining levels of volatile solids. No observable differences in color or physical characteristics were noted between compost with mix ratios of 25 and 12.5 percent and virgin biosolids compost. The introduction of metals from the alum residuals did not produce a degradation in compost quality and actually caused a higher allowable loading rate for the finished compost, based on a lowering of the copper and zinc concentrations in the final compost.

Chapter 7
Landfill Options

As a drinking water utility evaluates its treatment plant residuals management options, landfilling will most likely be considered. In nearly every case, landfilling falls into two categories: co-disposal and monofilling. This discussion assumes that the water treatment plant (WTP) residuals to be managed have not been classified as hazardous wastes. When the WTP residuals exhibit hazardous characteristics, they must be handled as RCRA Subtitle C wastes. In most instances, WTP residuals will be classified as nonhazardous solid waste, subject to the requirements of RCRA Subtitle D.

Under RCRA Subtitle D regulations (40 CFR Parts 257 and 258), criteria have been established for the design and operation of nonhazardous solid waste landfills. The requirements of 40 CFR Part 257 apply to landfills that receive only WTP residuals (monofills), as well as landfills that accept solid waste other than household waste (e.g., industrial waste). These criteria, which are performance based and do not include any specific design criteria, address seven landfill design and operation areas (see Table 7-1).

Table 7-1. Solid Waste Landfill Criteria: Monofill for WTP Residuals and Co-disposal of Residuals With Nonhousehold Solid Waste (40 CFR Part 257)

Section	Subpart
Floodplains	3-1
Endangered species	3-2
Surface water	3-3
Ground water	3-4
Disease	3-6
Air	3-7
Safety	3-8

Municipal solid waste landfills (MSWLFs) are subject to the criteria of Part 258. Like the provisions of Part 257, Part 258 covers performance-based criteria, but it also includes specific design criteria. If a utility disposes of its drinking water residuals in a monofill, then Part 258 criteria do not apply. If, however, the WTP residuals are co-disposed of with municipal solid waste, including household waste, the requirements established for MSWLFs apply. The landfill criteria in 40 CFR Part 258 address six major areas, as listed in Table 7-2.

Table 7-2. Solid Waste Landfill Criteria: Co-disposal of WTP Residuals With Municipal Solid Wastes (40 CFR Part 258)

Section	Subpart
Location restrictions	B
Operating criteria	C
Design criteria	D
Ground-water monitoring and corrective action	E
Closure and postclosure care	F
Financial assurance criteria	G

RCRA Subtitle D establishes minimum criteria to be applied in all states. In addition, each state may develop its own landfill requirements provided that the criteria at least meet the minimum federal standards. In many states, landfill requirements are more restrictive than federal criteria. Many states have developed unique solid waste landfill programs that reflect specific statewide factors and concerns.

Any evaluation of landfilling options for the management of WTP residuals should include a careful examination of state landfill requirements. In many states, the flexibility exists to design and operate landfills according to site-specific conditions and limitations.

7.1 Landfill Siting

Landfill siting criteria usually address two concerns—landfill performance and protection of public health and the environment. Restrictions on siting a landfill in or near airports, floodplains, wetlands, fault areas, seismic impact zones, and unstable areas are typically established to protect the landfill's integrity, thus protecting the public and minimizing environmental damage should the landfill fail.

7.1.1 Airports

Scavenger birds such as crows, gulls, and starlings are the most common aircraft safety concern at landfills, especially those that receive household solid waste. Birds are attracted to landfills to nest, roost, and search for food and water. Therefore, regulations restrict the construction and operation of landfills in areas surrounding airports. For example, MSWLF criteria (40 CFR Part 258) require that owners/operators of landfills within 10,000 feet of the end of a runway used by turbojet aircraft and within 5,000 feet of the end of a runway used only by piston-type aircraft demonstrate that design and operation of the landfill does not pose a bird hazard to aircraft. In addition, landfills within a 5-mile radius of the end of a runway must notify the Federal Aviation Administration of their existence.

For monofills, the issue of airport safety may only be a concern if co-disposal with household solid waste is used.

Reducing bird problems at MSWLFs is difficult because of the tenacity of scavenger birds. Operational practices that can minimize the attraction of birds to the landfill include:

- Frequent covering of wastes that provide a source of food.
- Baling, milling, or shredding food-containing wastes.
- Attempting to eliminate food-type wastes from the landfill using waste management techniques such as source separation and composting.

7.1.2 Floodplains

Floodplains are areas inundated by water during a 100-year flood. These areas usually include lowland and relatively flat areas next to inland and coastal waters, including flood-prone offshore islands. Landfills located in floodplains can restrict river flow during flood events, reduce the temporary water storage capacity of the floodplain, or fail, resulting in the washout of solid waste which can pose a threat to human health and the environment. Therefore, siting landfills in floodplains is usually restricted.

In most cases, these restrictions do not prohibit the location of a landfill in a floodplain, but a technically sound demonstration of the design and operational aspects of the landfill that address these concerns is needed before the applicable regulatory approvals can be secured. At a minimum, the technical demonstration requires the landfill owner/operator to identify the boundaries of the floodplain and its relation to the landfill and future landfill expansions, and to address engineering considerations. The velocity during base flow and flood flow conditions and the temporary water storage capacity of the floodplain must be determined. Modeling of the effects of the landfill on these velocity and storage volume characteristics is usually required. The U.S. Army Corps of Engineers has developed several computer models to assist in the evaluation of these flow and storage parameters.

Cost-effective methods to adequately protect a landfill from flood damage have been developed. These methods include using rip-rap and geotextiles to prevent erosion or other damage to the landfill.

7.1.3 Wetlands

With the concern over increasing loss of wetland resources, the location of landfills in or near wetlands is severely restricted. The concern relates to the effects of construction and operation on surface water quality and fish, wildlife, and other aquatic resources and their habitat, especially endangered or threatened species.

Requirements for the protection of wetlands are found in numerous federal statutes and orders including:

- Clean Water Act (Sections 401, 402, and 404)
- Rivers and Harbors Act of 1989
- Executive Order 11990, Protection of Wetlands
- Nation Environmental Policy Act (NEPA)
- Migratory Bird Conservation Act
- Fish and Wildlife Coordination Act
- Coastal Zone Management Act
- Wild and Scenic Rivers Act
- National Historic Preservation Act

Besides federal requirements, most states and many local jurisdictions have requirements to safeguard the integrity of wetlands. The use of wetlands for the construction of a landfill requires a permit from the U.S. Army Corps of Engineers, which oversees wetlands under authority delegated by the U.S. Environmental Protection Agency (EPA). Usually, owners/operators are required to demonstrate that no other alternatives exist to siting a landfill in a wetland, and only under very limited conditions will landfill construction in wetlands be approved. Owners/operators are required to develop extensive plans for mitigating and offsetting wetland impacts, including the creation of wetlands in other areas.

7.1.4 Fault Areas

Many landfill siting restrictions include a prohibition against locating a landfill within a certain distance from an earthquake fault. Seismologists generally believe that the structural integrity of an engineered unit cannot be unconditionally guaranteed when built within 200 feet of a fault along which movement is highly likely to occur.

MSWLF criteria (40 CFR Part 258) have established 200 feet as the minimum allowable distance between a fault and a landfill. Fault areas affected by the 200-foot limitation are those that have experienced movement in Holocene time, which is approximately during the last 10,000 years.

The U.S. Geological Survey (USGS) has developed maps that identify the location of Holocene faults in the United States. Landfill owners/operators should use these maps to determine the location of fault areas that may affect the location of a proposed landfill. A state may allow the construction of a landfill within a fault area if a detailed engineering demonstration of the survivability of the landfill has been developed. Any variance from these criteria requires a demonstration that no other practicable waste management alternative exists.

7.1.5 Seismic Impact Zones

In addition to fault areas, most landfill siting restrictions include consideration of seismic impact zones. A seismic impact zone (40 CFR 258.14) is an area in which a greater than 10 percent chance exists that the maximum horizontal acceleration in the bedrock, expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10 g in 250 years. Owners/operators can review the seismic 250-year interval maps in the USGS report entitled, *Probabilistic Estimates of Maximum Acceleration of Velocity in Rock in the Contiguous United States* (USGS, 1982). In addition, the National Earthquake Information Center at the Colorado School of Mines in Golden, Colorado, can provide seismicity maps of all 50 states.

For landfills to be located in seismic impact zones, an evaluation of the seismic effects should consider both foundation soil stability and waste stability under seismic loading. The evaluation of waste stability for monofills should be more reliable because of the homogeneous nature of the waste compared with the heterogeneous waste properties of municipal solid waste.

While no standard procedures exist for designing landfill components to withstand seismic events, engineering evaluations should be conducted on the influence of local soil conditions on ground response and shaking intensity, soil settlement, soil liquefaction, and slope stability during earthquakes.

7.1.6 Unstable Areas

An unstable area is an area susceptible to natural or manmade events or forces capable of reducing the integrity and performance of all or some of the landfill components. Factors that contribute to inadequate support are poor foundation conditions; down-slope movement of soil, rock, or debris caused by the influence of gravity; and sinkholes resulting from Karst terraces underlain by soluble bedrock that may contain extensive subterranean drainage systems and relatively large subsurface voids.

The construction and operation of landfills in unstable areas should be avoided. If built, the landfill should be engineered and constructed to accommodate the conditions that classify the area as unstable. A detailed geotechnical and geological evaluation of a landfill site should be conducted to assess the subsurface under natural and manmade conditions. This evaluation should analyze the potential for inadequate support for the structural components of the landfill.

7.2 Landfill Design

As previously discussed, monofills are subject to the performance-based criteria specified in 40 CFR Part 257. These criteria require that the owner of a monofill, usually the drinking water utility, demonstrate that the construction of such a landfill will not adversely affect surrounding floodplains, endangered species, surface water, ground water, or air quality, and will not create disease or safety threats. The criteria do not specify design considerations that must be followed. Table 7-3 provides some general guidance on the type of information that is necessary before design begins.

The major sludge monofilling methods are trench filling and area filling. Trench filling can be further subdivided into narrow trench and wide trench monofilling techniques. Area filling methods include area fill mound, area fill layer, and diked containment. Sludge solids content, sludge stability, site hydrogeology (location of ground water and bedrock), ground slope, and land availability determine the monofilling method that is selected. The following sections describe specific aspects of the various sludge monofilling methods. Figure 7-1 highlights the design consideration for monofills.

A landfill that accepts municipal solid wastes is subject to the criteria described in 40 CFR Part 258. Under federal regulations, WTP residuals are subject to the provisions of Part 258 only if they are co-disposed of in a landfill that also accepts municipal solid waste. In that case, the landfill is designated as an MSWLF.

Landfill design under Part 258 can be based on performance or minimum technology standards. These two approaches are described in detail below.

7.2.1 Performance-Based Design Under 40 CFR Part 258

Performance-based landfill designs are developed so that the landfill system will result in compliance with established performance standards. By allowing a designer to use a performance-based approach, site-

Table 7-3. Field Investigations for New Information, Landfill Design (U.S. EPA, 1978b)

General Information	Specific Information	Method and Equipment
Base Map	Property boundaries	Field survey
	Topography and slopes	Field survey
	Surface water	Field survey
	Utilities	Field survey
	Roads	Field survey
	Structures	Field survey
	Land use	Field survey
	Vegetation	Field survey
Soils	Depth	Soil boring and compilation of boring log
	Texture	Soil sampling and testing via sedimentation methods (e.g., sieves)
	Structure	Soil sampling and inspection
	Bulk density	Soil sampling and testing via gravimetric, gamma ray detection
	Porosity	Calculation using volume of voids and total volume
	Permeability	Soil sampling and testing via piezometers and lysimeters
	Moisture	Soil sampling and testing via oven drying
	Ease of excavation	Test excavation with heavy equipment
	Stability	Test excavation of trench and loading of sidewall of Hueem stabilimeter
	pH	Soil sampling and testing via pH meter
	Cation exchange capacity	Soil sampling and testing
Bedrock	Depth	Boring and compilation of boring log
	Type	Sampling and inspection
	Fractures	Field survey
	Surface outcrops	Field survey
Ground water	Depth	Well installation and initial readings
	Seasonal fluctuations	Well installation and year-round comparison of readings
	Hydraulic gradient	Multiple well installation and comparison of readings
	Rate of flow	Calculation based on permeability and hydraulic gradient
	Quality	Ground-water sampling and testing
	Uses	Field survey via inspection
Climatology	Precipitation	Rain gauge
	Evaporation	Class A Evaporation Pan
	Temperature	Standard thermometer
	No. of freezing days	Minimum-maximum temperature thermometer
	Wind direction	Wind arrow

specific factors can be considered in the design such as:

- Hydrogeologic characteristics of the landfill and the surrounding land.
- Volume and physical and chemical characteristics of the leachate.
- Quality, quantity, and direction of ground-water flow.
- Proximity and withdrawal rate of ground-water users.
- Existing quality of the ground water.

The MSWLF criteria require compliance with performance standards that set maximum contaminant levels (MCLs) (Table 7-4) not to be exceeded in the ground water at the relevant point of compliance. The MSWLF criteria allow for a relevant point of compliance to be established as far as 150 meters from the waste management unit boundary. In some cases, the relevant point of compliance must be at the waste management unit boundary.

Successful application of the performance-based approach, leading to a reliable landfill system design, depends on the designer's ability to accurately model the rate of pollutant movement through the landfill system and the site stratigraphy. Contaminant transport at the landfill site must be studied carefully to determine the direction, speed, and concentration of contaminant flow. Because contaminant transport in ground water can be very complicated, accurate prediction of contaminant movement significantly increases the cost of design and site characterization for an MSWLF (U.S. EPA, 1994a).

7.2.2 Minimum Technology-Based Design Under 40 CFR Part 258

The MSWLF criteria have established minimum technology-based design standards for landfills that cannot apply performance-based design standards. Technology-based standards require a composite liner system consisting of an upper geomembrane liner and a lower compacted-soil liner. The geomembrane liner minimizes the exposure of the compacted soil liner to leachate, thus significantly reducing the volume of leachate reaching the soil liner. Reducing membrane penetration is vital to controlling the escape of leachate into ground water. The geomembrane must be at least 30 millimeters thick; high density polyethylene (HDPE) geomembranes must be at least 60 millimeters thick. The compacted soil liner must be at least 2 feet thick and have a hydraulic conductivity of less than 1 x 10^{-7} centimeters per second.

Many state MSWLF programs have established alternative technology-based design requirements. Whenever technology-based designs are being considered, the state MSWLF program should be contacted to obtain applicable design requirements.

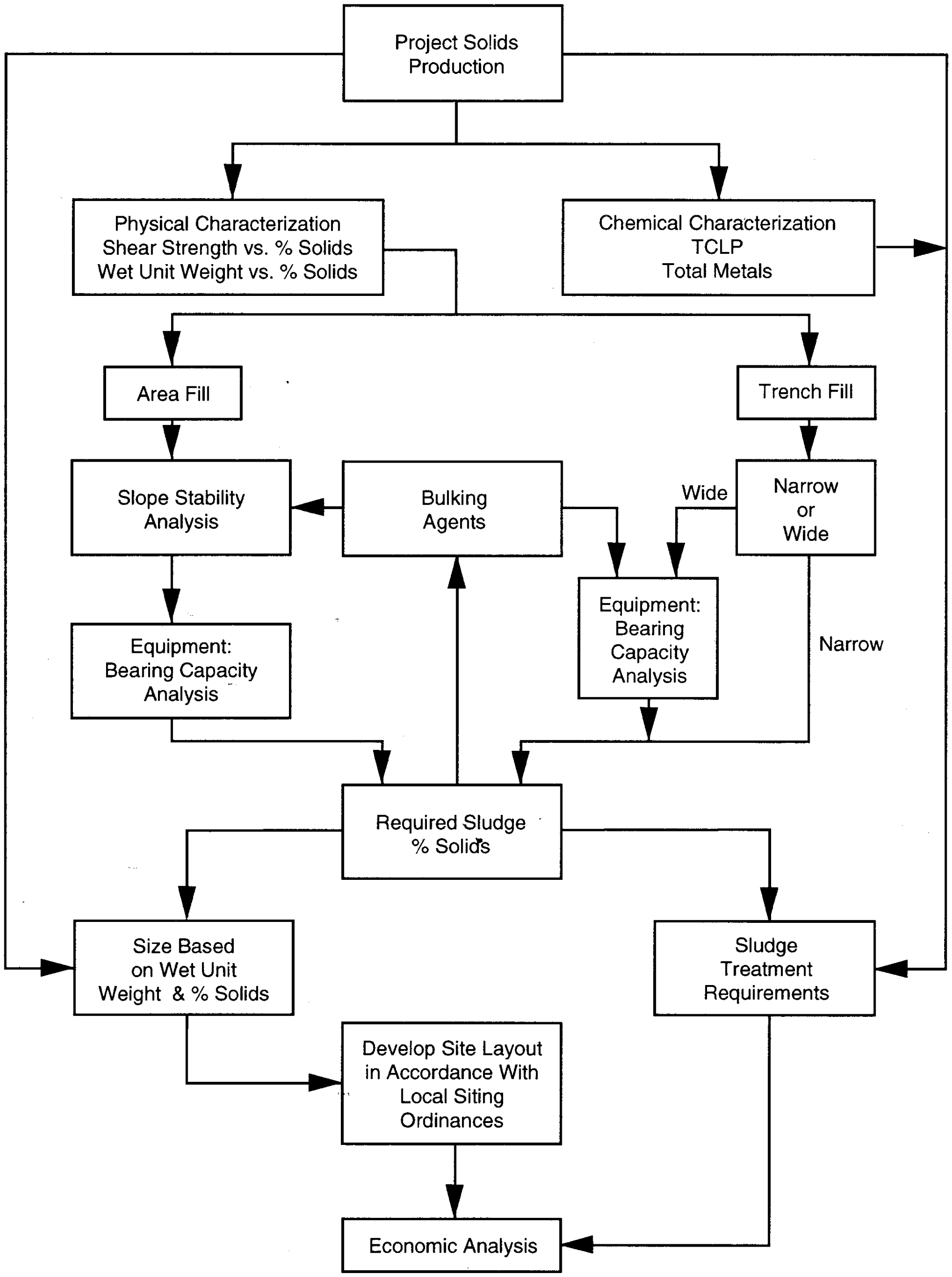


Figure 7-1. Considerations for sludge monofill design (Cornwell et al., 1992).

Many factors must be considered for a successful geomembrane design and installation:

- Selection of proper geomembrane materials that consider chemical resistance and biaxial stress-strain properties.
- Proper subgrade preparation to ensure that the compacted soil liner is smooth and strong enough to provide continuous support for the geomembrane.
- Proper geomembrane transportation, storage, and placement.
- Favorable installation conditions, including fair weather, low winds, and proper temperature.
- Proper geomembrane seaming and seam testing.
- Development and implementation of a reliable and reasonable construction quality assurance (CQA) program.

Because of its low hydraulic conductivity characteristics, clay is usually the soil of choice for compacted soil liners. Unfortunately, clay is a difficult engineering material to work with because of its highly moisture-dependent physical properties. As a basic landfill liner,

Table 7-4. Maximum Contaminant Levels in Uppermost Aquifer at Relevant Point of Compliance (Fed. Reg., October 9, 1991)

Parameter	MCL (mg/L)
Arsenic	0.05
Barium	1.0
Benzene	0.005
Cadmium	0.01
Carbon tetrachloride	0.005
Chromium (hexavalent)	0.05
2,4-Dichlorophenoxy acetic acid	0.1
1,4-Dichlorobenzene	0.075
1,2-Dichloroethane	0.005
1,1-Dichloroethylene	0.007
Endrin	0.0002
Fluoride	4.0
Lindane	0.004
Lead	0.05
Mercury	0.002
Methoxychlor	0.1
Nitrate	10.0
Selenium	0.01
Silver	0.05
Toxaphene	0.005
1,1,1-Trichloromethane	0.2
Trichloroethylene	0.005
2,4,5-Trichlorophenoxy acetic acid	0.01
Vinyl chloride	0.002

clay must meet certain criteria to protect ground water from leachate contamination. To meet the requirements of the MSWLF criteria and to produce a reliable barrier against leachate movement, the following steps should be taken during construction of a compacted clay liner: destroy soil clods, eliminate lift interfaces, conduct proper compaction, meet moisture-density criteria, and avoid desiccation.

To minimize holes in a geomembrane liner caused by product defects, transportation, installation, and seaming, and to produce a reliable soil liner that prevents the movement of leachate into the ground water, a CQA program should be developed and implemented (U.S. EPA, 1993a). The CQA program is a planned system of activities implemented by the landfill owners or their representatives, to ensure that the components of the landfill system are constructed as specified in the design. The CQA program should be developed along with the landfill design. All involved state and federal regulators should review the CQA program before approvals and permits for construction are issued.

CQA is different and distinct from construction quality control (CQC). CQC is a program that the construction contractor develops and implements to ensure the quality of its work. In contrast, the CQA program is an additional check of the construction contractor's work conducted by the owners of the landfill. Several key components to a successful CQA program are:

- *Responsibility and authority:* Landfill owners give CQA personnel the responsibility and authority to represent their interest and to ensure that the landfill components meet design specifications.
- *Personnel qualifications:* The CQA inspector must have extensive experience and knowledge about the work performed in the field. A program administered by the National Institute of Certifying Engineering Technicians (NICET) gives formal examinations and certifies CQA inspectors.
- *Inspection activities:* The CQA program must clearly define the testing program and acceptance criteria for significant components of the landfill system. For the liner system, the CQA program should specify the frequency of testing to be done on the compacted soil and geomembrane liner, outline the sampling strategy, and define the specific tests to be performed.
- *Sampling strategies:* CQA testing is performed using a combination of statistical and judgmental sampling strategies. Typical statistical sampling strategies include defined interval testing, such as one destructive seam test per 5,000 feet of geomembrane seam or one moisture/density test per 5,000 cubic yards of soil liner. Judgmental testing allows the CQA inspector to call for testing when the quality of workmanship is suspect.
- *Documentation:* Before a permit to operate the landfill is issued, most states now require documentation that a CQA program was performed. All CQA activities must be clearly documented so that a third party can understand and verify the testing and inspection program.

Another requirement of technology-based systems in the design and construction of a leachate collection system is to collect leachate and convey it out of the landfill. The MSWLF criteria require this system to ensure that less than 30 centimeters of leachate accumulates over the composite liner system to minimize hydraulic head and possible contamination of the ground water. When a leachate collection system is designed and constructed, the following components should be included:

- *Area collector:* the drain that covers the liner and collects the leachate.

- *Collection laterals:* the pipe network that drains the area collector.
- *Sump:* the low point where leachate exits the landfill.
- *Stormwater/Leachate separation system:* a system for minimizing leachate generation and possible treatment costs.

These components must be designed to handle larger leachate flows associated with initial operations and to resist problems such as biological clogging that can destroy the long-term performance and flow capacity of the system.

7.3 Landfill Operations

To meet the regulatory requirements for liners and leachate collection systems, landfill design and construction have become increasingly complex. Thus, the integrity of landfill components can be threatened by careless or inappropriate operations. Facility operators should be fully aware of landfill operational requirements and the reasons for these requirements to ensure that the landfill performs as designed, especially in situations where the reason for some operational procedures may not be readily apparent. At the same time, landfill designers and regulators should obtain feedback from landfill operators on day-to-day operational requirements. A complex, sophisticated design that cannot be operated in the field will not achieve its intended purpose.

Operational requirements for landfills are designed to ensure the safety of people on the landfill, including landfill operators, waste haulers, and the public, and to protect the environment. MSWLF criteria require the following list of measures to be implemented at all landfills:

- Exclude hazardous waste and polychlorinated biphenyls (PCBs).
- Provide daily cover.
- Control onsite disease vectors.
- Provide routine methane monitoring.
- Eliminate most open burning.
- Control public access.
- Institute run-on and runoff controls.
- Control discharges to surface waters.
- Eliminate the disposal of most liquid wastes.
- Keep records that document implementation of operational requirements.

7.4 Metal Content Considerations

The most prominent environmental consideration in the siting, construction, and operation of a residuals monofill under 40 CFR Part 257 is the potential for ground-water contamination through the leaching of metal constituents in the residuals. Assessing the potential for contamination involves determining the metals concentration in the residuals and the degree to which these metals can be expected to mobilize into ground water.

The major sources of metals are raw water and treatment chemicals. Table 6-2 presents the findings of one investigation into the metals content of water treatment chemicals (Dixon et al., 1988). Aluminum is, of course, a major constituent of alum coagulant, and iron is a major constituent in ferric chloride coagulant; these metals predominate in WTP residuals as a result. The data in Table 6-2 indicate that many heavy metal impurities can exist in coagulants and in other treatment chemicals. Raw water sources also can contain metals naturally or because of source contamination. Raw water metals largely wind up in WTP residuals.

Metals levels in both treatment chemicals or raw water can vary, especially in the case of raw water. Because metals concentrations in WTP residuals vary widely at different water treatment facilities, a general characterization of metals concentrations in WTP residuals is difficult to make. Such a characterization is complicated by the lack of comprehensive surveys of metals concentration in WTP residuals.

Nonetheless, a rough characterization of typical WTP residuals metals concentrations is presented in Table 7-5. The number of samples from which each range was compiled should be noted when interpreting the ranges. The American Water Works Service Co. survey of 19 water plants in Pennsylvania covered only seven metals—cadmium, chromium, copper, lead, mercury, nickel, and zinc (Dixon et al., 1988)—which accounts for the disparity in sample set sizes.

Typical metals concentration ranges in natural soils and sewage sludge are provided in Table 7-5 for comparison with WTP residuals (Dragun, 1988). Median concentrations for sewage sludge are also presented (U.S. EPA, 1984b). The concentration data for both natural soils and sewage sludge are based on more comprehensive information than could be compiled for WTP residuals. The three substances covered in Table 7-5 are roughly comparable. WTP residuals usually contain much lower metals levels than typical sewage sludge contains and often levels are very similar to those found in natural soils.

Table 7-5. Comparison of Metals Concentrations in WTP Residuals, Natural Soils, and Sewage Sludge

Metals	Number of WTP Residuals Samples Included in Compilation	Range of Concentrations in WTP Residuals (mg/kg, dry)[a]	Average Concentration in Compiled WTP Residuals Samples (mg/kg, dry)[a]	Typical Range of Concentrations in Natural Soils (mg/kg, dry)[b]	Typical Range of Concentrations in Sewage Sludge (mg/kg, dry)[c]	Median Concentration in Sewage Sludge (mg/kg, dry)[c]
Aluminum	7	10,000–170,000	72,729	10,000–300,000	N/A	N/A
Arsenic	6	5.7–36	20	1.0–40	1.1–230	10
Barium	6	30–333	131	100–2,500	N/A	N/A
Cadmium	24	0–16	3.3	0.01–7.0	1–3,410	10
Chromium	26	6.7–200	59	5.0–3,000	10–99,000	500
Copper	25	7–1,300	176	2.0–100	84–17,000	800
Iron	4	15,200–79,500	51,400	7,000–550,000	1,000–154,000	17,000
Lead	26	1–100	50	2.0–200	13–26,000	500
Manganese	5	68–4,800	1,453	100–4,000	32–9,870	260
Mercury	24	0–9.8	0.9	0.01–0.08	0.6–56	6
Nickel	23	12–1,319	209	5.0–1,000	2–5,300	80
Selenium	5	0–36	8	0.1–2.0	1.7–17.2	5
Silver	4	0–2	1	0.1–5.0	N/A	N/A
Zinc	25	6.2–3,325	598	10–300	101–49,000	1,700

[a] Cornwell et al., 1992; Dixon et al., 1988.
[b] Dragun, 1988.
[c] U.S. EPA, 1984b.
Key
N/A = not analyzed.

7.4.1 Classification as Hazardous or Nonhazardous Waste

The total metals concentrations in a material do not, by themselves, give a good indication of the potential for a material to contaminate ground water with metals in a landfill situation; the degree to which the metal constituents readily solubilize into ground water is an equally important factor. Laboratory extraction testing is one method to assess the extent to which a material's metals will readily solubilize.

As part of the RCRA Subtitle C regulations, EPA has defined four extrinsic characteristics, any one of which can qualify a waste as hazardous. The four characteristics are reactivity, corrosivity, ignitability, and toxicity. Since the inception of the RCRA regulations, toxicity has been defined in terms of extraction testing results. Prior to May 1990, the extraction procedure (EP) toxicity test was the standard test for determining toxicity. Since May 1990, the standard test has been the toxicity characteristics leaching procedure (TCLP) test. Both tests involve the preparation of an aqueous extract from a waste sample, which is in turn analyzed for a number of toxic parameters including metals (the same eight in each test) and organic compounds.

Although extraction testing is the means by which a substance is determined to be toxic, according to the RCRA regulations, it is also a means for determining the mobility of toxic analytes in a waste that is not toxic. For WTP residuals, failure of the TCLP test is not likely because the metals content of most residuals samples is low enough that even if 100 percent of each metal were extracted into the extraction fluid, the TCLP concentration limits would not be exceeded (Cornwell et al., 1992).

Table 7-6 demonstrates this point by presenting the TCLP concentrations that would result from 100 percent extraction of the metals in a residuals sample having metals concentrations equal to the maximums given in Table 7-5. These TCLP concentrations are based on the assumption of a 35 percent solids sample. As part of the TCLP test, samples are subjected to pressure filtration to remove excess water. Dewatered coagulant residuals will not yield any water when subjected to pressure filtration and will, therefore, be tested at their dewatered solids concentration. Even 100 percent extraction of the TCLP metals from a hypothetical dewatered residuals sample that contained all the maximum metals concentrations given in Table 7-5 would not cause an exceedance of any of the TCLP regulatory limits. This clearly indicates that WTP residuals should rarely, if ever, qualify as toxic under RCRA regulations.

This conclusion is supported by actual extraction test results for WTP residuals. Table 7-7 presents actual results for EP toxicity tests performed on several WTP residuals samples and Table 7-8 presents TCLP results for WTP residuals from five facilities. In all cases, no TCLP regulatory limits were even approached (Cornwell et al., 1992). In fact, TCLP metals often cannot be detected in extraction fluid because concentrations are so low.

Table 7-6. Worst Case TCLP Results Using Maximum Metals Concentrations in Table 7-5, Compared With TCLP Regulatory Limits (Cornwell et al., 1992)

Metal	Maximum Possible Concentration in Extraction Fluid (mg/L)	TCLP Regulatory Limit (mg/L)
Arsenic	0.63	5.0
Barium	5.83	100.0
Cadmium	0.28	1.0
Chromium	3.50	5.0
Lead	1.75	5.0
Mercury	0.17	0.2
Selenium	0.63	1.0
Silver	0.04	5.0

Because the detection limits of metals are often encountered in such testing, it is difficult to measure the percentage of a metal's original amount. The results from pilot lysimeter testing, however, often can be expressed in terms of percentages of metals extracted.

7.4.2 Mobility of Trace Metals

Lysimeter testing can provide additional insight into the mobility of metals from WTP residuals. A recent study used the lysimeter setup shown in Figures 7-2A to 7-2C to test the leachability of metals from three residuals samples (Cornwell et al., 1992). The residuals used in this lysimeter test came from three WTPs. Two alum residuals samples were used, one from the Williams WTP in Durham, North Carolina, and one from the Chesapeake WTP in Chesapeake, Virginia. The third residuals sample was a ferric chloride sample from the Aldrich WTP operated by the Pennsylvania-American Water Co. TCLP test results are presented in Table 7-8.

The study lasted 24 weeks. Each week, 44.1 liters of simulated rainwater were pumped over a 24-hour period into each lysimeter column. The simulated rainwater was made in the laboratory, having a pH of 4.5 and containing the constituents listed in Table 7-9. A perforated plate was used to hold the rainwater and allow it

Table 7-7. EP Toxicity Test Results for Alum Residuals (Cornwell et al., 1992)

Contaminant	Regulatory Threshold (mg/L)	Saltonstall, CT (mg/L)	West River, CT (mg/L)	City of Chesapeake, VA (mg/L)	American Water Works Service Co. 66-Plant Survey (mg/L)
Arsenic	5.0	<0.01	<0.01	<0.003	<0.2–0.4
Barium	100.0	0.21	0.1	<0.1	<0.1–34.0
Cadmium	1.0	<0.005	<0.005	0.005	<0.005–0.06
Chromium	5.0	<0.01	<0.01	<0.05	<0.1–3.8
Lead	5.0	<0.01	<0.001	<0.001	<0.0004–0.003
Mercury	0.2	<0.001	<0.001	<0.001	<0.0004–0.003
Selenium	1.0	0.09	<0.01	<0.007	<0.001—0.08

Table 7-8. TCLP Results for Five Water Treatment Plant Coagulant Residuals (Cornwell et al., 1992)

Contaminant	Regulatory Threshold Level (mg/L)	Contra Costa, CA, Water District Alum Sludge (mg/L)	Phoenix, AZ, Alum Sludge (mg/L)	Durham, NC, Alum Sludge (mg/L)	Chesapeake, VA, Alum Sludge (mg/L)	Aldrich, PA, WTP Ferric Sludge (mg/L)
Arsenic	5.0	0.04	<0.3	0.0088	0.0023	0.006
Barium	100.0	1.1	1.1	1	<1	<2
Cadmium	1.0	<0.05	<0.02	<0.02	<0.02	0.02
Chromium	5.0	0.06	<0.04	<0.1	<0.1	<0.1
Lead	5.0	<0.2	<0.5	<0.1	<0.1	<0.1
Mercury	0.2	<0.001	<0.01	<0.002	<0.002	<0.002
Selenium	1.0	<0.02	<1	<1	<0.002	<0.002
Silver	5.0	<0.05	<0.01	<0.1	<0.1	<0.1

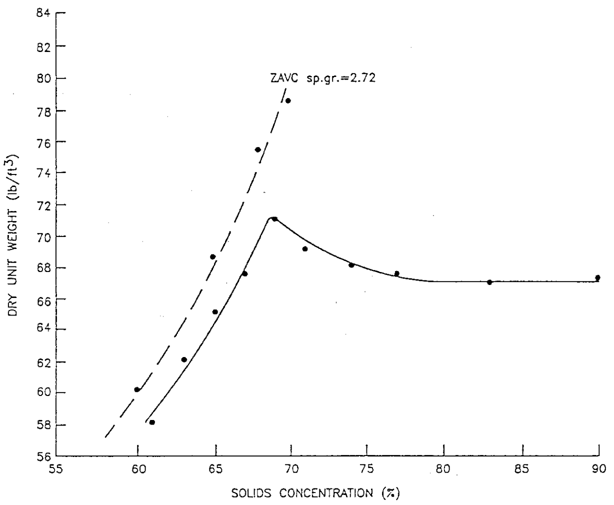


Figure 7-2A. Compaction curve, Ferric Sludge 3 (Cornwell et al., 1992).

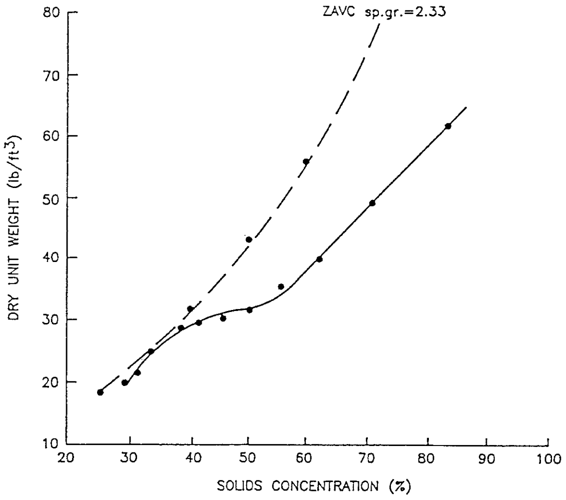


Figure 7-2B. Compaction curve, Alum Sludge 1 (Cornwell et al., 1992).

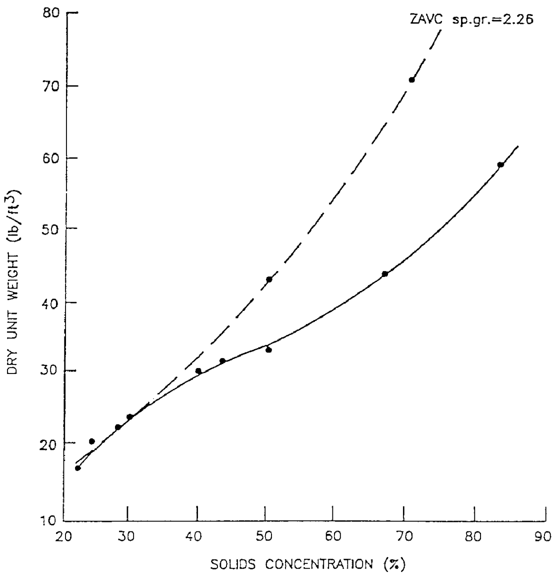


Figure 7-2C. Compaction curve, Alum Sludge 2 (Cornwell et al., 1992).

Table 7-9. Chemical Constituents of Synthetic Rainwater (Cornwell et al., 1992)

Constituent	Concentration (mg/L)
NH^3	0.045
CA^{2+}	0.14
Mg^{2+}	0.073
Na^+	0.46
K^+	0.078
Cl^-	1.63
N^{3-5}	0.036
SO_42-	1.54
pH	4.5

to drip to the surface of each lysimeter in a manner similar to rainfall. Residuals were placed loosely in each lysimeter column to allow the rainwater to percolate through. Because residuals are a nearly impermeable material, had they been compacted the water would have formed a pond at the top of the columns and not drained through. Rainwater was allowed to sit in the columns for a week before being drained for sampling. The total amount of rainwater applied to each lysimeter was equivalent to 450 inches of rainfall, about 12 years of rain in Virginia. The total metals concentrations for these residuals are presented in Table 7-10.

Table 7-11 summarizes the results from the lysimeter study in terms of the percentages of each metal leached. Manganese was the only metal that leached at a rate of over 1 percent in all three residuals samples. This lysimeter study clearly indicates that not only do WTP residuals generally contain small concentrations of metals, but only small portions of these metals tend to leach. In the test, aluminum, barium, chromium, lead, and silver did not leach measurably from any of the residuals samples. Cadmium and nickel leached only from the ferric chloride residuals. Arsenic, copper, iron, manganese, and zinc leached from all the residuals samples.

Table 7-12 presents the maximum leachate concentrations of each metal measured as part of the lysimeter study. As indicated in the table, no primary MCLs were exceeded in any of the leachate samples, although the secondary MCLs (SMCLs) for iron and manganese were exceeded. The iron SMCL was exceeded in the leachate of one of the alum residuals samples and the ferric residuals sample, while the manganese SMCL

Table 7-10. Total Metals Analysis for Sludges Used in Leaching Research (Cornwell et al., 1992)

Metal	Durham, NC, Alum Sludge 1 (mg/kg dry weight)	Chesapeake, VA, Alum Sludge 2 (mg/kg dry weight)	Aldrich, PA, WTP Ferric Sludge 3 (mg/kg dry weight)
Aluminum	107,000	123,000	28,600
Arsenic	25.0	32.0	9.2
Barium	30	<30	230
Cadmium	1	1	2
Chromium	120	130	50
Copper	168	16	52
Iron	48,500	15,200	79,500
Lead	11	9	40
Manganese	1,180	233	4,800
Mercury	0.1	<0.1	0.2
Nickel	24	23	131
Selenium	<2	<2	<2
Silver	<2	<2	<2
Zinc	91.7	393	781

Table 7-11. Leaching of Metals in Lysimeter Test (Cornwell et al., 1992)

	Percentage of Total Metal Present That Actually Leached		
Metal	Alum Sludge 1	Alum Sludge 2	Ferric Sludge 3
Aluminum[a]	0.00	0.00	0.00
Arsenic	0.05	0.05	2.77
Barium	0.00	0.00	0.00
Cadmium	0.00	0.00	2.03
Chromium	0.00	0.00	0.00
Copper[a]	0.12	0.42	0.08
Iron[a]	0.03	0.05	0.01
Lead	0.00	0.00	0.00
Manganese[a]	12.48	3.38	2.45
Nickel[a]	0.00	0.00	0.03
Selenium[b]	—	0.00	—
Silver	0.00	0.00	0.00
Zinc[a]	0.13	0.05	0.08

[a] These metals were not analyzed in a standard TCLP test.
[b] Selenium leaching could not be quantified.

was exceeded in the leachate from all three residuals samples. These SMCLs are not a part of the list of indicator parameters for ground-water protection monitoring given in federal municipal landfill regulations.

Table 7-12. Maximum Metals Concentrations in Lysimeter Leachate Compared With MCLs (Cornwell et al., 1992)

Metal	Durham, NC, Alum Sludge 1 (mg/L)	Chesapeake, VA, Alum Sludge 2 (mg/L)	Aldrich, PA, WTP Ferric Sludge 3 (mg/L)	MCL (mg/L)
Arsenic	0.0035	0.0019	0.0366	0.05
Cadmium	—	—	0.01	0.01
Copper	0.12	0.03	—	1.0
Iron	2.00	0.30	1.00	0.3[a]
Manganese	9.50	0.36	22.80	0.05[a]
Nickel	—	—	0.06	N/A
Zinc	0.036	0.017	0.373	5.0

[a] Secondary MCL.

Leachate pH was monitored in addition to metals concentrations as part of the lysimeter study. pH has a crucial impact on the leaching of metals from wastes. The simulated rainwater applied to each lysimeter had a pH of 4.5; the pH of leachate samples was considerably higher than this. The leachate from Durham's alum residuals had a pH range of 5.5 to 6. The other two residuals samples produced leachate in the pH range of 6.5 to 7. pH buffering was exhibited by all three residuals samples. This quality was probably a key factor in keeping the solubility of metals low and limiting metal leaching.

The testing of metals concentrations and mobility in WTP residuals indicates that leachate from WTP residuals should not, in most cases, cause ground water to exceed drinking water MCLs for metals. The permeability of the residuals was not accounted for, however, in any of the extraction or lysimeter tests. As part of an American Water Works Association Research Foundation (AWWARF) landfill study, the Unified Soil Classification System was used to determine that the residuals samples tested exhibited the characteristics of a CH soil group (i.e., inorganic clay of high plasticity). The permeability of this class of soil is typically less than 2×10^{-8} inches per second. This is a lower permeability than that of typical clay liners, which must be at least 4×10^{-8} inches per second.

7.5 Dewatering

Before landfilling, WTP residuals are often dewatered to reduce the liquid content and increase the solids concentration. This is necessary so that the residuals will have a high enough solids content to qualify for disposal in a landfill. Sludge dewatering methods are both mechanical and nonmechanical. Mechanical dewatering methods use vacuum or pressure to remove the liquid portion of the sludge, whereas nonmechanical methods depend on gravity and evaporation to reduce the undesired liquid in the sludge.

7.5.1 Mechanical Dewatering

The equipment used most frequently for mechanical dewatering includes the vacuum filter, continuous belt filter press, plate pressure filter, and centrifuge. An essential element of the vacuum filter is the rotating cylindrical drum. Lying on its side, the drum rotates around its lengthwise axis with its lower part submerged in a tank of water sludge. Its periphery surface is covered with a permeable fabric or other filter material. A vacuum is applied to suck water through the fabric. Solids left on the drum surface, called filter cake, are then removed using a scraper blade.

The continuous belt filter press is equipped with two filter belts, one on top of the other, which wind through a series of rolls. The water sludge contained between the two belts is subjected to bending and shearing action as the belts continuously move around the rolls. This bending and shearing action causes the sludge to drain and form a sludge cake.

A plate pressure filter contains a series of recessed plates covered with a filter cloth. The water treatment sludges are pumped between the plates, and the liquid seeps through the filter cloth, leaving behind the sludge cake. When the spaces between the plates are filled, the sludge cakes are removed.

The centrifuge method of dewatering uses centrifugal force to generate pore water pressure in the sludge. When pore water pressure develops, water drains out of the sludge, leaving the sludge cake in a machine. The centrifuge rotating speed typically ranges between 800 and 2,000 rotations per minute. With other parameters being equal, the faster the rotating speed, the greater dewatering power.

7.5.2 Nonmechanical Methods

In nonmechanical dewatering, WTP residuals are spread out in a lagoon or a sand drying bed, exposed to the air, and allowed to dry by evaporation.

Lagoons are large basins that are either enclosed with earthen embankments or excavated for WTP residuals deposits. They can be used for either dewatering or residuals storage. Dewatering lagoons have pervious bottoms such as sand layers with underdrains to facilitate drainage; storage lagoons must have impermeable bottoms to protect the ground water. Storage lagoons generally are equipped with a decanting facility such as pumps to remove the standing water.

Sand drying beds are shallow basins with sand bottoms and underdrain systems. They are also equipped with facilities for decanting the standing water. Sand drying beds perform the same function as dewatering lagoons; the main difference is the thickness of deposited sludge and the frequency of deposition. Sand drying beds receive thinner layers and more frequent deposition of sludges than dewatering lagoons receive.

The degree of dewatering depends on the effectiveness of the dewatering method and the water-holding capacity of the sludge. Because the water-holding capacity of sludges can vary, the obtainable solids concentration of sludge cakes also varies by both dewatering methods and sludge types.

7.6 Physical Characteristics of Water Sludges

The physical characteristics of water sludges that need to be addressed in any effective landfill plan include plasticity, compaction behavior, compressibility, and shear strength. Of these, plasticity characteristics affect sludge handleability. Compaction data, which provide moisture-density curves, are needed for landfill construction control. Compressibility data are necessary to determine landfill settlement. Shear strength affects the landfill stability, which controls the maximum height and slope of a stable landfill and the ability of the landfill to support heavy equipment.

7.6.1 Plasticity

The plasticity characteristics of water sludges can be evaluated from the value of Atterberg limits, especially liquid limit and plasticity index. The plasticity index equals the difference between the liquid limit and the plastic limit. Both liquid and plastic limits are expressed as water content in percentage. The water content is defined as the ratio between the weights of water and solid phases, which can be related to the solids content as shown in Equation 7-1, in which the solids content is the ratio between the weights of solids and the sludge mass expressed in percentage:

$$\begin{matrix}\text{water}\\\text{content}\end{matrix}\,(\%) = \left(\frac{100}{\text{solids content }(\%)} - 1\right)(100) \qquad \text{(Eq. 7-1)}$$

This equation can be reduced to:

$$\begin{matrix}\text{water}\\\text{content}\end{matrix}\,(\%) = 1 - \frac{\left(\dfrac{\text{solids content}(\%)}{100}\right)}{\dfrac{\text{solids content }(\%)}{100}} \times 100 \qquad \text{(Eq. 7-2)}$$

The Atterberg limits of WTP residuals vary greatly with the type of sludge. The lime/alum/polyamine coagulant water sludge of the Jersey City Water Treatment Plant has only slight to little plasticity (Raghu et al., 1987). As

Table 7-13. Liquid Limit, Plastic Limit, and Plasticity Index of Water Sludges (Cornwell et al., 1992)

Water Sludge	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index
Alum Sludge 1	423	137	286
Alum Sludge 2	550	239	311
Ferric Sludge 3	108	47	61

shown in Table 7-13, however, the liquid limit, plastic limit, and plasticity index of Alum Sludge 1, Alum Sludge 2, and Ferric Sludge 3 are very high (Cornwell et al., 1992). Alum Sludge 1 was produced from treatment of medium-color, medium-turbidity raw water, dewatered using the sand drying bed method and obtained from Williams Water Treatment Plant in Durham, North Carolina. Alum Sludge 2 was generated from high-color, low-turbidity raw water, dewatered using the sand drying bed method and obtained from Chesapeake Water Treatment Plant in Chesapeake, Virginia. The ferric sludge was generated from medium-color, medium-turbidity raw water, dewatered using the lagoon method and obtained from Aldrich Water Treatment Plant in Pittsburgh, Pennsylvania.

Materials with a high plasticity index are considered highly plastic and difficult to handle. Thus, of these four different sludges, the lime/alum/polyamine coagulant sludge is the easiest to handle, and Alum Sludge 2 is the most difficult.

7.6.2 Compaction Behavior

Compaction data are used to establish the degree of sludge compaction necessary to increase stability, decrease permeability, and enhance resistance to erosion. The shape of the moisture-density relation (compaction curve) varies greatly with sludge type. For the lime/alum/polyamine coagulant sludge (Raghu et al., 1987), the moisture-density curve of the modified Proctor compaction exhibits the typical one-hump shape; the optimal water content and maximum dry unit weight equal 65 percent and 51 pounds per cubic foot (lb/ft^3), respectively. The moisture-density curve of a coagulant sludge under the Standard Proctor compaction also exhibits the typical one-hump shape, with the optimal water content and maximum dry unit weight of 17 percent and 105 lb/ft^3 (16.5 kN/m^3), respectively. Of the three coagulant sludges shown in Table 7-13, however, only the ferric sludge exhibits the one-hump shape; the optimal water content and maximum dry unit weight of the Standard Proctor equal 45 percent and 72 lb/ft^3 (11.3 kN/m^3), respectively.

Both Alum Sludges 1 and 2 show a decrease in dry unit weight with increasing water content without a peak formation. Meanwhile, for Alum Sludge 2, admixing a bulking agent—a slaked lime, a class C fly ash, or a natural soil—insignificantly affects the moisture-density relation. The amount of additive equals 60 percent by dry weight of the sludge. The moisture-density relations of these three sludges are plotted in terms of percent solids concentration versus dry unit weight in Figure 7-3. Each curve is accompanied by the zero-air-void curve (ZAVC) that outlines the upper limit of the curve regardless of the compaction effort used. The ZAVC is obtained from Equation 7-3:

$$\begin{matrix}\text{dry}\\ \text{unit}\\ \text{weight}\end{matrix} = \frac{(\text{specific gravity})\ (\text{unit weight of water})}{(1 + \text{specific gravity})\left(\dfrac{\text{water content (\%)}}{100}\right)} \qquad \text{(Eq. 7-3)}$$

For landfill construction, the sludge should be compacted to the highest possible density that can be accomplished only when the amount of water in the sludge is at the optimal water content. For Alum Sludges 1 and 2, however, the compaction water content should be as low as possible.

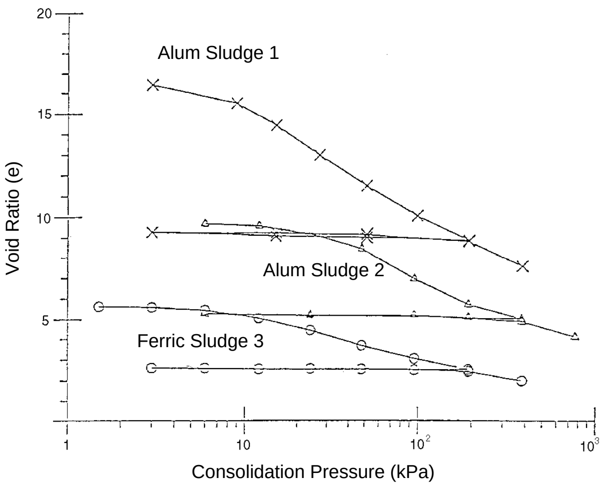


Figure 7-3. Consolidation curves of Alum Sludges 1 and 2 and Ferric Sludge 3 (Wang et al., 1992b).

7.6.3 Compressibility

In the landfill, under the sustained weight of the overlying mass, sludge may undergo volume reduction from the extrusion of water. As a result, the landfill may settle, creating air space for depositing additional sludge on top of the landfill. To determine the amount of landfill settlement, the compressibility property of the sludge is required.

Normally, the compressibility property is obtained from the consolidation test. The test results can be analyzed using the conventional small-strain theory or the finite-strain theory of consolidation. Although the small-strain theory is more widely used, the resulting analyses are not as accurate as those that use the finite-strain theory. Based on the traditional small-strain theory, Raghu et al. (1987) reported a compression index (C_c) of approximately 0.3 for their lime/alum/polyamine coagulant sludge. The compression index (C_c) is computed from the slope of the field compression curve, which relates the void ratio (e) with the logarithm-of-consolidation pressure (p), namely, e versus log p curve. The void ratio is the ratio between the volume of void and the volume of solid phase of the sludge.

The e versus log p curves of Alum Sludges 1 and 2 and Ferric Sludge 3 are shown in Figure 7-3 (Wang et al., 1992b). From these curves, C_c equals 6.69, 5.29, and 1.99 for Alum Sludge 1, Alum Sludge 2, and Ferric Sludge 3, respectively. Under the same sustained pressure, the higher the C_c value is, the greater the landfill settlement will be. Thus, of these four sludges, it can be expected that the landfill of Alum Sludge 1 will settle most, while the lime/alum/polyamine sludge will settle least, other factors being equal.

From Figure 7-3, the swelling index (C_S) can also be obtained. The value of C_S is computed from the slope of the swelling (or rebound) portion of the graph and is normally used to determine the amount of landfill rebound caused by reduction of the sustained weight. The values of C_S are 0.17, 0.15, and 0.03 for Alum Sludge 1, Alum Sludge 2, and Ferric Sludge 3, respectively. Thus, of these three sludges, the amount of landfill rebound will be greatest for Alum Sludge 1 and smallest for Ferric Sludge 3.

The effect of a bulking agent on the e versus log p relation of Alum Sludge 1 is illustrated in Figure 7-4 (Wang et al., 1992a). As shown, admixing of a bulking agent results in a lower e versus log p curve. To consider the effect of pozzolanic reaction in the lime and fly ash treated sludge, these two treated sludges were curved to two different durations: 2 weeks and 4 weeks. The compression indexes of these treated sludges are 4.04 for the soil-treated, 3.56 for the lime-treated and 2-week cured, 4.34 for the lime-treated and 4-week cured, 3.70 for the fly ash-treated and 2-week cured, and 3.93 for the fly ash-treated and 4-week cured.

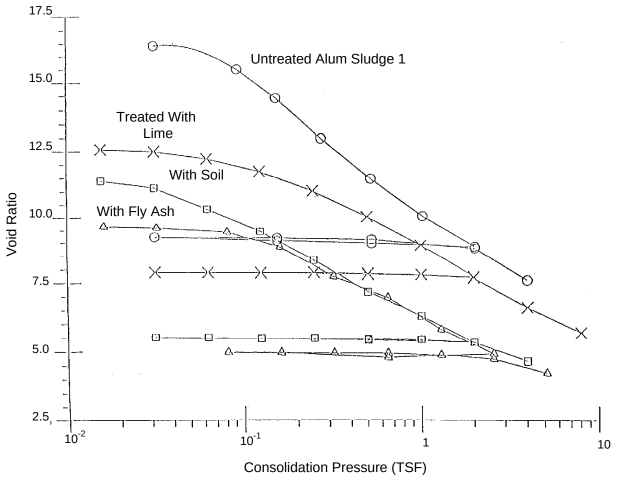


Figure 7-4. Void ratio versus consolidation pressure of treated and untreated Alum Sludge 1 (Wang et al., 1992a).

7.6.4 Shear Strength

Shear strength is essential for the design and analysis of landfill stability with or without heavy equipment operating on the landfill. Different testing methods are available for the determinations of shear strength, including cone penetration test, vane shear test, and triaxial compression test. The first two methods normally are used to measure undrained shear strength, while the third method is used to determine both undrained and drained strengths. The undrained shear strength is for use when there is insufficient time for the sludge to drain during shearing. Under the undrained loading condition, excess pore water pressure develops, reducing the capacity of sludge particles to resist shear and resulting in a lower shear strength. On the other hand, the drained shear strength is for when, during shearing, the sludge drains completely without developing excess pore water pressure. The sludge particles more effectively resist shear, resulting in a higher shear strength. The undrained shear strength is used for analysis of landfill stability during or immediately after construction; the drained shear strength is needed in the analysis of long-term stability of the landfill.

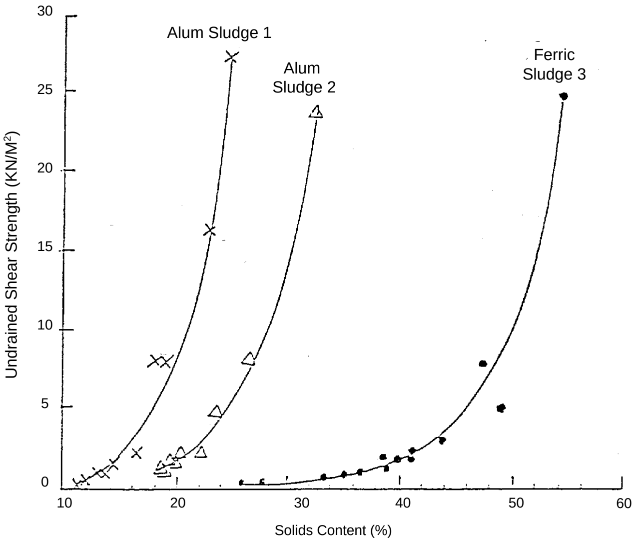


Figure 7-5. Strength versus solids content for Alum Sludges 1 and 2 (Cornwell et al., 1992).

The shear strength property of water treatment sludges varies not only with the type and nature of the sludge but with solids content as well. The rate and amount of strength increase with increasing solids content also differ for various sludge types. Figure 7-5 shows the undrained shear strength versus solids content curves for Alum Sludges 1 and 2 and Ferric Sludge 3. As shown, the shear strength increases very slowly in the beginning and should approach a constant value at very high solids content for each sludge, although the later portion of the curve is not seen within the range of conditions investigated. The solids content required for a given shear strength is highest for Alum Sludge 1 and lowest for Ferric Sludge 3. Also, at a given solids content, the shear strength is greatest for Alum Sludge 1 and least for Ferric Sludge 3.

The shear strength parameters and pore water pressure parameter at failure (A_f) obtained from the triaxial compression test are summarized in Table 7-14 (Wang et al., 1992b). Also included is the range of initial solids content of the test specimens. The initial solids content of each sludge is near the value required for ease in test specimen preparation. The A_f value of 0.75 to 0.79 falls within the range for normally consolidated clays or sensitive clays. The effective internal friction angle (ø), however, is unusually high when compared with highly plastic clay soils.

Water treatment sludges exhibit rheotropic and thixotropic behavior; namely, sludges undergo strength reduction from disturbance or remolding, and after disturbance the strength increases with curing time without a change in solids content. The reduction in sludge strength from disturbance has been experienced by workers who operated a hydraulic excavator on a trench fill of Chesapeake sludge (Alum Sludge 1). According to the machine operator, the sludge that initially was stable enough to operate the machine on became weak and unstable over time (Cornwell et al., 1992).

The rate and amount of strength increase from curing depends on sludge type and solids content. The increase of undrained shear strength at different solids contents for the ferric sludge is shown in Figure 7-6. From these strength data, the strength-gain ratios are computed for different sludges at different solids contents and are shown in Table 7-15. The strength-gain ratio is the ratio between cured strength and remolded strength. The data in Table 7-15 indicate that the strength-gain ratio appears first to increase, then to decrease with decreasing solids content. Thus, there is an optimal solids content for the strength-gain ratio to reach the maximum.

Table 7-14. Shear Strength Parameters of Test Sludges (Wang et al., 1992b)

		Total Stress		Effective Stress		
	Initial Content	c (kPa)	ø (°)	c′ (kPa)	ø′ (°)	Pore Pressure Parameter A_f
Alum Sludge 1	13.4–15.5	4.14	19.3	6.89	42.3	0.75
Alum Sludge 2	24.3–26.3	4.83	19.0	8.27	44.0	0.77
Ferric Sludge 3	36.2–40.4	8.27	17.5	8.27	42.8	0.79

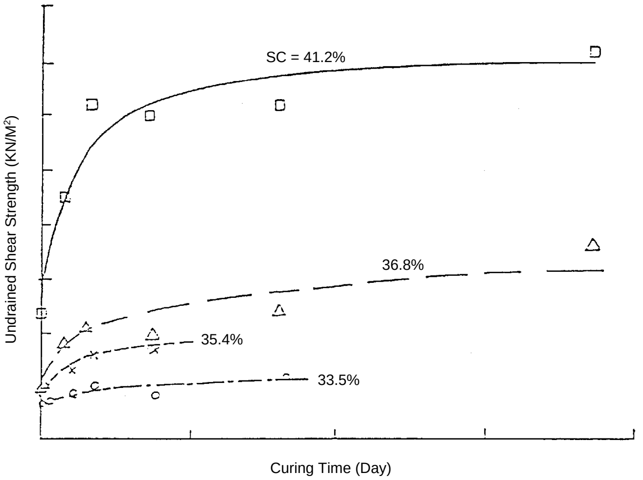


Figure 7-6. **Strength versus curing time at various solids contents for ferric sludge (Wang et al., 1992b).**

Table 7-15. Remolded and Cured Undrained Strengths and Strength Gain Ratio (Wang et al., 1992b)

	Solids Content (%)	Remolded Strength (kPa)	Cured Strength (kPa)	Strength Gain Ratio
Alum Sludge 1	16.3	2.21	12.96	5.9
	13.6	1.10	7.65	7.0
	11.9	0.48	3.86	8.0
	11.1	0.34	1.93	5.7
Alum Sludge 2	19.3	2.76	17.93	6.5
	18.6	2.07	115.17	7.3
Ferric Sludge 3	41.2	2.34	7.24	3.1
	36.8	1.03	3.45	3.4
	35.4	0.83	1.86	2.2
	33.5	0.62	1.24	2.0

The rheotropic and thixotropic behavior of sludge should be properly considered in the analysis, design, and construction of sludge landfills. Because construction operation inevitably causes sludge remolding, it is appropriate to use the remolded undrained shear strength to design the landfill and also to analyze the stability of the landfill during construction. After construction, the shear strength of sludge increases with time from thixotropic hardening and consolidation. Consequently, the landfill becomes more stable.

Admixing a bulking agent enhances the shear strength of the sludge. Figure 7-7 shows that for Alum Sludge 2 without curing, the degree of strength improvement varies with the type of bulking agent, being greatest for the slaked lime, fly ash, and finally the soil. Furthermore, for each bulking agent, strength increases with increasing treatment level (Cornwell et al., 1992). The strength increase from curing also differs for various bulking agents, as illustrated in Figure 7-8 (Cornwell et al., 1992). Although the ultimate cured strength is greatest for lime, then fly ash, soil, and the untreated sludge, the strength-gain ratio is in reverse order, as shown in Table 7-16.

The data in Table 7-16 indicate a drastic increase in the remolded shear strength of Alum Sludge due to treatment, by a factor of more than four times for lime, almost three times for fly ash, and about one-and-a-half times for soil. Such an improvement in shear strength makes it possible to construct a higher landfill with a steeper slope. Additionally, admixing stabilization makes the sludges less plastic and therefore easier to handle.

Using the shear strength data in landfill analysis and design has been demonstrated by Cornwell et al. (1992) and Wang et al. (1992b). Specifically, the strength data were used to make a hypothetical monofill design for the three different sludges with varying solids concentration. The maximum fill height for a desired slope angle or the maximum slope angle for a desired fill height were determined. The hypothetical monofill has a uniform slope with a constant slope angle from the toe to the top of the slope. The top of the landfill is level, extends very far, and carries no surface loading. Meanwhile, the landfill is supported on very large, firm, level ground, which allows it to be treated as a two-dimensional problem.

Figure 7-9 shows landfill height versus the ratio between shear strength and wet unit weight of sludge for different slope angles using a safety factor of 1.20. The minimum solids content required for maintaining a stable slope can be determined by:

1. Selecting the desired slope angle and landfill height.
2. Finding the required shear strength to wet unit weight ratio from Figure 7-9.
3. Assuming a wet unit weight of the sludge and computing the shear strength.
4. Finding the solids content for the computed shear strength from Figure 7-5.
5. Checking the wet unit weight for the obtained solids content using Equation 7-4:

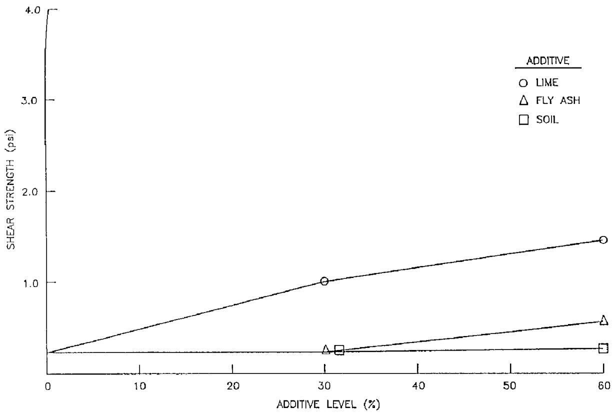


Figure 7-7. Shear strength versus additive level, Alum Sludge 2 (nonaged) (Cornwell et al., 1992).

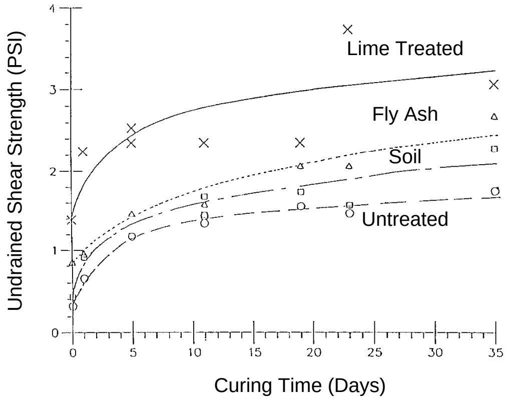


Figure 7-8. Undrained sheer strength versus curing time for untreated and treated Alum Sludge 1 (Cornwell et al., 1992).

Table 7-16. Undrained Shear Strength of Alum Sludge, Untreated and Treated (Wang et al., 1992a)

Sludge	Remolded (psi)	Cured (psi)	Strength Gain Ratio
Untreated	0.30	1.70	5.7
Treated with soil	0.42	2.10	5.0
Treated with fly ash	0.82	2.50	3.0
Treated with lime	1.39	3.25	2.3

$$\gamma = \frac{\gamma_W}{1 - (1 - \frac{1}{G_s})\frac{S_c(\%)}{100}} \qquad \text{(Eq. 7-4)}$$

where

γ = wet unit weight of the sludge

γ_W = unit weight of water

G_s = specific gravity of the solid phase of sludge

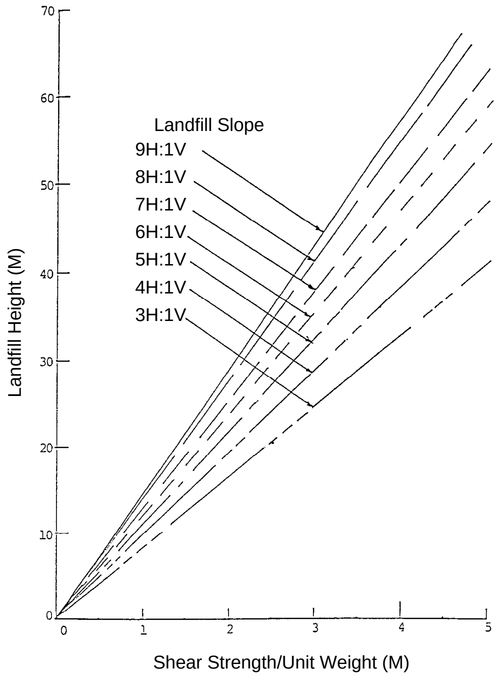


Figure 7-9. Landfill height versus shear strength/unit weight for different slope angles (Wang et al., 1992b).

S_c = solids content in percentage

6. Repeating the procedure from step 3 until satisfactory results are achieved.

Table 7-17. Required Shear Strength and Solids Concentration for Hypothetical Monofill Supporting Various Types of Heavy Equipment (Cornwell et al., 1992)

	Ground Pressure		Minimum Shear Strength		Approximate Sludge Concentration (%)		
Type	(psi)	(kN/m^2)	(psi)	(kN/m^2)	Alum Sludge 1	Alum Sludge 2	Ferric Sludge 3
Crawler dozer	4.2	28.7	2.0	13.7	29	22	52
Crawler dozer	6.7	45.8	3.3	22.6	33	25	54
Excavator	5.9	40.4	2.9	19.8	32	23	53
Dump truck (empty)	25	171.0	12.2	83.5	37	28	58
Dump truck (full)	50	342.0	24.4	166.9	41	33	62

The shear strength data were also used to analyze the stability of heavy construction equipment operation on the landfill. The types of equipment analyzed were crawler dozer, dump truck, and excavator. The shear strength and solids content required to support the various types of heavy equipment with a safety factor of 3.0 are summarized in Table 7-17. The minimum shear strength required to support heavy equipment varies greatly among the different types of equipment, and the data were obtained for a safety factor of 3.0. When a smaller safety factor is used, the required minimum shear strength is reduced. For example, for a safety factor of 1.5, the required minimum shear strength to support the excavator decreases from 19.8 to 9.8 kiloPascals. This value is very close to 10 kiloPascals of the European standard (Cornwell and Koppers, 1990). It is not known, however, whether the European standard was developed for the conditions stated. The minimum shear strength required to support heavy equipment should be selected with consideration of the factors that influence equipment stability: equipment type (weight, dimension, wheel design, and nature of loading), sludge layer thickness, sludge surface area and condition (level or sloped), and safety factor, among other factors.

According to Table 7-17, the minimum solids concentration required to support heavy equipment varies considerably among the three sludges. Thus, the required minimum solids concentration must be determined for every sludge, precluding a single value of minimum solids concentration for general use. The minimum solids concentration for Alum Sludge 1 in Table 7-17 reportedly correlates well with field conditions (Cornwell et al., 1992). Therefore, the data should be useful in determining minimum solids concentration and other information pertinent to landfilling of water treatment sludges.

Chapter 8
Land Application

Land application is an increasingly popular disposal option in the water supply industry due to the escalating regulatory and environmental constraints associated with other disposal options. Land application options for water treatment plant (WTP) residuals include agricultural use, silvicultural application, application for reclamation to disturbed and marginal lands, and disposal to dedicated lands (U.S. EPA, 1995b). In addition to the obvious advantages of waste disposal, this application process may beneficially modify soil properties while recycling residual components. Potential disadvantages of land application of WTP residuals include an increase in the concentration of metals in the soil and possibly in ground water; adsorption of soil phosphorus by water residuals, decreasing the productivity of the soil; excessive application of nitrogen, resulting in the transport of nitrate to ground water; and possible effects caused by the application of poorly crystallized solids of aluminum (Dempsey et al., 1990).

This chapter provides decision-makers with the basic information they need to evaluate land application as a disposal option for WTP residuals. A simplified planning procedure for land application is presented in Figure 8-1.

8.1 Regulatory Requirements

The Water Pollution Control Act of 1972 defined WTP residuals as industrial waste, and they are listed as a solid waste under 40 CFR Part 257. If land application is to be considered a viable option, the residuals cannot be hazardous. Section 2.7 outlines a method for determining whether the residuals are hazardous. Other than those listed in 40 CFR Part 257, specific federal regulations do not exist for the water industry, and most state regulatory agencies use the same criteria for WTP wastes as they use for other industrial sludges. According to 40 CFR Part 257, criteria in the following areas must be met: floodplains, endangered species, surface water, ground water, disease vectors, air emission, and safety. The regulations also contain application limits for cadmium (Cd), and polychlorinated biphenyls (PCBs).

The Clean Water Act (40 CFR Part 503), establishes criteria for land application of sewage sludge solids generated during the treatment of domestic sewage. Although these criteria are only designed for sewage sludge, or biosolids, they may provide general guidelines for use or disposal of WTP residuals. The guidelines contain numerical values for biosolids application rates that are based on experiments with biosolids, and risk assessment algorithms developed using the biosolids data. The criteria should not be used for determining land application rates of WTP residuals.

The risk assessment algorithms that were used as a technical basis for the Part 503 rule contain data that were obtained from field studies using biosolids. There is no way to determine, without similar field studies done specifically on WTP residuals, whether residuals would demonstrate the same behavior as that of biosolids. Therefore, land application of WTP residuals should be based on the criteria set forth in 40 CFR Part 257, and not on the criteria developed under Part 503. If WTP residuals are field tested and their behavior relative to biosolids is known, then it is possible that the risk assessment methodology used in Part 503 could be used to develop criteria for WTP residual use in land application.

Regulations vary from state to state but typically require that residuals be tested for total metals concentrations, nutrients, and pH. The application rate of WTP sludge may be selected to limit hydraulic loading, nitrogen content, or metal content.

8.2 Environmental Considerations

When land application was originally investigated as a potential disposal option for WTP residuals, it was thought that the lack of organic components and the high metals content in WTP residuals would inhibit the water industry from accepting this option. Since then, a number of studies have examined the potential limiting effects of WTP residuals on soil parameters. Most studies to date have focused on identifying the effects of WTP residuals on the physical characteristics of soil and on determining the potential for these wastes to cause phosphorus deficiency in plants. Current research has yielded a better understanding of the chemical characteristics of WTP residuals and has produced analytical techniques to quantitatively predict these effects when

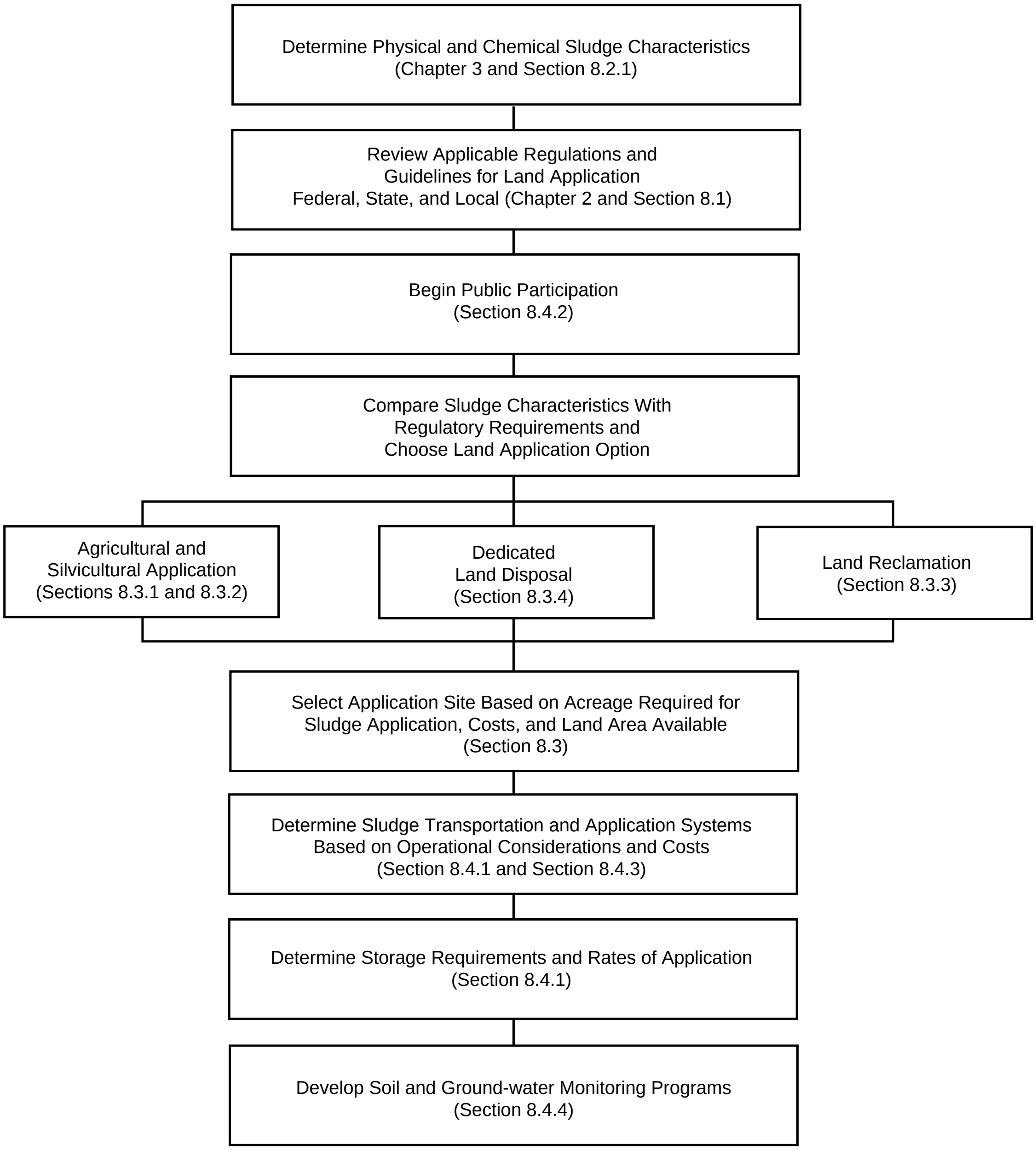


Figure 8-1. Simplified planning procedure for land application of WTP residuals.

applied to land. The conclusions of this research are described below.

8.2.1 Major Components of Water Treatment Residuals and Their Impact on Soil Parameters

A 1990 American Water Works Association Research Foundation study of land application of WTP residuals was conducted to establish a scientific basis for land application of WTP residuals. This study examined data regarding the chemical composition and fractionation of WTP residuals, the phytotoxicity of WTP residuals, and the effects of these residuals on the physical and nutrient status of soil. The study also investigated concentrations of residuals' major components, and the impact these components might have on agricultural soil after land application. Table 8-1 shows the composition of WTP residuals compared with sewage sludge and agronomic soils in the United States. The cadmium reading of 5.15 parts per million (ppm) is much higher than is typically found in WTP residuals. This is particularly true now that chemical suppliers are more conscious of heavy metals contaminants in coagulants, the major source of metals in residuals. Most cadmium levels in residuals are less than 1 to 2 ppm and are often undetectable.

WTP residuals are predominantly inorganic and consist of clays, humic substances, and other materials typically suspended or dissolved in lakes and streams. Typical WTP residuals contain 3 percent or less organic carbon by weight and 0.5 percent or less organic nitrogen by

Table 8-1. Composition of WTP Residuals Compared With Sewage Sludge and Agronomic Soils[a]

Constituent	WTP Residuals[b]	Sewage Sludge[c]	Agronomic Soil
Aluminum	60,111.0 (52,095)		72,000[d]
Arsenic	14.7 (14.7)	9.93 (18.84)	7.2[d]
Barium	122.5 (127.52)		580[d]
Boron	40.7 (61.3)		33[d]
Cadmium	5.15 (11.67)	6.94 (11.76)	0.265 (0.253)[e]
Calcium	20,815.0 (33,108)		24,000[d]
Chromium	49.5 (56.5)	118.6 (339.2)	54[d]
Copper	270.2 (326.1)	741.2 (961.8)	29.6 (40.6)[e]
Iron	52,751.0 (63,642)		26,000[d]
Lead	79.9 (100.2)	134.4 (197.8)	12.3 (7.5)[e]
Manganese	385.4 (398.5)		9,000[d]
Mercury	1.5 (2.5)	5.2 (15.5)	0.09[d]
Molybdenum	9.1 (12.9)	9.2 (16.6)	0.97[d]
Nickel	37.8 (53.8)	42.7 (94.8)	23.9 (28.1)[e]
Selenium	2.8 (1.5)	5.2 (7.3)	0.39[d]
Strontium	84.3 (60.6)		240[d]
Vanadium	35.4 (17.4)		80[d]
Zinc	1,047.4 (3,036.1)	1,201.9 (1,554.4)	56.5 (37.2)[e]

[a] Data are arithmetic mean and standard deviation ( ) presented as mg/kg dry weight.
[b] Survey of 12 land-applied WTP residuals, including 10 alum and 2 ferric (Brobst, 1994).
[c] U.S. EPA, 1990a.
[d] Shacklette and Boerngen, 1984.
[e] Holmgren et al., 1993.

weight. Both values are representative of agricultural soils but are much lower than those found in sewage sludge. WTP residuals are similar to fine-textured soil. Elliott et al. (1990a) reported no coliform organisms in alum or ferric WTP residuals. This absence may be attributed to clean raw water, disinfection processes during treatment, and/or lagoon storage of residuals over long periods. In a waste stream recycling study (Cornwell and Lee, 1994), however, *Giardia* and *Cryptosporidium* were found to concentrate in the sedimentation basin residuals. The study did not examine the issues of the viability and persistence of these pathogens, nor did it discuss the survival of pathogens in WTP residuals over time.

Table 8-2. Agronomic Components in WTP Residuals[a]

Parameter	Range of Values[b]
Total solids (%)	8.14–81
Volatile solids (%)	9.32–29.09
Conductivity (µmho/cm)	563.8 (530.2)
pH (S.U.)	6.98–8.82
Total Kjeldahl nitrogen (%)	0.495 (0.256)
Organic nitrogen (%)	0.752 (0.399)
Ammonia–nitrogen (%)	0.016 (0.016)
Nitrate–nitrite (%)	0.003 (0.003)
Total phosphorus (%)	0.226 (0.248)
Total potassium (%)	0.225 (0.317)

[a] Data are presented as a range or as arithmetic mean and standard deviation ( ). These data should be used with caution. Individual residuals may vary significantly from those used in this table.
[b] Survey of 12 land-applied WTP residuals, including 10 alum and 2 ferric (Brobst, 1994).

WTP residuals largely consist of materials that are removed from water before its distribution. Coagulant addition can enhance turbidity removal and contribute to the quantity of residuals. Depending on the quality of the intake water, the final residuals can be clay-like or sand-like in texture, which may become important in choosing the final use. WTP residuals by their nature are usually low value fertilizer, as can be seen in Table 8-2. The concentration of aluminum or iron in WTP residuals is generally 5 to 15 percent of dry solids. These concentrations stem from the use of coagulants in the water treatment process and may be higher when a specific coagulant is overdosed. The aluminum content of WTP residuals treated with alum is similar to aluminum concentrations in soils. The speciation of aluminum or iron in WTP residuals, however, is different from that occurring in native soils. While soils contain significant concentrations of aluminum in the form of aluminosilicates, WTP residuals contain aluminum (or iron) in amorphous hydrous oxide forms, which exhibit greater reactivity than the corresponding soils materials. The various aluminum forms, however, quickly convert to the stable aluminum oxides away from aluminum hydroxides, and do not exhibit reactivity.

Moderate applications of WTP residuals may improve the physical condition, or tilth, of soils by flocculating colloidal particles, thereby promoting soil aggregation via reactions that are analogous to those occurring during water treatment. The hydrous metal oxides contained in WTP residuals are strong adsorbents of trace metals and phosphorus. In terms of trace metals, this is beneficial to soils and results in a decreased and buffered concentration of free trace metals. Elliott et al. (1990b) concluded that in the case of soils treated with WTP residuals, soluble aluminum usually decreases, and the level of the dithionite-citrate-bicarbonate (DCB)[1] extractable aluminum residuals is typically elevated over original levels but remains within the range normally observed for native soils (see Table 8-1). The amorphous hydrous oxides in alum-treated WTP residuals may also benefit coarse soils by increasing the cation exchange capacity, particularly at the neutral or slightly

---

[1] DCB is an extracting solution that removes the more reactive fraction of the total iron and aluminum present in a substance.

alkaline soil pH that can increase the nutrient holding capacity.

8.2.2 Effects of Trace Metals Concentrations on Soil Properties

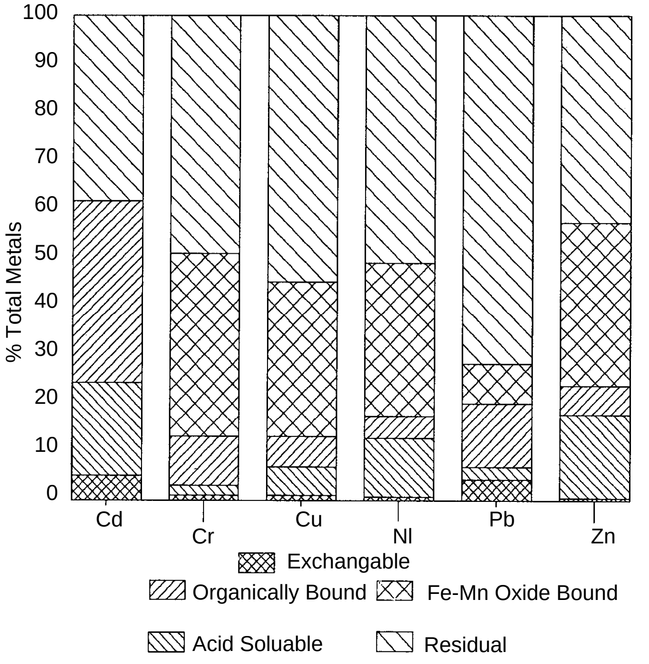
Trace metals in most WTP residuals originate from coagulant chemicals and or the water source. These metals are strongly adsorbed to the aluminum hydroxide or ferric hydroxide precipitates formed during the coagulation process. Concentrations of trace metals will likely determine the lifetime application of water treatment residuals (metric tons per hectare, dry weight) on agricultural soils.

Elliott et al. (1990a) assessed potential metal mobility under field conditions using a five-step fractionation procedure. Most heavy metals in WTP residuals are bound in forms not readily released into solution due to their strong adsorption and coprecipitation by aluminum (Al) and iron (Fe) hydroxides freshly formed in the coagulation treatment step. From 76 to 87 percent of the chromium (Cr), copper (Cu), nickel (Ni), lead (Pb), and zinc (Zn) examined were bound within an oxide or silicate matrix. Less than 6 percent of the total concentration of each of these metals was in the exchangeable fraction, which is considered to represent the easily available metal pool. If soil conditions should become acidic (pH less than 5), approximately 25 percent of the residuals cadmium could become mobile.

Greenhouse pot studies showed that with the more acidic aluminum-based WTP residuals, application rates of 20 and 25 g/kg increased the cadmium concentration of plant tissue to greater than 2 mg/kg (Heil and Barbarick, 1989). Figure 8-2 illustrates the partitioning of trace metals in WTP residuals. For example, most chromium is released upon dissolution of $Al(OH)_{3(s)}$ and $Fe(OH)_{3(s)}$ (i.e., the iron-manganese oxide-bound fraction), whereas a very small portion of chromium is released in the acid-soluble fraction. Similar results can be obtained for other trace metals. Soil pH conditions are critical in preventing the dissolution of $Al(OH)_{3(s)}$ or other hydrous metal oxides; if nonacidic soil conditions exist, trace metals will be tightly bound, rendering them immobile and unavailable for plant uptake. Therefore, in the case of moderate WTP residuals application rates (20 dry mt/ha) and properly managed soils (pH greater than 6.0), movement of metals into ground water or into plant tissues can be minimized.

8.2.3 Impact of Water Treatment Residuals on the Availability of Phosphorus in Agricultural Soils

Because phosphate deficiency greatly affects agricultural soil fertility, much research has been conducted into the decreased availability of phosphorus in soils following the addition of WTP residuals. This decrease in available phosphorus occurs because of the large amounts of aluminum and iron hydroxide solids present in WTP residuals. These hydroxides are strong adsorbents of inorganic phosphorus and contain only low concentrations of phosphorus. This phosphorus-limiting characteristic of WTP residuals is illustrated in Figure 8-3.



Definitions:

1. Exchangeable: 16 mL 1M $MgCl_2$ (pH 7.0); shake time, 1 hr.
2. Dilute acid extractable: 16 mL 1M NaOAc adjusted to pH 5.0 with HOAc; shake time, 5 hrs.
3. Fe-Mn oxide bound: 40 mL of 0.175M $(NH_4)_2C_2O_4$ and 0.1M $H_2C_2O_4$; shake time, 4 hrs.
4. Organically bound: 40 mL of 0.1M $Na_4P_2O_7$; shake time, 24 hrs.
5. Residuals: Dry 0.1 g of material remaining after 4/. Add 4 mL HNO_3, 1 mL $HClO_4$ and 6 mL of HF, heat at 140°C for 3.5 hrs. Add 5.0 g Boric acid and dilute to 100 mL volume.

Figure 8-2. Partitioning of trace metals in WTP residuals (after Elliott et al., 1990a).

Greenhouse studies found phosphorus deficiencies in sorghum-sudangrass were possible when application rates of aluminum- or iron-based WTP residuals exceeded 10 g/kg soil or approximately 22.5 mt/ha (Heil, 1988; Heil and Barbarick, 1989). The study also found that both alum- and iron-based WTP residuals decrease the availability of phosphorus but can effectively increase iron availability in iron-deficient soil. WTP residuals applied at a rate of 60 g/kg or approximately 135 mt/ha, produced phosphorus deficiencies in tomato plants (Elliott and Singer, 1988).

This reduction in plant-available phosphorus is qualitatively different from the potential toxicity problem that may be associated with land application of WTP residuals. The depletion of plant-available phosphorus necessitates the addition of supplemental phosphorus

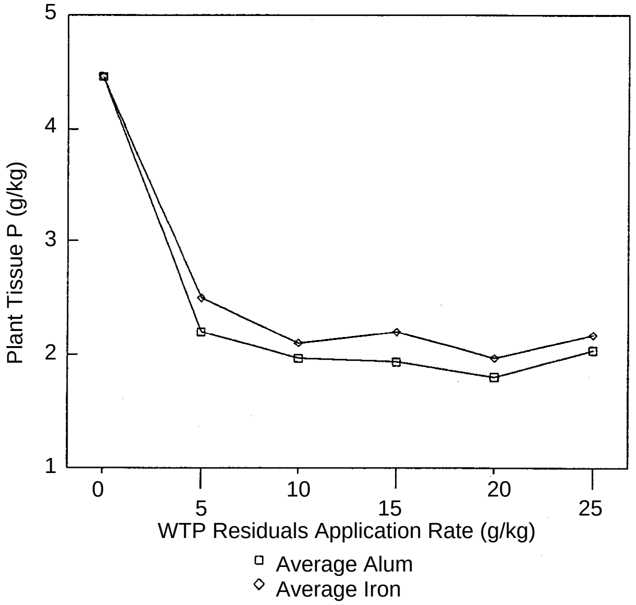


Figure 8-3. **Average (of three cuttings) phosphorus concentration in sorghum-sudangrass grown in Colby soil (after Heil and Barbarick, 1989).**

fertilizer to obtain optimal plant yields. This is a management problem rather than a public health issue; therefore, the problem is easy to correct with either adjusted management practices or additional phosphorus fertilizer.

Soluble phosphorus levels are commonly determined by the equilibrium phosphorus concentration (EPC) test (White, 1981). The EPC is the concentration of phosphorus at which no adsorption or desorption of phosphorus occurs. The phosphate buffer capacity (PBC) provides information about the response of a soil to fertilization, specifically whether the added phosphorus will be adsorbed by the soil or will remain in the soil solution and be available for plant uptake. Elliott et al. (1990b) used both EPC and PBC techniques to investigate the incremental amount of phosphorus fertilizer that should be applied to soils treated with WTP residuals to obtain good plant growth.

The typical range of EPC values for optimal production is 50 to 200 μg/L EPC. Between 50 and 100 of the triple superphosphate (TSP) fertilizer used in this study is conventionally added to agricultural soils to maintain crop production (White, 1981). Farmers usually apply nitrogen and potassium fertilizers to agricultural soils in addition to various pesticides. Therefore, the cost of restoring the EPC after residuals application should be a fraction of the entire cost of preparing a soil for planting.

Application techniques may be used to mitigate the impact of WTP residuals on the availability of phosphorus in agricultural soils. WTP residuals can be applied to the top soil layers or injected below the surface, depending on the growth stage of specific crops. Seedlings generally have higher phosphorus requirements than mature plants do; therefore, deep injection (greater than 18 cm) may be preferable at this growth stage. When residuals are applied to the soil surface, the resulting drying action decreases the surface area of the aluminum and iron hydroxides, and organic matter in the surface litter will localize the adsorption sites for phosphorus. This may in fact be beneficial in areas of high naturally occurring phosphates, and WTP residuals may be able to reduce phosphorus loadings to surface waters by adsorbing the phosphorus. In addition, in areas in which stormwater runoff carries excess phosphorus to lakes and streams, WTP residuals could be used to bind excess phosphorus and improve the runoff water quality.

Crops may be selected that tend to use labile phosphorus rather than just soluble phosphorus. Large quantities of phosphorus fertilizer are applied in close proximity to seeds to accommodate the phosphorus requirements of seedlings. Sewage sludge contains significant amounts of phosphorus that can add to total soil phosphorus. Several Colorado facilities have studied the coapplication of both sewage sludge and WTP residuals to mitigate the effects of phosphorus adsorption by WTP residuals. The experiments have met with varying degrees of success in the short evaluation periods, however, they have shown that the assumption of phosphorus being bound on a one-to-one molar basis, as required by state regulations, may be conservative.

In the case of excessively fertilized soils, phosphorus adsorption by WTP residuals may be agriculturally advantageous. WTP residuals applied to high phosphorus soils can produce a lower EPC for optimal crop growth.

8.2.4 Effects of WTP Residuals on Soil Physical Properties

Research has been conducted to determine whether application of WTP residuals conditions agricultural soils by increasing the soil organic matter content. Soil conditioning is treatment that modifies a soil's physical properties for the improvement of crop growth (Chang et al., 1983). The organic matter content of dry WTP residuals varies depending on the original raw water source and the treatment methods employed at individual WTPs. Estimates of percent organic matter in WTP residuals range from 14.4 percent (Lin, 1988) to 25 to 35 percent (Bugbee and Frink, 1985). These values seem high and should be used with caution. Appropriate methods for determining organic matter in WTP residuals should be used. Current research in this field (Elliott et al., 1990a) determined that the average organic matter content of seven test residuals was 3 percent, with an average loss on ignition (LOI) value of 33 percent. Thus, the soil conditioning effects of organic matter in WTP residuals are probably quite small and may not be measurable under field conditions.

At a land application site in Colorado with cumulative application rates of up to 179 mt/ha (80 tons/acre) (by 1984) researchers theorized that the cumulative loadings of quartz and aluminum hydroxides exhibited some potential to change soil texture in fields near an industrial water treatment facility (King et al., 1988). The annual application rate was approximately 45 mt/ha (20 tons/acre). Soils on the fields amended with alum residuals have shown a potential for dusting and more rapid drying than the nonamended fields. Application rates were lowered to control this situation.

8.3 Land Application Options

8.3.1 Agricultural Land

Agricultural use of WTP residuals is the beneficial application or disposal of residuals on agronomic lands. Successful implementation of this disposal option requires an understanding of the effects of WTP residuals on soil fertility and physical properties (discussed in the previous section). The physical characteristics of soil that determine whether it can support vegetative growth include cohesion, aggregation, strength, and texture. These parameters directly affect the hydraulic properties of a soil, such as moisture-holding capacity, infiltration, permeability, and drainage. Any adverse impact on these hydraulic soil characteristics from land-applied WTP residuals can effect crop growth and ultimately degrade ground water quality (Elliott et al., 1990b).

Concentrations of the WTP residuals listed in Table 8-1 tend to be elevated compared with the concentration of trace metals described in Chapter 3. Many of the sampled WTPs in Colorado treat surface waters affected by past mining activities. The length of time a site may be used depends on the quality of the WTP residuals applied. Two basic types of residuals are generated at a WTP—lime softening residuals and coagulant (alum or ferric) residuals. These residuals are discussed below.

8.3.1.1 Lime Softening Residuals

Addition of lime to agricultural soil is a common practice in areas where the soil pH is too low for optimal plant growth; lime modifies the balance between acidity and alkalinity in the soil. Soil pH should be maintained at 6.5 or above to minimize crop uptake of metals (U.S. EPA, 1983). In addition to providing a desirable pH for plant growth, lime residuals can be substituted for agricultural limestone and offer calcium carbonate equivalence (CCE) or neutralization effectiveness. Lime residuals can be dewatered to higher solids content than alum residuals can; therefore, the potential for unique disposal or co-disposal opportunities is greater than for other residuals. Studies show the neutralizing power of lime residuals is equal or superior to that of agricultural limestone (Che et al., 1988). Lime residuals increase the porosity of tight soils, rendering the soils more workable for agricultural purposes.

The best way to estimate the liming requirement of a soil is to titrate individual samples of the soil with a standard base. The result is the quantity of base required to raise the pH of the soil to a specified pH. The theoretical chemical process is illustrated below:

$$\mathrm{CaCO_3 + H_2O = Ca^{2+} + HCO_3^{\,-} + OH^-}$$

Ten mmoles of a base per kilogram consumed during titration is equivalent to 4.5 metric tons of pure $CaCO_3$ per hectare at a 30-cm plough layer (ha-30 cm) of the sample soil.

Field liming results may differ considerably from these experimental results, depending on the mixing of lime residuals with the soil, the solubility of the liming agent, and the soil sampling procedures. Use of a conversion factor generally corrects this difference. The U.S. Department of Agriculture’s Extension Service may be consulted for the appropriate conversion factor for a given area.

Lime residuals can be applied as a liquid or dewatered and applied as a cake using an agricultural manure spreader. The application rate for lime residuals must be calculated on a dry weight basis, then converted to the equivalent wet weight for application to the farm field.

8.3.1.2 Alum Residuals

Alum residuals when dry generally have the consistency of very fine soils. Bugbee and Frink (1985) indicate that alum residuals improve the physical characteristics of soil media but inhibit plant growth by adsorbing phosphorus. As stated earlier, this reduction in phosphorus available to plants is different from the toxicity problem that may be associated with land application of WTP residuals. To obtain optimal plant yields when land applying alum residuals, phosphorus fertilizer must be added.

As previously mentioned, soil conditioning effects from organic matter in WTP residuals are probably quite small and may not be measurable under field conditions. Management practices such as lowering application rates can control the dusting and rapid drying seen in fields amended with alum residuals. Application of alum residuals at rates that are not considered excessive (20 mt/ha) does not cause environmental degradation. Alum residuals are applied as a liquid unless the WTP has dewatering capabilities. Liquid alum residuals can be applied with a liquid manure spreader, like lime residuals, or with conventional irrigation equipment.

Example 1: Co-disposal With Sewage Sludge

Colorado in the late 1980s developed a WTP residuals land application policy based on research completed in northeastern Colorado. This approach assumed that aluminum and iron would bind with the phosphorus in the WTP residuals on a one-to-one basis. Because of this potential phosphorus binding effect, Colorado developed a conservative method of estimating the fraction of WTP residuals that can be coapplied with sewage sludge. That method is described in Equation 8-1.

$$\text{WTP residual fraction} = \frac{\text{P(SS)} - 1.15\ \text{Al(SS)} - 0.55\ \text{Fe(SR)}}{[1.15\ \text{Al(WTR)} - 1.15\ \text{Al(SR)}] + [0.55\ \text{Fe(WTR)} - 0.55\ \text{Fe(SR)}]} \qquad \text{(Eq. 8-1)}$$

where

P(SS) = phosphorus content (mg/kg) in sewage sludge
Al(SS) = aluminum content (mg/kg) in sewage sludge
Fe(SS) = iron content (mg/kg) in sewage sludge
Al(WTR) = aluminum content (mg/kg) in WTP residuals
Fe(WTR) = iron content (mg/kg) in WTP residuals

For example, if

P(SS) = 25,000 mg/kg (average value from U.S. EPA, 1995)
Al(SS) = 8,000 mg/kg (typical value)
Fe(SS) = 15,000 mg/kg (typical value)
Al(WTR) = 60,111 mg/kg (average value from Table 8-1)
Fe(WTR) = 52,751 mg/kg (average value from Table 8-1)

then, the WTP residuals fraction =

$$\frac{25{,}000\ {}^{mg}\!/_{kg}\ \text{P} - 1.15\ (8{,}000\ {}^{mg}\!/_{kg}\ \text{Al}) - 0.55\ (15{,}000\ {}^{mg}\!/_{kg}\ \text{Fe})}{[1.15\ (60{,}111\ {}^{mg}\!/_{kg}\ \text{Al}) - 1.15\ (8{,}000\ {}^{mg}\!/_{kg}\ \text{Al})] + [0.55\ (52{,}751\ {}^{mg}\!/_{kg}\ \text{Fe}) - 0.55\ (15{,}000\ {}^{mg}\!/_{kg}\ \text{Fe})]}$$

$$= 0.193$$

The values used in this example are only for demonstration purposes. Because each WTP residual and sewage sludge has different qualities, the most current data from a given treatment plant should be used. It should be noted that Colorado is no longer using this method and has been evaluating land application requests on a case-by-case basis.

The result of this equation and agronomic information for the sewage sludge can be used to determine a land application rate for residuals. If the nitrogen in a WTP residual is higher than expected (higher than in Table 8-2), the nitrogen content must also be used in calculating the agronomic rate. The agronomic rate of sewage sludge is the amount needed to supply the recommended amount of nitrogen (N) for a particular crop without allowing excess nitrogen to migrate below the root zone and into the ground water. According to 40 CFR Part 503.14.d, "Bulk sewage sludge shall not be applied at rates above agronomic rates, with the exception of reclamation projects and when authorized by the permitting authority."

When calculating the agronomic rate, note that soil data are required to determine the amount of nitrogen needed for a specific crop. Therefore, for dry land farming, deep soil monitoring should be conducted once every 5 years, with samples collected at 1-foot intervals for a total of 5 feet. For irrigated farm land, samples should be collected within the plow zone on an annual basis. If soil data are not available, residual nitrogen from previous sewage sludge applications can be determined using the mineralization rate of organic nitrogen. Annual soil tests, however, provide a more accurate estimate of the nitrogen content of the soil. Because an individual soil sample may not provide an accurate determination for an entire land area, multiple soil samples should be taken from different locations within the field.

Example 2: Land Application With Supplemental Phosphorus

Fertilizer recommendations are specific to crop, climate, soil type, and yield goals. Therefore, this option is discussed only generally.

Generally, the phosphorus (P) soil test rates soils as low, medium, high, and very high with respect to available phosphorus. Thus, the soil test is interpretive rather than purely analytical. For soils with low available phosphorus, as indicated by the test, good agricultural practice is to add significantly more phosphorus than the crop needs. This extra phosphorus is added to compensate for the phosphorus-depleting effects of land-applied alum/iron residuals in addition to the intrinsically low phosphorus availability. For soils rated as medium, slightly more phosphorus should be added than the crop demands. Soils rated as high in phosphorus need none added; however, enough phosphorus fertilizer should be added to compensate for any phosphorus depletion occurring due to the presence of alum/iron residuals in the soil.

For example, assume that the phosphorus demand for WTP residuals applied at 1 mt/ha is equivalent to 98.13 kg/ha (196.26 lb/acre). Also, the soil test indicated that soil phosphorus demand is 15 mg/kg. Using Table 8-3's figures for a wheat crop:

- Phosphorus demand from the alum/iron residuals = 98.1 kg/ha.
- Phosphorus required to satisfy the crop need (from the table) = 19.5 kg/ha.

Therefore,

- Estimated soil demand for phosphorus = 5 kg/ha.

The total supplemental phosphorus required for this example is 132.6 kg/ha (265 lb/acre), with the alum/iron

Table 8-3. Phosphorus Recommendations for Several Agronomic Crops[a] (Foth and Ellis, 1988)

Soil P Test mg/kg	Phosphorus Applied kg/ha (lbs/acre) Corn	Wheat	Soybeans	Potatoes
15	15 (30)	19.5 (39)	11 (22)	38 (76)
30	6.5 (13)	11 (22)	0	35 (70)
50	0	0	0	30.5 (61)
80	0	0	0	24 (48)
175	0	0	0	0

[a] Yield goals assumed: corn, 140 bushels/acre; wheat, 70 bushels/acre; soybeans, 50 bushels/acre; potatoes, 400 cwt/acre.

residuals applied at 1 mt/ha. Again, the assumptions made here are for illustration purposes only; the WTP must modify this information to reflect local conditions.

8.3.2 Silviculture

Silviculture, another agricultural practice, is the establishment, growth, reproduction, and care of forest trees as a crop. Land application of both wastewater and WTP residuals is relatively uncommon in silviculture compared with application to agricultural lands. Bugbee and Frink (1985) indicated the following effects of residuals application to silvicultural use: no significant effects on tree growth, nutrient levels, or the appearance of the forest floor; slightly diminished uptake of phosphorus; and increased soil pH for trees treated with WTP residuals. Grabarek and Krug (1987) concluded that limited application of WTP residuals to a healthy forested area does not upset the phosphate cycle to the point of affecting forest growth patterns.

The effects of long-term (30 months) application of alum residuals were evaluated by Geertsema et al. (1994). Alum residuals were applied to experimental plots with two to four replicates of each application rate: 0, 36, and 52 mt/ha (0, 16, and 23 dry tons per acre). The study found no statistically significant differences in soil characteristics, ground-water characteristics, or loblolly pine growth analyses between the unamended and the residuals-amended plots. Geertsema et al. also found no significant metal migration through the soil profile. At the application rate used in the study, no problems were observed in available phosphorus as evidenced by the similar pine tree growth in the treated and untreated plots. The results of the study did not indicate any environmental problems; therefore, the City of Newport News has implemented a full-scale project.

8.3.3 Land Reclamation

Sewage sludge is more commonly used than WTP residuals to reclaim surface-mined areas or other disturbed lands and to establish vegetative growth and/or restore or enhance soil productivity. Disturbed lands include highway construction sites, overgrazed rangelands, and other construction sites. In combination with other fertilizers, however, WTP residuals may benefit reclamation efforts. WTP residuals can be used to treat a particular or site-specific concern. For example, lime residuals can control soil pH just as they do in agriculture, but in these cases the pH adjustment may be more critical because mine soils can be very low in pH. In addition, alum residuals can control runoff of excess phosphate into surface waters. Care must be taken to ensure that the site is suitable for use of WTP residuals and that controls (best management practices) are in place to protect public health and the environment.

Application rates may vary from 5 mt/ha on arid rangeland to 450 mt/ha or higher on a mineland reclamation site. On a mineland reclamation site, WTP residuals may actually be used as a topsoil replacement; thus, the application rate may be as high as 450 mt/ha.

8.3.4 Dedicated Land Disposal

Dedicated land disposal (DLD) of WTP residuals is generally the alternative chosen by those operations that either generate residuals continuously or have quantity or quality concerns. DLDs are designed to treat and dispose of large quantities of residuals through soil microorganisms, sunlight, and/or oxidation. In addition, the soils are used to bind or fix the metals, thus making them unavailable.

A distinct advantage to using a DLD is that, for the most part, the WTP controls the application times, rather than other factors (e.g., crop growth cycles). DLD application rates are generally much higher than those of other land application operations, ranging from 20 mt/ha to greater than 200 mt/ha. This increased application rate can greatly decrease the amount of land required for the WTP.

The tradeoff between DLDs and other land application options is that stricter requirements generally apply to the DLDs. Increased application rates require that the site be carefully designed, managed, and monitored. The site must be designed to contain any of the residual constituents that might threaten public health or the environment. The site must take more care with surface runoff controls and the generation of contaminated leachate or infiltration. Generally, surface water controls include physical structures such as dikes, ditches, and lagoons. Control of leachate that is generated may require collection and treatment prior to discharge or disposal. Discharge of the treated leachate may require a National Pollutant Discharge Elimination System (NPDES) permit and/or additional state permits.

8.3.5 Other Use Options for WTP Residuals

Other applications for WTP residuals include use at turf farms, parks and recreational areas, highway rights of way, airports, and construction sites. Most of these projects utilize WTP residuals as a soil replacement or soil amendment. For example, in turf farming, each time a crop is harvested, a significant amount of soil is removed. WTP residuals can be used as a partial replacement. The other options also typically require imported soil, and in areas where topsoil is scarce residuals may be used as a partial or total replacement. As with all land application projects, however, the quality of the residuals must be matched with the needs of the user.

8.4 Operational Considerations in Land Application

8.4.1 Application Procedures

Once land application is selected as the preferred disposal option, the next step is to evaluate the land application site, the costs associated with that site, and the potential social and environmental impacts on the site. In addition, the costs of dewatering and hauling must be reviewed because the higher the percentage of solids, the less volume of residual must be hauled.

The land area requirement depends on the application rate. In general, application rates are determined using agricultural methods, as with fertilizer. In the case of WTP residuals, this fertilizer recommendation is at least two fold; the nutrient requirements for the crop are based on the expected yield and the available nutrients in the soil. The first nutrient to consider is nitrogen. WTP residuals tend to be very low in this nutrient. A WTP should always check the nitrogen content of the residuals to be land applied. Basing the application rate on nitrogen alone, however, may cause problems with the crop yield because aluminum and/or iron hydroxide solids present in WTP residuals are strong adsorbents of inorganic phosphorus. The low concentrations of phosphorus in WTP residuals may further diminish total phosphorus in soils, restricting plant growth.

As previously noted, concentrations of trace metals will likely determine the lifetime application amount of WTP residuals (metric tons per hectare, dry weight) on agricultural soils. Most metals in WTP residuals are bound in forms not readily released into solution due to their strong adsorption and co-precipitation by aluminum and iron hydroxides freshly formed in the coagulation treatment step.

Buffer zones between residuals applications, such as surface water, drinking water wells, drainage ditches, property lines, residences, schools, playgrounds, airports, and public roadways, play an important role in a successful WTP residual program. Identification of site-specific buffer zones may be critical to the land application program. In addition, many states have buffer zone regulations that must be followed. Guidelines for buffer zone delineation can be found in the following EPA publications:

- *Process Design Manual Land Application of Municipal Sludge* (U.S. EPA, 1995b).
- *Biosolids Management Handbook for Small Municipal Wastewater Treatment Plants, U.S. EPA Regions 7, 8, and 10* (U.S. EPA, 1995a).

Land application may be feasible only at certain times of the year, depending on the moisture conditions of the soil and weather conditions. Another, often overlooked factor is the local cropping cycle. Farmers will want to restrict application to certain times of the year, which will require additional or supplemental storage of WTP residuals. This timing and storage must be factored into the land application program.

Land application of WTP residuals can be accomplished through injection, incorporation, or surface application. Surface application requires the least additional equipment. Land application equipment is widely available, often from farm implement dealers.

8.4.2 Public Participation

Involving area residents may seem an obvious step but is often not considered. Those responsible for the use or disposal of WTP residuals must involve the community before starting the project. Holding public hearings is only a small part of a broad-based participatory process. Often the public can help make necessary decisions. Given information and allowed to participate, the public often becomes an advocate for the WTP. Target community leaders, place information in libraries, and talk to people. Community leaders include elected officials, church pastors, community activists, and others.

Consensus on the best solution is gained only through the joint effort of all involved: regulators, contractors, and the public.

8.4.3 Transportation

Transportation of WTP residuals represents a major cost and should be carefully examined. The first consideration is the type of WTP residuals being handled and transported, liquid or cake. The lower the percentage of solids, the more liquid the residuals, and the greater the volume of residuals that must be handled and transported.

Liquid can be transported either by pipeline or by truck. Pipeline offers the convenience of transporting residuals regardless of weather or other external factors. The distinct disadvantage, however, is that pipelines generally go to one location and limit the options for land application. To overcome this disadvantage, the WTP can maintain a large tract of land, enough to handle the residuals generated. Truck transportation offers WTPs more options for land application; therefore, most land application programs use this transportation method. Before various residuals dewatering and disposal alternatives can be compared, the costs of loading, dewatering, hauling, and disposal must be evaluated.

8.4.4 Monitoring

8.4.4.1 Residuals Sampling and Analysis

Either grab samples or composites can be used to analyze WTP residuals. WTP residuals range in consistency from liquid to solid; sampling techniques differ for each residual consistency. The U.S. Environmental Protection Agency (EPA) has developed guidance for sampling methods for sewage sludge that both explain (U.S. EPA, 1989) and show (U.S. EPA, 1993c) residuals sampling. The goal of all sampling is to obtain a representative sample, which is easiest to accomplish with a liquid being pumped.

One method to obtain a representative sample from a residual pile was developed in the early 1900s (first referenced in 1902) in the mining district of England. Many samples are taken from different depths and at different locations in the pile. A cone is built of the subsamples, and the cone is then flattened and quartered, thus the term “quartering.” Opposite quarters are re-coned and quartered again. This process continues until the required sample volume is reached.

Analytical methods for metals are found in the latest edition of *Test Methods for Evaluating Solid Waste* (U.S. EPA, 1986b). A common analytical error is for labs to conduct the metals analysis using a method developed for water and wastewater. In addition, the sample preparation method is as important for metals as the correct analytical method. All metals analysis samples should be prepared using SW-846 method 3050. With this preparation method, it is important to use equivalent to 1 gram dry weight of solids for the digestion. The results should be reported as dry weight, which requires the analysis of percent total solids so that dry weight can be calculated.

8.4.4.2 Soil Sampling and Analysis

Soil sampling is essential for accurate fertilizer recommendations. The recommendation may be used to calculate the quantity of WTP residuals to apply. With alum and ferric residuals, however, supplemental phosphorus fertilizer may be needed.

Total metals in the soil should be analyzed using the method described in *Methods of Soil Sampling and Analysis* (Page et al., 1982). Total metals should be analyzed rather than plant-available levels; methods vary depending on locale, and the scientific community still has concerns about the comparability of results. Local land grant universities should be consulted for more information. As a basic guideline, however, one set of composite samples should represent no more than 16 hectares.

When sampling an area of a field, samples should be taken from areas of similar soil characteristics, such as color, slope, and texture. Areas with different characteristics may require additional samples. A minimum of 20 subsamples for compositing should be taken from the sampling area, regardless of size. The sample should be taken to a depth approximately equal to the plow depth, discarding the surface litter of each sample. Samples can then be air dried and sent to the laboratory for analysis. Typically, the cost of metals analysis of WTP residuals ranges from $100 to $250 per sample. Other analysis, such as TCLP, may cost as much as $1,500. Soil analyses are similar in cost. The local land grant university may be contacted for both analysis and fertilizer recommendations.

8.4.4.3 Other Analyses

Other sampling techniques and chemical analyses may be required to assess a specific site or residual, such as plant tissue sampling and monitoring, ground-water monitoring, or deep soil monitoring. Deep soil monitoring samples soils to a specific depth; in EPA Region 8, depth is 5 feet. Each 1-foot increment is analyzed to assess migration of pollutants of concern, thus a facility can use deep soil monitoring instead of ground-water monitoring.

Chapter 9
Brine Waste Disposal

Currently, reverse osmosis (RO), electrodialysis (ED) or electrodialysis reversal (EDR), nanofiltration (NF), ultrafiltration (UF), and microfiltration (MF) are the only membrane processes being used or considered for use in treating public water supplies. All of these processes produce residuals waste streams, but not all generate waste streams that could be classified as brine—that is, as having a concentration of dissolved salt higher than that of ordinary seawater (Lapedes, 1978; Ingram, 1969). Only seawater RO, ED or EDR, and brackish water RO can produce brine waste streams. All other residuals waste streams from membrane processes are concentrates rather than brine. In fact, the American Society for Testing and Materials (ASTM) publishes a standard for definitions of water-related terms (ASTM, 1982) that interprets the residuals portion of an aqueous solution applied to a membrane as a concentrate. This chapter focuses primarily on the disposal of brine waste, with information and background on concentrates supplied as appropriate.

9.1 Background Information

9.1.1 Amount of Concentrate Generated and Disposal Methods

The U.S. Office of Technology Assessment (OTA) evaluates waste concentrate generation from membrane processes in terms of percent recovery of feed water and percent disposal as waste concentrate (Table 9-1).

Conventional methods of concentrate disposal involve disposal to surface bodies of water, spray irrigation combined with another dilution stream, deep well injection, drainfields, boreholes, or wastewater collection systems. These methods are usually the most cost-effective. Nonconventional methods are generally considered cost-prohibitive and are usually associated with a zero discharge scenario. These include use of evaporation and crystallization technologies and other ancillary equipment to concentrate the waste stream into a cake. Other nonconventional methods include evaporation by solar ponds or solar distillation.

Table 9-1. Membrane Concentrate Generation

Membrane Process	Percent Recovery of Feedwater	Percent Disposal as Concentrate
UF	80–90	10–20
NF	80–95	5–20
Brackish water RO	50–85	15–30
Seawater RO	20–40	60–80
ED	80–90	10–20

It should be noted that water treatment sometimes involves ion exchange, a separation process in which water is passed through a column containing an ion exchange resin or medium, often a cationic or anionic synthetic polymer. As the water passes through, the resins exchange their ions with those in the water according to the preference of the exchange sites for specific ions and the concentrations of these ions. In this way, ions from the water adsorb onto the resin. When the resin approaches its capacity for taking up ions, the ion exchange process is taken off line and the column regenerated by exposing the resin to a regenerant solution that, through ion exchange, converts the resin back to its initial form. The off-line steps include backwashing, regeneration, and rinsing. After regeneration, the ion exchange column is returned to service.

9.1.2 Constraints and Concerns

Concentrate disposal methods must be evaluated in light of geographic, environmental, and regulatory impacts. Geographic constraints generally have to do with limitations in local geology, hydrology, weather, and land availability. Environmental concerns such as impacts on wetlands, flora and fauna, and surface or ground-water pollution limit the use of some disposal methods in some locations.

At least eight states have established regulations to protect water resources from further degradation from brine or concentrate disposal. Section 305(b) of the Clean Water Act requires states to report to the U.S. Environmental Protection Agency (EPA) the extent to which their waters are meeting the goal of the Act and to recommend how compliance may be accomplished. According to the *National Water Quality Inventory 1986*

Report to Congress (U.S. EPA, 1986a), 36 of 52 (69 percent) U.S. states and territories reported brine/salinity as a major ground-water contaminant, indicating that proper disposal of concentrate should be a concern to all professionals engaged in the use of membrane processes. Table 9-2 summarizes concerns and requirements associated with conventional concentrate disposal methods.

9.1.3 Early Disposal Regulations

The State of Florida has by far the highest percentage of membrane process plants in the United States. A case study of permitting requirement, disposal option, and regulatory trends in Florida indicates the direction that concentrate disposal regulation is likely to take in the rest of the country. In Florida, regulatory agencies began governing concentrate disposal in the late 1960s. The relatively simplistic mass balance or dilution approach taken in that era evolved to a more complex approach by the mid-1980s. The trend towards more stringent regulations has continued into the 1990s. This trend is likely to occur in other states, such as California, Texas, and states along the east coast where use of membrane process plants is increasing.

Table 9-2. Concerns and Requirements Associated With Conventional Disposal Methods

Disposal Method	Regulatory Concerns	Other Requirements
Disposal to surface water	Receiving stream limitations Radionuclides Odors (hydrogen sulfide) Low dissolved oxygen levels Sulfide toxicity Low pH	Mixing zone Possible pretreatment Multiple port diffusers Modeling of receiving stream
Deep well injection	Confining layer Upconing to USDWs Injection well integrity Corrosivity	Well liner Monitoring well Periodic integrity test Water quality of concentrate must be compatible with the water quality in the injection zone
Spray irrigation	Ground-water protection	Monitoring wells Possible pretreatment Backup disposal method Need for irrigation water Availability of blend waters
Drainfield or borehole	Ground-water protection	Monitoring wells Proper soil conditions and/or rock permeability
Sanitary sewer collection systems	Effect on local wastewater treatment plant performance (toxicity to biomass or inhibited settleability in clarifiers)	None

Movement towards increasingly stringent regulation of concentrate disposal has been brought on by regulations passed to preserve and protect surface water bodies, bird sanctuaries, endangered flora and fauna, ground-water resources, and other environmentally sensitive areas. In particular, the permitting process has become more difficult. In the late 1960s, no permit was required to discharge concentrate to surface waters. Later, the State of Florida required a permit, but did not have a separate permit category for concentrate. Thus, the State of Florida permitted its first concentrate discharge in the early 1970s as an industrial waste. In retrospect, placing concentrate disposal under a separate waste permitting category would have been better because EPA later issued a new permitting requirement, the National Pollution Discharge Elimination System (NPDES), that required discharges to surface waters to be permitted. This resulted in dual permitting of concentrate discharges to surface waters.

As more recent membrane process technology began being applied to potable water treatment, systems began being sited farther away from coastal surface bodies of water and new disposal methods such as deep well injection began being employed. The first spray irrigation of diluted concentrate occurred in 1978, and the first deep well injection of concentrate occurred in 1983. Between 1978 and 1983, regulatory requirements had changed so much that only monitoring requirements were imposed on the spray irrigation project, while very stringent well construction integrity and monitoring requirements were required for the deep well injection project.

9.1.4 Current Regulations and Their Trends

Regulations vary widely from state to state and, in some cases, from county to county. In general, regulations are designed to protect surface- and ground-water resources; rarely do they specifically address the disposal of concentrates or brine. One specific regulation concerning the disposal of concentrate to ground waters was passed in Florida on July 28, 1988. Passage of an amendment to Chapter 17-28.700 of the Florida Administrative Code (FAC) allowed nonhazardous concentrates to be discharged through land application to aquifers containing greater than 1,500 milligrams per liter (mg/L) of total dissolved solids (TDS). More recently, increased public awareness, the activities of environmental organizations, and public concern have increased the difficulty of siting and permitting membrane process plants in Florida. In recent years, bills have been introduced in the Florida legislature to ban deep well injection. To date, these bills have failed, but one might be successful in the future.

After his election in 1994, Governor Bush of Texas banned the use of deep well injection. At the same time, Safe Drinking Water Act (SDWA) regulations are causing membrane processes to be declared best available technology (BAT) for the removal of many contaminants. Yet, without a viable method of concentrate disposal, membrane processes cannot be used. If pretreatment requirements drive the cost of disposal too high, some proposed membrane projects will not be cost-effective. Already, some cities have been unable to obtain disposal permits. At a recent meeting of the Florida Department of Environmental Protection (FDEP) Fact Finding Committee on Concentrate Disposal, it was recommended that a permit application be made in advance when alternative concentrate disposal methods will be proposed; application before beginning the detailed design of a proposed membrane process plant is intended to reduce losses if a plant cannot be permitted. It was further recommended that the permit application be made for the full plant nominal capacity required at buildout. Given that regulations are a moving target, it should be kept in mind that what is permittable now might not be permittable in the future.

9.2 Conventional Disposal Methods

9.2.1 Surface Water Discharge

Disposal of brine is a critical issue facing WTPs throughout the country. Because the majority of surface water discharges occur in Florida, the discussion that follows is based on Florida's experience.

The question of the environmental impact of brine disposal to surface waters was first addressed by the State of Florida in 1986. At that time, the state determined that concentrate disposal posed a significant threat to surface water quality and began requiring that any discharge to surface waters be permitted.

9.2.1.1 Constituents in Water Treatment Brines

Several constituents of WTP brines have been identified as potential threats to surface water quality. In Florida, ground water is the principal source of raw water. Ground waters are devoid of dissolved oxygen and contain hydrogen sulfide. This is a concern in relation to surface waters, because a lack of dissolved oxygen and the presence of detectable quantities of hydrogen sulfide are toxic to aquatic organisms. Fortunately, both conditions are easily remedied via a relatively simple and inexpensive treatment: aeration, which simultaneously adds dissolved oxygen and removes hydrogen sulfide. Some facilities have also removed hydrogen sulfide through chemical treatment. Typical Florida "end-of-pipe" permits require a minimum dissolved oxygen concentration of 5.0 mg/L and a maximum hydrogen sulfide concentration of 0.04 mg/L.

The presence of radionuclides in the brine can also pose a major water quality problem. Florida water quality criteria contain limits for combined radium (Ra-226 + Ra-228) and gross alpha. RO brines can contain combined radium at levels greater than 100 picoCuries per liter (pCi/L)—much higher than the Florida surface water standard of 5.0 pCi/L. Gross alpha levels are more difficult to measure due to interference from the dissolved solids in brines. High gross alpha values, however, typically occur with high levels of combined radium and should be handled similarly. The presence of radioactivity in any waste, including brine, can become an issue of great public concern, increasing the need to deal with radionuclide exceedances promptly and appropriately.

With brine, an additional disposal concern with surface water discharge is the impact of the concentrated salts (dissolved solids) on the receiving water. Water quality criteria for fresh water surface waters greatly limit the capacity of a fresh water body to receive brine. Unless the water body is a stream or river with a flow many times greater than the brine flow volume, discharge is not likely to meet the surface water criteria, even after mixing. Therefore, outfalls should be located to discharge brine waste to brackish or marine waters with adequate flushing.

Fluoride and several metals have been found in high concentrations in the brines of some facilities. Differences in the concentrations of these substances in brines from different facilities reflect differences in the composition of the ground water that provides the raw water to the facilities. In some cases, fluoride and metal concentrations in ground water exceed surface water criteria; in other cases, they do not exceed these criteria, but they are higher than the fluoride and metal concentrations in the receiving waters. When brine concentrations exceed receiving water concentrations, antidegradation issues must be addressed. The new metals criteria proposed by EPA and adopted in Florida are hardness-dependent for many metals. The ultimate impact of these changes has not been determined, but will require careful examination.

Nutrients are a new area of concern in Florida. As facilities have increased in size, their brine volumes have also increased. Today, nitrogen and phosphorus concentrations in large volume brines can provide a significant nutrient load to receiving waters. Because many Florida surface waters are already overenriched, significant nutrient additions are not likely to be approved.

A permit to discharge brine to a surface water will require establishing an effluent limitation—the brine quality necessary to ensure that the receiving water quality is not degraded beyond what is allowed by state and federal regulations. While effluent limitations are typically developed through application of sophisticated mathematical computer models, this is not always necessary for brine discharges.

9.2.1.2 Dilution, Mixing Zones, Modeling, and Multiport Diffusers

The hydrologic characteristics of receiving waters are important in considering surface water discharges of brines. The receiving waters' flow is the first issue that must be addressed. Most brine constituents are conservative materials that can be assimilated only through dilution. If the discharge dominates the receiving water, little dilution will occur. Small streams, for example, have little ability to dilute a discharge. Neither do lakes (even large lakes), because mixing and transport away from outfalls are slow, resulting in a localized buildup of brine constituents.

With the exception of aeration and hydrogen sulfide removal, treatment of brine has not been practical in Florida. Therefore, dilution has been the only way to meet receiving water quality criteria that are not met at the end of the pipe. Dilution calculations do not usually require sophisticated models. Often, desktop mass balance calculations are sufficient to determine the necessary effluent limitations. This is especially true when the surface water flow past the brine outfall is easily measured and an assumption of complete mixing is appropriate.

When an assumption of complete mixing is not appropriate, simple mixing models such as the EPA-supported CORMIX1, CORMIX2, UPLUME, and UMERGE can be used to establish effluent limits. The CORMIX models are particularly attractive because they are compiled from routines that the model chooses from based on input specified by the user. EPA has identified methods for determining tidal prisms, flushing times, and initial dilution that can aid in the calculation of mixing areas (U.S. EPA, 1985b). Many mixing models also permit analysis of the use of a multiport diffuser, which is often necessary to achieve adequate mixing in a reasonable area. Models should be used that consider the number, size, and orientation of multiple ports. Analysis is helpful because multiple ports do not always provide better mixing than a single port.

When concentrations in brine exceed surface water criteria, regulatory agencies may require the development of a mixing zone. In Florida, for example, a mixing zone is required for any discharge with end-of-pipe constituent concentrations in excess of numeric surface water criteria. No implied opportunity for mixing is given. In these cases, it is necessary to use a mixing model that establishes geographic boundaries as well as numeric constituent limits.

Assumptions specified in modeling exercises should be conservative. That is, they should examine worst case situations to ensure that the effluent limits developed will protect receiving water quality and comply with permit conditions. Minimum and maximum flows should be modeled to examine the dilution provided and estimate variations in brine plume boundaries under different conditions. These analyses require data on constituent concentrations in the receiving water and in the brine. If the facility is not yet constructed, analyzing brine from a pilot test facility might be possible. Otherwise, analysis of the raw water and calculation of expected brine concentrations will be necessary.

9.2.1.3 Toxicity Relative to Surface Water Discharges

Any discharge to surface waters must address the issue of toxicity. Discharges are typically screened for toxicity using whole effluent toxicity bioassay. These bioassays expose sensitive test organisms to 100 percent effluent for 48 (EPA) or 96 (some states) hours, while control organisms are exposed to “clean” water. At the end of the test, the number of deaths in the effluent is compared to that in the controls to determine if the effluent is acutely toxic. If an effluent is found to be toxic in this type of screening test, it is subjected to a definitive bioassay in which the organisms are exposed to various concentrations of effluent to determine the concentration lethal to 50 percent of the test organisms in the test period.

For toxic effluents, the degree of toxicity determines whether effluent limitations can be established, whether a mixing zone will allow the establishment of effluent limitations, or whether additional treatment to remove or reduce the toxicity will be required. In some cases, the facility may need to determine the cause of the toxicity through a Toxic Identification Evaluation (TIE).

In Florida, bioassays are required for all permitted surface water discharges. The frequency with which these tests must be performed varies with the volume of the discharge and the type of discharge. Because several discharges have failed bioassays, more test requirements are likely. The cause of brine toxicity has not been definitively determined and might vary among facilities.

9.2.2 Disposal to Sanitary Sewers

Discharge of concentrates to sanitary sewer systems is sometimes feasible if the concentrate mixture is not toxic and does not adversely affect the clarifier settleability or restrict final effluent disposal. Generally, the restrictions and requirements of sewer agency user ordinances must be followed. WTP agencies typically require detailed analytical screening before agreeing to accept a proposed brine or concentrate discharge. Special attention will be given to the discharge's effect on metal and hazardous constituent levels in WTP residuals, which must be disposed of safely.

In California and other western states, regional dedicated brine interceptor lines have been in place for many years. Originally installed to carry away waste brines from the oil industry and other industrial sources, they are now being used for residual brines from municipal well head treatment, such as brines from nitrate reduction ion exchange systems and concentrates from inland desalters used to reduce TDS in ground waters contaminated by years of agricultural operations. The cost of using these regional brine interceptors is often high—in some cases, higher than the cost of the proposed treatment processes. Costs include contribution-in-aid to construction of treatment facilities, other capacity charges, construction of pipelines from the treatment process to the interceptor lines, cost per acre or foot of discharging to the interceptor sewer, and pretreatment costs (if required).

9.2.3 Deep Well Injection

9.2.3.1 History of Deep Well Injection and Injection Regulations

During the last two decades, intensifying awareness of the problems associated with wastewater disposal into the surface and near-surface portions of the earth (e.g., lakes, rivers, ponds, irrigation fields, and shallow aquifers) has increased interest in deep well injection. As its name implies, deep well injection is the pumping of waste into deep geologic formations. The injection depth depends on site-specific geologic conditions and the quality of the receiving waters as set by minimum criteria in federal and local regulations.

Regulatory criteria for deep well injection of non-oil drilling waste waters was first formalized at the federal level in a Federal Water Quality Administration order dated October 15, 1970. That order only gave general guidance; it was not until 1973 that true criteria were established through the issuance of Administrative Decision Statement No. 5 by the Administrator of EPA. After the 1980 amendments to the 1974 SDWA, EPA consolidated the rules into the underground injection control (UIC) regulations and compiled them in the Code of Federal Regulations (40 CFR Parts 144 and 146), with general program requirements located in 40 CFR Part 122.

In May 1981, EPA developed a mechanism for granting individual states primary enforcement responsibility, or primacy, on UIC and issued a corresponding guidance paper. Many states have since developed their own UIC regulations, which, by law, can be stricter but not more lenient than the federal rules. Most states have opted to allow full regulatory control by EPA on certain types of injection wells by choosing not to develop and enforce rules that would give them primacy over those types of wells. Regulations in 40 CFR Part 147 describe how individual states operate under EPA rules.

9.2.3.2 Classes of Injection Wells

In the federal UIC regulations, injection wells are defined as wells into which fluids are injected. A well itself is defined as a bored, drilled, or driven shaft, or a dug hole, with a depth greater than the largest surface dimension. By definition, well injection means the subsurface emplacement of fluids through a bored, drilled, or driven well, or through a dug well, where the depth of the dug well is greater than the largest surface dimension (40 CFR Part 144.3). Specific inclusions covered by UIC regulations are septic tanks and cesspools used for hazardous waste disposal and those serving more than 20 persons per day.

Within this broad definition, EPA has classified injection wells into five different groups as described in 40 CFR Part 144.6 and 40 CFR Part 146.5. Paragraph 146.5 provides a detailed definition of Class V injection wells, while paragraph 144.6 does not. Otherwise, the two paragraphs are the same. The five classes of injection wells are:

- Class I injection wells:
  - Inject hazardous wastes beneath the lowermost formation containing a source of drinking water at least a quarter mile from the well.
  - Inject nonhazardous industrial and/or municipal fluids beneath the lowermost formation containing a source of drinking water at least a quarter mile from the well.
- Class II injection wells:
  - Inject nonhazardous fluids associated with production and storage of oil and natural gas.
- Class III injection wells:
  - Inject fluids for the extraction of minerals by solution mining, leaching mining, etc.
- Class IV injection wells:
  - Inject hazardous or radioactive water into or above a formation that contains a source of drinking water within a quarter mile of the well or that injects hazardous or radioactive water into aquifers exempted from UIC rules by the EPA Administrator.
- Class V injection wells:
  - All injection wells not included in the other four classifications, including air conditioning return wells, cesspools, cooling water return wells, drainage wells, recharge wells, salt intrusion barrier wells, thermal recovery return wells, experimental wells, etc.

Under these five classifications, injection wells used for the disposal of WTP residuals, such as concentrated salt water from desalination, are classified as Class I injection wells; within that classification, they belong to Group 2, which includes nonhazardous industrial wells. This industrial classification is significant because it invokes stricter requirements for well construction and monitoring, even when the injection fluids are of better quality than domestic injection fluids that can routinely be disposed of into Group 1 Class I injection wells.

On the other hand, when WTPs are used to remove hazardous waste from a source water, the residuals are considered hazardous water and extra precautions must be taken to ensure proper operation and monitoring of the injection system. In some states, hazardous waste injection wells are banned, and in several other states with primacy rights, banning hazardous waste injection wells is being considered at this time. In some cases, chemicals used in the treatment process could be considered hazardous wastes. When these chemicals are removed with the treatment residuals, they must be neutralized or removed before they can be injected.

9.2.3.3 Water Treatment Plant Residuals

Two types of residuals are important: those derived from the water being treated and those derived from the chemicals being used to treat the water. Assuming a noncontaminated source, residuals from water generally consist of concentrated salt and minerals, the concentrations and types of which depend on the source water. Added chemicals usually include antiscallants, acids, flocculants, and pH adjusting chemicals; they may also include nonhazardous cleaning solutions and other chemicals.

Residuals can be suspended or dissolved. When deep injection is considered, it is essential to evaluate the effect of mixing these residuals with the receiving waters in the injection zone. Mixing can cause the formation of precipitates, flocculants, gases, and bacterial mats, which can hinder the injection process and can harm the receiving aquifer by plugging it or by otherwise reducing its permeability.

9.2.3.4 Requirements of Injection Sites

An important element in the Class I Injection Well definition is the reference to injection below the lowermost formation containing an underground source of drinking water. An underground source of drinking water (USDW) is any nonexempted aquifer or portion of an aquifer that supplies water to a public water system (or is large enough to supply a public water system) and that does indeed currently supply drinking water to someone, as long as that water contains less than 10,000 mg/L of TDS. Thus, a Class I injection well must be sited such that injection occurs into an aquifer zone with water having at least 10,000 mg/L TDS (i.e., into waters that are not USDWs). In addition, the injection zone of the aquifer must be separated from the USDW zones above it by hydrologically impermeable formations that preclude upward migration of the injected fluids into the USDWs. These formations are referred to as confining beds, although in reality they are never fully confining, since, theoretically at least, every formation has some measure of permeability.

A third requirement is that the injection zone be a good receiving zone from both a water quality and hydrologic point of view. From the water quality point of view, the quality of the receiving water must not be aggressive to the injection fluid. The two waters must be compatible physically, chemically, and bacteriologically. From the hydrologic point of view, the receiving zone must be of high permeability and effective porosity such that the volumes of water injected can be discharged into the zone without excessively raising the pressure in the receiving formation. Excessive injection pressure can fracture the receiving zone and the confining layer above it, causing upward migration of the injection fluids into the USDWs. Similarly, fluid should not be injected into areas where the confining zone has fissures or fractures or is crossed by known faults, as these geologic features will become routes of injected fluid migration into the USDWs. In addition, injection should not occur in areas where seismic forces are active; in some cases, injection fluids have been suspected of acting as a lubricant to induce slippage in underground formations, resulting in seismic disturbance at the land surface.

9.2.3.5 Environmental Concerns of Deep Well Injection

The primary environmental concern with deep well injection is the potential for the injected fluids to migrate into potable water zones. This migration can occur via two mechanisms:

- Natural migration across confining layers that are not as confining as they should be, in which case the injection fluid migrates laterally into other potable zones through geologic faults or by simply flowing into other areas where the water is less than 10,000 mg/L TDS.
- Artificial migration of injected fluids through improperly constructed injection systems, such as defective cement, construction induced fracturing near the borehole, casing failures, etc.

Depending on the types of residuals being disposed, the waste to be discharged will have a higher or lower density than the receiving waters. Assuming a homogeneous aquifer, when the injection fluids are more dense, they sink to the bottom of the injection zone, migrating first downward and then horizontally. When the injection fluids are less dense than the receiving waters, they float on top of the injection zone, forming a bubble of fresher (less dense) fluids that produces an upward force across the confining zone. Similar conditions, in reverse, occur when the permeability of the injection zone is not homogeneous, (i.e., when the injection zone is more permeable in one direction than the other). For example, if the permeability is higher in the vertical than in the horizontal direction, vertical buoyant forces will induce a vertical movement of the injection fluid.

When vertical forces across the confining layer are induced by any of these injection pressures, vertical migration through the confining bed eventually occurs. This may take years, hundreds of years, or longer, but because no formation is totally impermeable, when the data are entered into a formula, there will always be a time when the fluid will move across the confining layer. This environmental problem is often mentioned qualitatively, although quantitatively it may be of little or no concern.

Two major concerns about confinement are uncertainty about whether the confining layer extends over a sufficiently large area and uncertainty about whether the confinement might be broken by faults, geologic caverns, or other geologic features. These issues are difficult to resolve because it is economically impossible to drill enough test holes to show that the confining layer exists everywhere over the area of concern. However, it is logical to expect that a thick confining bed will not suddenly thin out and disappear in a short distance. Similarly, while it is possible to show the existence of faults (typically identified through other sources in nearby sites), it is not possible to affirm that no faults exist.

Environmental concerns related to well construction and the problems associated with improper construction are discussed in Section 9.2.3.7.

9.2.3.6 Regulatory Requirements

Because ground water is the primary source of water for many communities, deep well injection is carefully regulated and monitored. Design and permitting are strictly regulated by EPA when not regulated by the state under primacy rules. Florida is a primacy state with a growing practice of using deep injection wells for desalination residuals disposal; it is a good example to describe how the permitting process works.

Permitting Requirements

In Florida, permitting and regulatory control of deep well injection is exercised by FDEP with aid from a technical advisory committee (TAC). This committee consists of five voting members who represent environmentally oriented government agencies. The voting members of the TAC are representatives of the regional office of the U.S. Geological Survey (USGS), EPA's Atlanta office, FDEP's Tallahassee office, the appropriate local Water Management District office, and the local FDEP district office. Two nonvoting members representing the local county pollution control office and the local county health department are also part of the TAC. The representative of the local FDEP district office serves as the TAC chairperson.

The TAC reviews and approves every engineering design and exercises control of every aspect of construction and testing from beginning to end. The TAC also reviews and approves specified construction materials and methods, including the size and weight of the steel casings, the cement and cement additives used in the casing installation, the method of drilling, the frequency and types of tests to be conducted, and other details. The TAC meets regularly with injection well engineers and owners to discuss project specifics, including review and approval of the depth of installation of each casing and the tests used to ensure mechanical integrity of the well. A preliminary Plan of Study is usually submitted to the TAC to inform the regulatory agencies about the project and the proposed method of construction and testing.

After extensive review, the TAC may suggest changes that are incorporated into a final Plan of Study. A formal permit application is then made for a test well which, if successful, usually becomes the first injection well of the system. In areas of Florida where the suitability of geologic conditions for deep well injection is in doubt, the test well may be preceded by an exploratory well, which is usually converted into a monitor well.

Under the Florida Administrative Code, there is a 90-day permitting clock for the permit application procedure. However, a permit is rarely issued within 90 days of application submission because the permitting clock stops each time a TAC member requests additional information. When the TAC chairperson is satisfied with the application, FDEP issues a notice of receipt of a complete application. This notice is published in the local newspaper and a public hearing date is set and advertised.

If there is no public opposition to the application at the hearing and no one requests additional public hearings, the TAC will begin reviewing of application and all associated data, including design documents and specifications. At the next regularly scheduled meeting of the TAC, voting members may give approval for the project to proceed with the test well or they may again request additional changes. After approval by the TAC, FDEP issues a second notice of intent to permit, which is also published in local newspapers, and another public hearing is advertised. If there is no opposition, FDEP will issue the permit to construct and test the test well. This permit usually lasts a year. At the end of the operational test period, the inspection well operator must apply for a final operating permit.

Design Requirements

Specifications and contract documents for permit applications must follow the design criteria required by regulatory agencies. These criteria require the use of:

- Well head vacuum release or air release valves to prevent fluid column separation and air entrapment.
- Hydropneumatic tanks to prevent water hammer.
- Cushioned valves and pump systems designed to maintain injection velocities below 8 feet/second (ft/sec) to prevent erosion in the well bore.
- Instrumentation and controls to maintain injection pressures below the fracturing limits of the formation receiving the treated effluent.

Designs must include a sufficient margin of error to ensure that the formation's ability to receive the effluent is not exceeded and that the opportunity for aquifer recovery is provided. The system must be designed to minimize water quality changes of injected fluid to avoid turbidity and air entrapment, to avoid formation of precipitates, and in general to ensure that nothing clogs the formation.

Construction requirements may include use of:

- A drilling pad capable of retaining spilled fluids during construction to avoid surface contamination.
- Multiple casings to protect each fresh water producing zone that is drilled.
- Casing and liner materials compatible with the injected fluids.
- Heavy wall casing (e.g., a minimum of 0.375-inch casing wall thickness).
- Sulfate-resistant cement only, a minimum of 2.5 inches of grout between exterior casings, and a minimum of 5 inches of grout outside of the injection casing.
- Appropriate annular fluid between the injection casing and the liner of the wells such that the packer in the annular space is not subject to chemical attack by that fluid.
- Backflow preventers and check valves.
- Well head designs that facilitate monitoring and logging without major disturbance to system operation and without requiring a drill rig on site.

Finally, monitoring, inspection, and testing—both during construction and during operation of the injection system—must be addressed in the design process. Principal elements include:

- Performing geophysical logging before and after each casing installation.
- Conducting downhole television surveys before casing installation and after final casing is installed.
- Sampling water quality at every change of formation or every 30 feet to detect the 10,000 mg/L TDS zone.
- Sampling and testing the quality of water in the injection zone, monitor zones, injection fluid, and shallow monitor wells.
- Monitoring annular pressure and the quality of annular fluid (industrial wells only).
- Conducting aquifer performance tests, injectivity tests, and mechanical integrity tests.
- Measuring and continuously monitoring and recording injection pressure and injection volume.
- Conducting recertification testing every 5 years.

9.2.3.7 Well Construction

A well must be properly constructed so that it serves its function without harming the environment. Proper construction ensures that a well passes construction-related and site condition-related mechanical integrity requirements. Of these, only construction-related mechanical integrity requirements can be controlled through proper construction, use of adequate materials, and proper operation and maintenance. Site conditions must be controlled by proper well siting, which usually occurs after the first well in a given area has been drilled to show that the site is adequate. Even in areas where other injection wells are operating satisfactorily, any new well should be nominated as a test injection well because of the possibility that the construction could turn out to be faulty or the site inadequate.

Contractor Selection

Selecting a qualified well driller for a project is best accomplished by prequalifying drillers based on their experience and reputation and by inviting qualified firms to bid on the specifications and drawings. Special attention must be given to ensuring that drillers have the proper equipment to complete the contract on schedule. The lowest responsible bidder is generally awarded the contract. Private firms may choose to negotiate contracts rather than select a driller through competitive bidding. Bid price is influenced by a number of conditions, especially the volume of work available at the time of bidding.

Drilling Pad and Drilling Equipment

The first stage in drilling the deep injection well involves drilling the shallow portion of the hole and setting the surface casing. By specifying a small drilling rig during this stage, costs can be minimized.

Once the surface casing is installed, an impervious drilling pad must be constructed to prevent contamination of the aquifer by salt water spills during construction. The pad is constructed of 4- to 6-inch thick reinforced concrete and is surrounded by a berm. All drilling equipment, fluid circulation equipment, and settling tanks are installed on top of this drilling pad. Shallow monitor wells drilled around the pad serve as an early warning system to detect salt water spills. As an additional precaution against spillage, a temporary flow control device is installed before drilling through salt water zones.

After the surface casing is set and the pad and shallow monitor wells are complete, a large drilling rig is brought on site. The well size and depth will determine the type of rig. Assembling a rig and installing all peripheral equipment can take up to a month.

Fluid Circulation System and Other Equipment

During drilling, all drilling fluid is recirculated through a series of settling tanks to allow collection of rock cuttings from the hole. Provisions are sometimes made for temporary storage of salt water overflow in polyvinyl chloride (PVC) lined ponds. This salt water can later be reinjected into the well. Cement and cement additives for grouting casings usually are brought to the site dry and are stored until they are mixed and pumped by a cement company specializing in well grouting.

Field offices of the engineer and drilling contractor are generally located on site. Once drilling starts, it usually continues 24 hours a day, 7 days a week. Construction supervision should be provided around the clock by the consulting engineer.

Geologic Sampling—Proper Well Siting

Formation samples are collected at preselected intervals and at every change of geologic formation. Samples are catalogued according to their physical characteristics and mineral composition to identify confining layers and injection zones.

Several formation cores must be obtained from specific depths in the well. The best core samples typically come from the confining zones, since samples from the injection zones are usually broken and fractured. After field inspection and cataloguing, the cores are shipped to a laboratory where they are tested for physical and chemical characteristics, including permeability to water. These tests determine if the confining beds are sufficiently impermeable to meet regulatory standards.

Construction Procedures

All drilling is preceded by a pilot hole 8 inches or less in diameter, which yields better formation samples and facilitates more accurate geophysical logs. The pilot hole is reamed to full size after it has reached total depth for casing installation. Full size is usually about 6 inches larger than the nominal diameter of the casing to be installed. The one exception is the reamed hole for the innermost (injection) casing. This hole is always 10 inches larger in diameter than the injection casing to maintain the minimum requirement of 5 inches of cement grout in the annular space around the injection casing

Casing Installation

Injection well construction requires several sizes of casings, with the largest diameter being used at the surface and successively smaller casings being used deeper into the ground. The smallest is the injection casing at the bottom of the well.

A drilling rig elevator and an auxiliary elevator are used to raise each casing to the top of the drilling platform and then to lower it into the hole. Special care should be exercised to prevent damage during installation. As sections of casings are lowered into the well bore, they are welded into place. To ensure that there are no voids in the welds, two full welding passes should be performed. Inspectors should examine each weld and pressure tests should be conducted to detect any imperfection. If a defect is found, corrective action can be taken before the well is put into operation. The welding operation usually continues around the clock over a period of several days. The pressure test on the final string of casing must be conducted at 1.5 times the estimated injection pressure when the well is completed and placed into operation.

Cementing Operations

After the casing has been installed, cement must be placed in the annular space between the walls of the well hole (or inside a telescoping casing) and in the casing itself. A sulfate-resistant cement (Type II, Class H) is generally used because deep aquifers contain unusually high levels of sulfate. Cementing is accomplished by pumping cement through a pumper truck into a small diameter “tremie” pipe in the annular space at the bottom of the well hole. As the annular space fills with cement, the tremie is slowly pulled out of the hole.

The first cement installation (lift) usually contains no additives. Subsequent cement lifts usually contain various concentrations of additives to give the cement more or less plasticity, faster or slower setting times, or greater or lesser density to fill voids or cavities in the formation. The cement mix and additives are selected by the cement engineer and the consulting engineer after a review of geologic samples and geophysical logs. The local regulatory program is responsible for and must review and approve the cement program. If no local program exists, EPA must review and approve the cement program prior to implementation.

Water Sampling

Throughout the drilling process, representative water samples must be collected from discrete sections of a formation to define water quality at specific depths in the hole. Samples are collected using a packer consisting of a perforated pipe with two inflatable rubber seals at each end. When the seals are inflated and the packer assembly is pumped, a water sample is obtained from the formation section between the seals. These samples are pretreated at the site and shipped to a laboratory for analysis. Time-dependent parameters are measured in the field.

Injection Test

The injection test is performed by injecting clean water into the injection well and monitoring pressure changes and water quality. Desalination reject water cannot be used for injection tests because system failure could result in ground-water or USDW contamination.

Performing an injection test requires that the monitor well be completed and equipped with gauges to monitor pressure changes—typically, a fluid-filled gauge backed by a mercury manometer that measures pressure in inches of mercury or 10-inch and 12-inch test grade gauges. Electronic gauges are usually avoided because they can be unreliable in injection test conditions. Use of pressure transducers installed at the bottom of the hole is ideal because they eliminate the need to correct for pipe friction losses and water density differences. All pressure readings are corrected to feet of water and are referred to a fixed datum, usually the surface of the drilling pad. This calculation facilitates monitoring of injection pressure and well injectivity over time.

Samples of the water being injected are collected at various intervals during the test to determine the quality, density, and temperature of the water.

After the well is constructed and the injection test is performed, a downhole television survey of the well should be conducted using a slim television camera equipped with a fish-eye lens and a light source. The videotape of the downhole survey is used to confirm that there are no holes in the casing and no defective joints. The videotape also provides a permanent visual record of the injection zone before injection occurred so that a comparative analysis of the casing and injection zone can be performed in the future.

Other geophysical surveys are usually conducted at the same time to complement those conducted during construction.

Well Head Completion and Monitoring

Proper design of the well head will facilitate collection of all future monitoring data. Easily readable gauges at the injection and deep monitor wells should always be specified and, if funding is sufficient, remote reading transducers are highly recommended.

The most important design elements are the pressure and vacuum release valves because they are the ultimate line of protection against high pressure and vacuums that can develop during electrical malfunctions or scheduled and unscheduled shutdowns. A hydropneumatic tank system should also be added to all wells to further increase protection.

Proper valve design also ensures safe access to the well when geophysical logs and television surveys are conducted in the future. For example, proper valve design permits insertion of probes (including the television camera) and avoids spills of injected residuals.

9.2.3.8 Operational Considerations

Any injection system operating program must reflect the hydrogeological characteristics of the injection zone and the volume and chemistry of the injection fluids. The injection rate determines the injection pressure, which may not exceed the maximum value for the injection system. The maximum allowable injection pressure is the highest pressure that:

- Preserves the integrity of the formations in the injection zone and the overlying confining zone.
- Prevents significant change in the fluid movement capabilities of the overlying confining zone.
- Protects the mechanical integrity of the well structure.

Similarly, the permissible injection rate for a deep injection well is that which:

- Protects the mechanical integrity of the well structure.
- Preserves the integrity of the formations in the injection zone.
- Preserves the fluid movement capabilities of the injection zone.

Injection Pressure and Preservation of Formation Integrity

Safe operating pressures can be determined with certainty only by conducting fracture tests on the formations in the injection zone. The maximum safe bottom-hole injection pressure typically ranges from about 0.5 to 1.0 pounds per square inch (psi) per foot of well depth, depending on the types of materials that make up the formations and geological conditions in the open hole. In Florida, for example, a value of 0.6 psi per foot of well depth is typically used to calculate the maximum safe bottom-hole injection pressure for a 2,700-foot deep injection well into a dolomitic formation. Multiplying 0.6 per foot of well depth by the well depth of 2,700 feet yields a maximum safe bottom-hole injection pressure of 1,620 psi.

The maximum safe *well head* injection pressure is the difference between the maximum safe bottom-hole injection pressure and the measured bottom-hole hydrostatic pressure. If the system in the example above produced a bottom-hole hydrostatic pressure of 1,200 psi during pumping at a design capacity, the difference between the maximum safe bottom-hole injection pressure and the measured bottom-hole hydrostatic pressure would be 420 psi (1,600 to 1,200 psi). Dividing 420 psi by a safety factor of 3.0 yields a maximum well head injection pressure of 140 psi for the well. The safety factor might differ from site to site because it is an engineering designer's choice, not a regulatory requirement. Friction losses are then calculated for the design flow and the diameter of the injection liner. High friction losses are subtracted from the 140 psi to yield the maximum safe well head injection pressure to be used in monitoring reports.

Injection Pressure and Prevention of Fluid Movement in the Confining Zone

Success in preventing significant change in the fluid movement capabilities of the confining zone is somewhat difficult to evaluate due to different interpretations of the word "significant." Analyses of cores and geophysical logs from the confining zone should show low permeability values. Regardless of how small, however, any permeability value entered into a leakage equation will result in some leakage value. Determining what is or is not significant leakage is subjective.

Injection into a well generates piezometric heads in all radial directions. The amount of leakage should be calculated within the area determined to be critical—usually the quarter-mile radius or area of review (AOR) from the injection well. Beyond the 1/4-mile radius of the AOR, the differential pressure across the confining zone should also be calculated to determine when pressure buildup will (at least in theory) produce leakage beyond the AOR.

Injection Pressure and Protection of Well Mechanical Integrity

The maximum well head injection pressure for a deep injection well must also protect the mechanical integrity of the well structure. In particular, it must be lower than the bursting pressure of the steel pipe used for the inner (injection) casing and the bursting pressure of the injection liner within the inner casing. The bursting pressure of water filled, unsupported pipes of various diameters and wall thicknesses is provided by the manufacturer. Because the steel pipe is always encased in cement grout the entire length of the well, the maximum permissible pressure inside the casing, is much higher than the bursting pressure of the unsupported casing. As a result, it is not usually a significant consideration in design, construction, or operation. For the liner, the collapsible pressure rather than the bursting pressure is more important, especially during construction. For example, an unsupported fiberglass casing with a bursting pressure of 600 psi might withstand only 100 psi in compression. Thus, the use of a 600-psi injection pressure must be weighed against the need to keep the annular pressure below 100 psi.

The annular packer also influences mechanical integrity. Packers should be tested to hold in tension and in compression.

Injection Pressure Data Reporting

A data reporting program must be part of operation and maintenance (O&M) procedures. The program should include regular reporting of operational well head injection pressures. If at any time the injection pressures show a rapid increase toward the maximum, measures should be taken to alleviate an impending violation before it occurs.

If, under normal operating conditions, the well head injection pressure suddenly starts to approach the permitted maximum, the injection pumps and deep injection well should be shut down until an emergency evaluation can be conducted and other contingency plans put into operation. On the other hand, if the well head injection pressure rises result from excessive injection rates, the injection should be throttled back immediately to a rate that will reduce the well head injection pressure to below the maximum.

Injection Rate and Protection of Well Structure

Due to the potential for damage, maximum injection rates cannot be established through empirical injection rate tests. Instead, standard practice has been to establish a maximum permissible injection velocity of 8 ft/sec, the maximum velocity that can be expected to not create erosion of the open hole in most formations. This rate has been adopted as the ruling criterion for maximum safe injection rate in some states.

The 8-ft/sec velocity limit protects the injection well from large flow velocities that can lower fluid pressure close to the saturated vapor pressure at the injection pump (or at any other flow obstruction in the pipeline, such as a sudden turn or constriction). Lowering fluid pressure within the injection line can lead to cavitation within the pump and the injection line, resulting in fluid flow interruptions that can damage the system. Because higher injection rates within a well require larger injection velocities, maintaining lower rates minimizes the possibility of damaging pressure reductions.

Injection Rate and Preservation of Formation Integrity

Stable operating conditions also help protect the integrity of the injection zone. Pressure fluctuations under variable injection rates create variable stress-strain relationships within formations, particularly the rock shelves that separate the porous zones. These stresses and strains in turn can induce rapid cyclical flexing of the rock matrix, which can affect the rock matrix and fracture the formations. To prevent fracturing, rapid cyclical injection should be kept to a minimum. This can be accomplished by using the hydropneumatic tank and pressure and vacuum release valves discussed above.

Injection Rate and Protection of Injection Zone Capabilities

The transmissivity of the injection zone and its ability to receive injection fluid must also be considered. The maximum rate of injection that can be received by the injection zone is largely limitless as long as the well structure and the rock matrix within the injection zone are able to receive the fluid. Nevertheless, economic considerations favor the use of the lowest possible injection pressure, since the frictional head loss increases exponentially with increasing flow rate.

In general, therefore, the limiting criterion for increasing injection rate is a matter of economics and the need to protect the well structure and injection zone. The maximum safe injection rate for a deep injection well is determined by the most restrictive of those considerations, usually the 8-ft/sec limitation on fluid velocity within the injection pipe.

9.2.3.9 Monitoring Requirements

During the time that a deep injection well is operational, some types of monitoring must be conducted on a regular basis and others must be carried out on an as-needed basis.

Regular Monitoring

Regular Injection Well Monitoring. Regular monitoring of a deep injection well consists of:

- Keeping continuous record of the variable nature of the flow rate for each 24-hour period and recording the daily flow volume. Data on the daily flow rate and the total monthly flow, together with the average and maximum daily flow and the maximum instantaneous rate, must be reported monthly.
- Recording the variable nature of the well head injection pressures. Daily average and daily maximum injection pressures must be recorded for each 24-hour period and reported monthly.
- Recording the annular space pressure to show any fluctuations with flow rate. The annular space volume contracts and expands with injection pressure changes as well as with the temperature changes of the injection fluid.
- Running and reporting a specific injectivity test at pre-approved intervals to ensure that the receiving formations are not being affected by excessive plugging or excessive dissolution.
- Monitoring the water quality of the WTP residuals in accordance with required parameters in the regulations. Water quality must be reported monthly, but is usually reported by the plant rather than the well operator.

A sample report format (adapted from forms used by FDEP) is shown in Figures 9-1 through 9-5.

The injected fluid must also be tested weekly as part of the operation of the well itself. These tests involve taking a sufficient quantity of injection fluid from the pipeline between the WTP and the deep injection well. For analysis for TDS, total suspended solids (TSS), chloride, sulfate, conductivity, pH, and temperature. The temperature reading and the pH must be taken immediately after the sample has been collected. The remaining water quality parameters may be determined later in the water quality testing laboratory. An ongoing record of the various parameters must be maintained.

All of the data must be kept on site and available for inspection by regulatory authorities on request. The log sheets must include any annotations and will constitute the record of operation.

At the time of sample collection or sample analysis, the following monitoring information must be provided:

- Date, exact place, and time of sampling or measurements.
- Person responsible for performing the sampling or measurements.
- Date(s) analyses were performed.
- Person responsible for performing the analyses.
- Analytical techniques or methods used.
- Results of the analyses.

Regular Monitor Well Monitoring. Monitor wells are usually used to monitor two water-producing zones above the injection zone: one at the first permeable zone above the injection zone and one at the first USDW above the confining zone. As part of overall ground-water monitoring, those zones must be tested on a monthly basis and the results reported to regulatory agencies.

MONTHLY OPERATIONAL REPORT

I. OPERATING PERIOD ______ MONTH ______ YEAR ______

II. INJECTION WELL OPERATOR

1. Name ____________________
2. Address ____________________
3. City ____________________ State ____________________
4. Phone Number ____________________
5. Permit Number ____________________

III. SUMMARY OF OPERATIONAL DATA

A. Injection Volumes

1. Maximum daily volume specified in permit ________ gal/day
2. Maximum daily volume during operating period ________ gal/day
3. Present average daily volume ________ gal/day
4. Total volume injected to date ________ gal

B. Injection Rate

1. Maximum injection rate specified in permit ________ gpm
2. Maximum injection rate during month ________ gpm
3. Minimum operational injection rate during month ________ gpm
4. Average injection rate during month ________ gpm

C. Injection Pressure

1. Maximum well head injection pressure specified in permit ________ psi
2. Maximum well head injection pressure during month ________ psi
3. Minimum operation well head injection pressure during month ________ psi
4. Estimated average well head injection pressure during month ________ psi

IV. INSTRUCTIONS

A. The operator of the injection system shall furnish information on this form not later than the 15th day of the month following the month reported.

B. Report any irregularities relative to daily injection practices on reverse side of this page.

C. All data will be retained on site and made available upon request.

D. All operational problems and significant changes in injection systems or wastes are to be reported when they occur.

Signed ____________________ Date ____________________

Figure 9-1. Monthly operational report, page 1.

MONTHLY OPERATIONAL REPORT - Page 2

1. Continuous Operating Period					2. Injection Rate (gpm)			3. Well Head Injection Pressure (psi)			4. Total Cumulative Fluid Injected	
Start		End		Length of Operating Period (Days:Hours)								
Date	Time	Date	Time		Maximum	Minimum	Average	Maximum	Minimum	Estimated Average	Daily	Cumulative
1 2 3 4 5												
6 7 8 9 10												
11 12 13 14 15												
16 17 18 19 20												
21 22 23 24 25												
26 27 28 29 30 31												

		SPECIFIC INJECTIVITY	Date:	By:
Injection Rage (gpm)	Total Pressure (psig)	Shut-In Pressure (psig)	Specific Pressure (psig)	Specific Injectivity Index (gpm/Specific Pressure psig)
				gpm/psig

Figure 9-2. Monthly operational report, page 2.

MONTHLY OPERATIONAL REPORT - Page 3

INJECTION WELL

I. Operating Period: ________________, 19________

II. Sampled by: 1. Name ______________________________ Address: ______________________________

City ______________________ State: __________ Phone Number: ______________________________

2. Permit Number: ______________________________

Signed: ______________________________ Date: ______________________________

Month		Date of Sample	BOD (mg/L)	Total Dissolved Solids (mg/L)	Total Suspended Solids (mg/L)	Chloride (mg/L)	Sulfate (mg/L)	Conductivity (µmhos/cm)	Temperature (°C)	Fecal Coliform (Colonies/ 100 ml)	pH (range)
Starting	Ending										
Starting	Ending										
Starting	Ending										
Starting	Ending										
Starting	Ending										

Figure 9-3. Monthly operational report, page 3.

MONTHLY OPERATIONAL REPORT - Page 4

MONITOR WELL

I. Operating Period: ____________________, 19_______

II. Sampled by: 1. Name __________________________________ Address: ____________________________

City __________________________ State: ________ Phone Number: ____________________________

2. Permit Number: ________________________________

Signed: ____________________________________ Date: ________________________________

Month		Date of Sample	Monitor Zone	BOD (mg/L)	Total Dissolved Solids (mg/L)	Chloride (mg/L)	Sulfate (mg/L)	Conductivity (µmhos/cm)	Temperature (°C)	Fecal Coliform (Colonies/ 100 ml)	pH (range)
			Upper								
Starting	Ending		Lower								
			Upper								
Starting	Ending		Lower								
			Upper								
Starting	Ending		Lower								
			Upper								
Starting	Ending		Lower								
			Upper								
Starting	Ending		Lower								

Figure 9-4. Monthly operational report, page 4.

MONTHLY OPERATIONAL REPORT - Page 5

DUAL ZONE DEEP MONITOR WELL
DAILY WATER LEVELS

I. Operating Period: ____________________, 19______

II. Sampled by: 1. Name ________________________________ Address: ________________________________

City ________________________ State: __________ Phone Number: ________________________________

2. Permit Number: ________________________________

Signed: ________________________________ Date: ________________________________

	January		February		March		April		May		June		July		August		September		October		November		December	
Days	Upper Zone (psi)	Lower Zone (feet)	Upper Zone (psi)	Lower Zone (feet)	Upper Zone (psi)	Lower Zone (feet)	Upper Zone (psi)	Lower Zone (feet)	Upper Zone (psi)	Lower Zone (feet)	Upper Zone (psi)	Lower Zone (feet)	Upper Zone (psi)	Lower Zone (feet)	Upper Zone (psi)	Lower Zone (feet)	Upper Zone (psi)	Lower Zone (feet)	Upper Zone (psi)	Lower Zone (feet)	Upper Zone (psi)	Lower Zone (feet)	Upper Zone (psi)	Lower Zone (feet)
1																								
2																								
3																								
4																								
5																								
6																								
7																								
8																								
9																								
10																								
11																								
12																								
13																								
14																								
15																								
16																								
17																								
18																								
19																								
20																								
21																								
22																								
23																								
24																								
25																								
26																								
27																								
28																								
29																								
30																								
31																								

Figure 9-5. Monthly operational report, page 5.

Monthly monitoring of the monitor zones must include the following steps:

- Recording the pressure or water level in each zone, noting monthly daily highs and lows on the monthly form.
- Collecting a monthly water quality sample from each of the two monitor zones (every 2 weeks for the shakedown test period).

The water quality samples collected monthly (or bimonthly at first) must be taken to a water quality testing laboratory to measure TDS, chloride, sulfate, and specific conductances (conductivity). Temperature and pH should be measured in the field immediately after sample collection. Records of monitoring information should include:

- Date, exact place, and time of sampling or measurements.
- Person responsible for performing the sampling or measurements.
- Date(s) analyses were performed.
- Person responsible for performing the analyses.
- Analytical techniques or methods used.
- Results of the analyses.

The results from the water quality analyses must be submitted on a monthly basis. A typical report format is shown in Figures 9-1 through 9-5.

Special As-Needed Monitoring

Special injection well monitoring may consist of the following tests, either alone or in combination: mechanical integrity test, pressure test, television survey, noise and radioactive tracer test, and controlled injection test (injectivity test).

The mechanical integrity of a deep injection well must be checked every 5 years via a 1-hour pressure test on the injection casing. To perform this test, a temporary packer must first be installed at the bottom of the injection liner. Within 3 months, the results of the pressure test must be submitted to regulatory authorities with an interpretative letter. Copies of the results must also be retained at the plant office.

A television survey of the deep injection well must also be made every 5 years. This involves running a television camera down to the maximum depth possible within the open hole. Within 3 months of the survey, a copy of the videotape must be submitted with an interpretative letter, and a copy of the videotape must be retained on site. Similarly, a radioactive tracer test and/or a noise log may also be required.

If regular monitoring records begin to indicate significant increases in well head injection pressures over time, a controlled injection test and/or television survey may be required before 5 years to ascertain the probable cause. If a sudden significant increase occurs in well head injection pressures, regulatory agencies must be notified within 24 hours. A testing program could then be required for the deep injection well, which would probably include a controlled injection test and a television survey. After the results from these tests are analyzed, an interpretative report must be submitted within 3 months. If the tests results require any work in the deep injection well, the results of the work must also be submitted within 3 months.

A routine injectivity test must be performed every month during the shakedown period. Subsequently, a special injectivity test must be performed quarterly or as needed. For meaningful comparisons between tests, standard test procedures must be followed exactly each time the test is conducted. These procedures include:

- Once a month, allowing the final effluent storage tank to fill sufficiently to allow the injection system to operate at the maximum permissible injection rate.
- Measuring the shut-in pressure in the well (prior to the start of the injectivity test).
- Connecting the injection pump directly to the injection well (bypassing the hydropneumatic tank) and operating it at the predetermined number of revolutions per minute that will result in the maximum permissible injection rate.
- After 1 hour of injection at the maximum permissible rate, recording the injection pressure at the well head. This must be repeated again just prior to the end of the test, at the end of the second hour.
- Arranging the two pressure readings together to determine the specific injection pressure for the injection rate used.
- Reporting the specific injectivity index as the ratio of the fixed volumetric rate to the specific pressure (the average of the two readings). This result is the specific injectivity index (in gallons per minute/specific pressure).
- Preparing a running chart by plotting the locations of the monthly results. In the chart, the x and y coordinates represent the volumetric rates and their pressures, respectively.

Sampling Procedures

Water quality monitoring of desalination plant residuals is discussed in Section 10.3.3.1. Sampling procedures for the deep monitor well require samples to be collected after the system has been properly evacuated. The methodology must be customized for each well.

Shutdown and Startup Procedures

Normal Operation. Once put into operation, injection wells operate automatically without the need to open or close any valves. The air release and vacuum release valves on the well head are the only valves that open and close with any regularity, and they do so automatically. Moreover, the hydropneumatic tank acts as a buffer and creates redundancy so that even the automatically operated air and vacuum release valves need not be controlled in any way. In fact, the control valves to the air release and vacuum release valves should always be in the open position whether or not the well is in operation. The valve that connects the release valves to the well should always be open as well.

Startup. Before the well is put into operation and when it is shut down, all of the valves should be closed. Once the injection pump is turned on, the master valve (a slow-closing butterfly valve that connects the well to the plant) should be opened within 30 seconds to prevent buildup of pressure in case the hydropneumatic tank is bypassed. If the hydropneumatic tank is not bypassed, the capacity available in the tank reduces the need to open the master valve immediately. Even then, however, the master valve must be opened before the hydropneumatic tank fills (within 1 to 2 minutes).

Shutdown. Shutting down the well is a straightforward operation, requiring only that the master valve be closed. If the shutdown is temporary, other valves may be left open. If the shutdown is long-term, all valves except the vacuum and pressure valves should also be closed.

Attaching a lockable chain to all valves to prevent tampering is strongly recommended. Alternatively, the valve handles can be removed. The master valve should also be specially protected by a chain lock or by removing its handwheel. Valves used for water sampling should be shut down at all times except while sampling. The handles should be removed when not in use to prevent unauthorized opening, which would waste the annular fluid.

Schedules and Procedures for Calibration and Operation of Pumps and Monitoring Equipment

Calibration procedures depend primarily on the types of equipment used and the manufacturers' recommendations. Each equipment manufacturer has specific procedures that are documented in reference manuals; these should be followed. Single copies of these manuals are provided by the manufacturer through the contractor and are to be retained on site at all times.

9.2.3.10 Data Reporting

Data reporting requirements for a deep injection well include:

- Regular monthly operational reports
  - Pressures and volumes (injection well)
  - Water quality (injection well)
  - Water quality (upper and lower monitor zones)
  - Pressures (upper and lower monitor zones)
- Special interpretative reports
- Noncompliance reports

Regular Monthly Operational Reports

Operational reports for each calendar month should be submitted no later than the 15th day of the following month or according to permit requirements.

Pressures and volumes (Injection well). This part of the monthly report (see Figures 9-1 and 9-2) follows the EPA-suggested reporting format for injection wells. Page 1 of the monthly operational report contains four sections. Sections I and II are self-explanatory. Section III details restrictions on the maximum daily injection volume, maximum injection rate, and maximum well head injection pressure. The maximum and average daily injection volumes and the cumulative total volume for the month are recorded in Section III-A. Sections III-B and III-C are used to record the maximum, average, and minimum injection rates and the well head injection pressures for the month. Section IV contains instructions for filling out the report.

The maximum daily injection volume during the operating period is the largest of the average daily injection volumes calculated for each of the operating periods during the month. The present average daily volume for the month is the total volume injected for the month divided by the sum of the lengths of the operating periods during that month. The total volume injected to date is taken directly at the end of the month from the daily volumes recorded.

The maximum injection rate during the month is the largest of the daily maximum injection rates. The minimum operational injection rate is the lowest of the daily minima. The average injection rate during the month is equivalent to the present average daily volume recorded in Section III-A-3, except for a time that includes only the length of actual injection.

The maximum well head injection pressure during the month is the largest of the daily maxima. The minimum operational well head injection pressure is the lowest of the daily minima. The estimated average well head injection pressure during the month must be estimated for each operating period from the daily values. The time-weighted average is calculated from the various estimated average well head injection pressures for the month.

Column 1 on page 2 is used to record the start, end, and length of each operating period during the month. The maximum and minimum injection rates must be read each day and recorded in Column 2. The average injection rate is calculated from the daily total flow at the end of each day, dividing the difference by the duration of operation that day. Both the maximum and minimum well head injection pressures are recorded each day in Column 3. The average daily well head injection pressure is also estimated each day and recorded in Column 3. The total volumes are recorded at the end of each day whenever operating periods exceed 1 day. If the operational period is less than a day, the flow at the end of the period is recorded.

Water quality (Injection well). Page 3 of the monthly report (see Figure 9-3) is used to list the laboratory results of tests performed on samples of the injection fluid after it has passed through the desalination treatment plant (just prior to injection). The report form provides space for recording the results of the tests recommended for inclusion in the monitoring program. The water quality samples should be sent to a certified water quality testing laboratory, which will return the test results to the operator for inclusion in the monthly operational report.

Water quality (Upper and lower monitor zones). Page 4 of the monthly operational report (see Figure 9-4) includes data from both the upper and lower monitor zones. It covers the range of parameters described earlier in this chapter. Data analysis must be performed by a certified laboratory.

Pressures (Upper and lower monitor zones). Page 5 of the monthly operational report (see Figure 9-5) should be filled in whenever water level measurements are taken. Water level data for both monitor zones should be recorded daily. The water level of the lower monitor zone can be measured by taping down to the water surface and recording the distance (as depth to water, in feet) below the measuring point (MP). Alternatively, a calibrated recording bubbler system installed on the well can be used as long as the calibration holds true. The water level for the upper monitor zone can be read directly from the gauge or recorder.

Interpretative Reports

Interpretative reports are usually submitted following any type of special monitoring. These reports typically consist of letters; the specific format depends on the nature of the special monitoring. Interpretive reports are usually developed by the engineering consultants for the WTP and are submitted within 3 months of completion of the special monitoring.

Noncompliance Reports

Noncompliance reports are required in the case of abnormal events (e.g., equipment breakdown, power failure, clogging of the well, etc.). In such cases, a verbal report must be made within 24 hours, a preliminary written report must be submitted within 3 days, and a comprehensive written report must be submitted within 2 weeks. These reports must describe the nature and cause of the event, the period of noncompliance (dates and times), steps taken to correct the problem and to prevent its recurrence, emergency procedures in use pending correction of the problem, and the estimated date by which the facility will again be functioning in accordance with permit conditions.

If the deep injection well is abandoned, the owner and/or operator must provide adequate documentation indicating that the well was properly abandoned. The documentation must describe the method of abandonment and evaluate the results of the operation.

9.2.3.11 Stimulation Program

An injection zone intersected by the open hole of a properly operating deep injection well derives its ability to receive injection fluids from natural cavities in the rock matrix and from larger interstices. These cavities are relatively large and numerous and are not usually susceptible to clogging, unlike the smaller cavities within the rock matrix itself. Therefore, an injection zone is not considered to be susceptible to any serious clogging effects, although it is possible that injectivity will decrease over time. Observing the restrictions that apply to the quality and composition of the injection fluid will reduce the chance of clogging and maintain the high injectivity essential to successful operation of the well.

Federal regulations require that a stimulation program be available in case it is needed. If the deep injection well becomes clogged or plugged and thus becomes less receptive to the injection fluid, it may be necessary to enact a well stimulation program. Unless the cause of the reduced intake is already known, the well operator might also be asked to conduct a special monitoring program. An interpretative report on the deep injection well must then be submitted, discussing the problem and reporting on the special monitoring program. The report should recommend corrective actions (including a stimulation program for the deep injection well) to rehabilitate the well.

A well stimulation program may include surging, acidizing, or a combination of both. Other stimulation techniques (shooting, hydraulic featuring, and vibratory explosions) are used primarily within the petroleum industry, are controversial, and are only recommended as a last resort.

Surging to stimulate the deep injection well involves use of a pump that can inject water into or discharge water from the well at continuously fluctuating rates. This action is designed to loosen any matter that might be clogging the rock openings. Well acidization is designed to chemically break down any matter clogging the rock pores, thereby improving flow movement through the injection zone. The acid used in this procedure should be 18°Baume (27.92 percent) hydrochloric (muriatic) acid. The acid should contain a nontoxic stabilizer to prevent after-precipitation of dissolved minerals and a nontoxic inhibitor to prevent corrosion of well head components. Use of the stabilizing and inhibiting chemical agents must first be discussed with the appropriate regulatory agencies.

The procedure used for acid treatment is subject to regulatory approval. The two most common methods are:

- *Method 1:* Full strength acid is introduced to the well by means of a 3/4- or 1-inch diameter drop pipe. The acid must not be introduced by pouring it from the top of the well casing. The drop pipe must be black iron or plastic pipe, not galvanized pipe. Overflows and vents required for safety must be provided. The acid is introduced to the well starting from the bottom of the open hole. The drop pipe is gradually retracted during filling operations to ensure that equal concentrations of acid are present in the entire length of open hole. The quantity of acid used should be 1-1/2 to 2 times the volume of water in the open hole section of the well. At no time during the acidizing process should the pH of the well contents be allowed to rise above 2.0. Additional acid is introduced to the well as required to ensure that the pH remains below 2.0 for the duration of treatment. After introducing the acid to the well, the acid should begin being agitated as soon as possible. Agitation can be accomplished by surge block bailers or by any other appropriate means available. The acid should be agitated for 3 to 4 hours. Following acid treatment, the well should be pumped clear of all sediment until the water is free of discoloration and the pH level returns to normal (between 6.0 and 8.0). Pumped out waste should be discharged to the plant.
- *Method 2:* If pressure acidizing is used instead, pressure control is provided by a sealing arrangement between the well casing and the acid drop pipe. The seal must be capable of maintaining a minimum operating pressure of 100 psig in the well. The sealing arrangement should include the pressure gauges and valves needed to accurately determine the gauge pressure in the well and to ensure that acid and air can be introduced into or vented from the well. Pressure should be maintained at 75 to 100 psig for a minimum of 2 hours. Following pressure acidization, the waste valve should be opened and the well pumped at a rate to produce substantial drawdown to clear the well of all sediment until the water is free of discoloration and the pH level returns to normal (between 6.0 and 8.0). Discharge should be to the plant. Following the initial acid treatment, a second acid treatment might be required to further increase the well yield.

The second method is generally used only if the clogging is so extensive that high pressures can be maintained in the injection zone during acid application.

The results of a stimulation program must be tested by a special monitoring program. Both the stimulation program and the special monitoring program should then be fully described and analyzed in an interpretative report submitted to regulatory agencies.

9.2.3.12 Costs for Concentrate Disposal

The estimated cost of deep well disposal (cost per gallon of membrane treatment plant capacity) is shown in Table 9-3. The actual cost of well construction is shown in Figure 9-6. Well construction costs depend primarily on well depth and diameter. Nevertheless, the injection liner material and the manner of liner installation can cause the cost of a typical well to vary by as much as $50,000.

Table 9-3. Concentrate Disposal Costs

Deep Well Disposal Facility (mgd)	Capital Cost ($/gallon)	Engineering Cost ($ per gallon)	Land Requirements (acre)	O&M Cost ($/1,000 gallons)
3	$0.58	$0.087	0.5	$0.032
5	$0.44	$0.066	0.5	$0.024
10	$0.40	$0.060	1.0	$0.022
15	$0.37	$0.056	2.0	$0.02
20	$0.30	$0.045	3.0	$0.016

In addition to the O&M costs shown in Table 9-3, a recurring cost is associated with the mechanical integrity test that must be conducted every 5 years. Contractor costs for this test usually range between $70,000 and $90,000; engineering fees add another $50,000 to $60,000. Permit renewal fees can also be high. In Florida, the fee for renewal of a Class I well permit is $10,000.

9.2.4 Boreholes

Drainfields and boreholes can be used with regulatory approval when the concentrate water quality meets the regulatory requirements for discharge into surficial ground-water aquifers and when soil conditions and

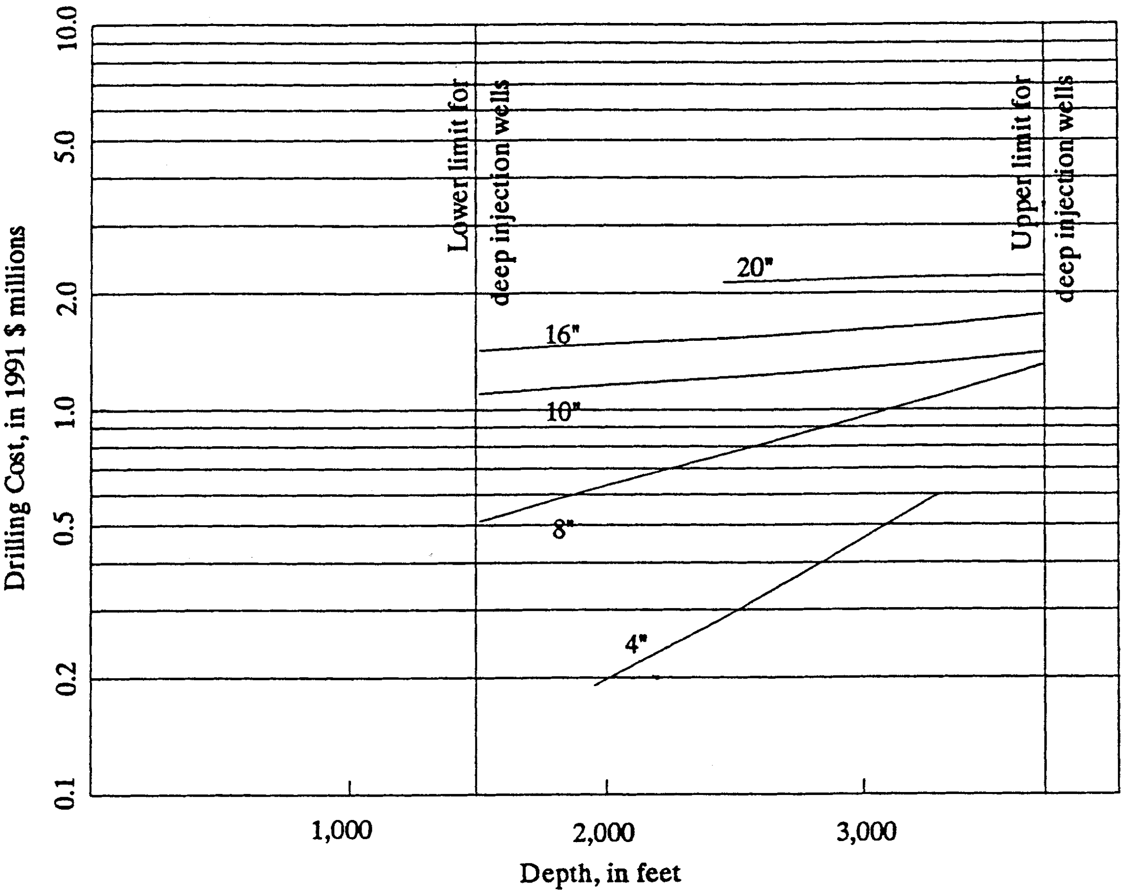


Figure 9-6. Well drilling cost.

permeability of rock strata permit. In Florida, this occurs in coastal areas where ground-water quality exceeds 10,000 mg/L TDS and USDW regulatory parameters are not exceeded. Surficial aquifer discharge of brine from coastal seawater RO plants that use shallow boreholes or seawells (where the ground water is very brackish) can be more cost-effective and environmentally palatable than an ocean outfall.

9.2.5 Spray Irrigation

In general, spray irrigation is viable only if the following conditions are met:

- The TDS, chloride content, or salinity of the concentrate or the mixture of concentrate and a blend liquid do not exceed a level that will damage the grass or crop being irrigated.
- There is a requirement for irrigation water in the vicinity of the WTP to avoid a long conveyance system.
- A backup disposal system (e.g., storage for use during sustained periods of rainfall, when irrigation is not needed) is available.
- The water quality of the surficial ground water is protected from degradation.
- Local, state, and federal regulations are met.
- A system of monitoring wells is in place to check overall irrigation system performance. A monitoring plan is generally a prerequisite to obtaining an operating permit.

If these conditions can be satisfied, spray irrigation can be an attractive option because it is economical, provides an appropriate source of irrigation water, and conserves natural resources.

Major variables affecting the cost of spray irrigation include land, concentrate conveyance systems to the disposal site, and site developments (e.g., relocation, land preparation, surface runoff control, subsurface drainage, distribution and irrigation, storage requirements, and pretreatment).

Spray irrigation of concentrates almost always requires dilution prior to irrigation to prevent pollution or degradation of ground-water resources while meeting regulatory requirements. Dilution also reduces the chloride content to acceptable levels to prevent damage to the grass or crops being irrigated.

Disposal of concentrates by spray irrigation is being proposed for several very large systems currently being designed. Recently, a sprayfield demonstration program in Boynton Beach, Florida, was funded jointly by the South Florida Water Management District and the City of Boynton Beach. The primary purpose of the demonstration program was to better manage water resources by using a product ordinarily disposed of as waste. Years after the permitting for the Boynton Beach concentrate disposal was finalized, deep well injection was chosen as the disposal method.

9.3 Nonconventional Methods of Concentrate Disposal

9.3.1 Evaporation and Crystallization of Brines for Zero Discharge

9.3.1.1 Brine Characteristics

Achieving a zero liquid discharge at any desalination facility almost always requires some type of evaporation and crystallization process. Evaporation is differentiated from crystallization only in that crystallization usually involves the development and processing of slurries, while evaporation refers mostly to liquid solutions. Techniques for handling high-density slurries are quite different from those for liquids, however, and are always more expensive per gallon of water evaporated. Figure 9-7 shows a basic zero discharge flow sheet.

Brines from desalination operations generally become wastewater because they cannot be concentrated further in membrane systems due to a high TDS content or the presence of scale-forming materials. Calcium sulfate and silica are the usual scaling components, although the barium and strontium sulfates can also contribute to scaling. Fortunately, techniques for dealing with scale-forming materials have been developed for use with both evaporators and crystallizers.

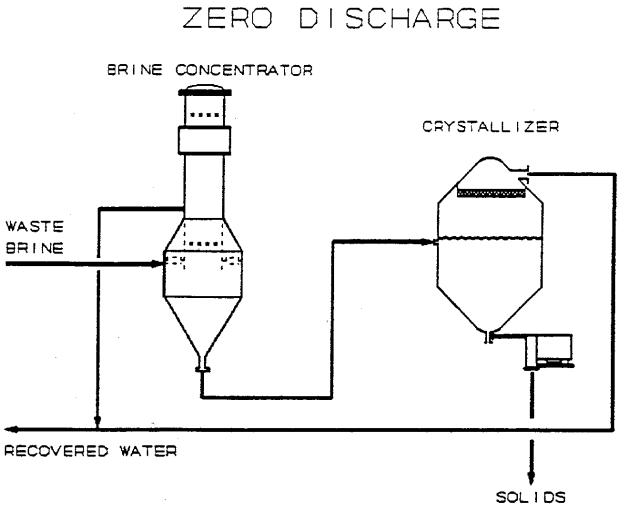


Figure 9-7. Zero discharge.

Control of scaling is generally accomplished through “seeding.” Both evaporators and crystallizers use the same fundamental approach, but the concentrations of solid particles differ greatly. In either case, sufficient crystals, or “seed slurry,” are maintained in suspension to provide enough crystalline surface area to effectively allow most or all of the precipitation to occur on existing crystals rather than on the heat exchange surfaces. This is a simplistic explanation, but it is correct in concept and will help guide the process designer in developing workable zero discharge systems.

Just as TDS limit the operation of membrane systems, they can limit the operation of evaporators and brine concentrators. Crystallizers can process streams with virtually any TDS level, even up to the point at which the circulating stream is more a melt than a solution. As an example, crystallizers are often used to concentrate sodium hydroxide to 50 percent by weight or higher (dissolved) while crystallizing sodium chloride from the liquor.

9.3.1.2 The Brine Concentrator

The workhorse of the zero discharge plant, the brine concentrator is most often designed as a vertical tube or falling film evaporator, although horizontal spray film and plate type evaporators are also used in this application. Various manufacturers provide brine concentrators; currently, most are used in the electric utility industry. These units concentrate cooling tower blowdown and other wastes to about 20 percent total solids, which are then processed further to a solid waste in either solar ponds or a crystallizer. The first brine concentrators entered commercial service at coal-fired utilities in the mid-1970s, and some of these units are still in service today.

The common characteristic of most brine concentrator designs is the circulation of a slurry of calcium sulfate crystals, which act as seeds. Calcium sulfate and other scale-forming compounds preferentially precipitate on the circulating seed crystals (rather than the heat transfer surfaces), preventing scaling. The development of the seeding technique for calcium sulfate and silica allowed brine concentrators to process wastewaters at or near saturation in calcium sulfate and silica to very high concentrations. Up to 20 percent total solids by weight is often achievable in the brine concentrator discharge, while a distillate of more than 10 parts per million (ppm) TDS can be returned to the power station for use as boiler makeup. Figure 9-8 shows the typical components of a brine concentrator. Because most brine concentrators are powered using a mechanical

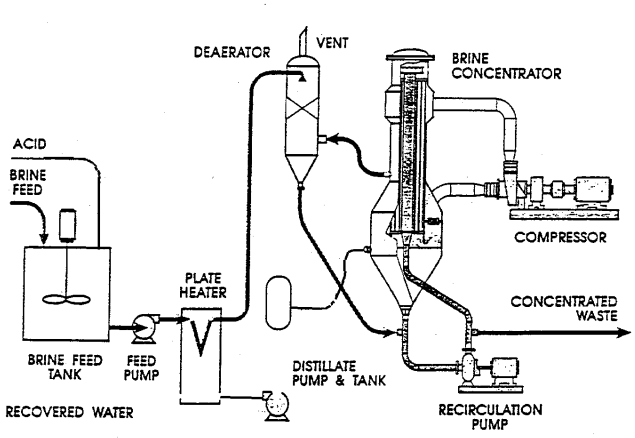


Figure 9-8. Brine concentrator system (HPD, 1994).

vapor recompression (MVR) cycle, the system shown in Figure 9-8 includes a compressor. Direct steam power could be used, but the cost of steam energy almost always exceeds that of MVR, except in special situations.

MVR evaporators or, more simply, vapor compression (VC), use a compressor to raise the condensing temperature and pressure of the evaporated water sufficiently to reuse the vapor as the primary heat source for evaporation. In this manner, the heat of vaporization in the evaporated water is recovered immediately in the evaporation of more process water. In a typical steam driven system, the heat of vaporization in the evaporated water is simply lost to the final condenser. At best, only a portion of the heat is recovered in a multiple-effect evaporation system.

In MVR evaporators, the compressor draws suction from the evaporating water in the vapor body of the evaporator. The condensing temperature and pressure are raised in the compressor 10° to 15°F above the boiling temperature in the evaporator. The now higher temperature vapor is then fed to the outside of the heat transfer tubes, where it gives up heat and condenses. More water evaporates inside the tubes, which is in turn recompressed, and so on. The steam is reused without first condensing it.

MVR systems can achieve very high thermal efficiencies compared to steam driven systems. MVR brine concentrators require evaporation energy that, if converted to steam, is equivalent to 1 pound of steam evaporating over 28 pounds of water; for example, only 35 British thermal units (BTUs) are required to evaporate 1 pound of water.[1] A single-effect evaporator, in contrast, requires 1 pound of steam per pound (or less) of water. In terms of energy consumption, MVR is at least 28 times more efficient than a single-effect steam driven evaporator.

Representative brine concentrator process conditions for zero discharge applications are listed in Table 9-4. These conditions are shown as ranges and should be used as guidelines. Many factors affect the operating limits of brine concentrators, but a few are important for planners of zero discharge systems.

Crystallizers are much more expensive to build and operate than evaporators or brine concentrators. Because of this great cost differential, operating a membrane system at its limits of concentration might become counterproductive in some instances. If the membrane system waste brine is too highly concentrated, a brine concentrator might be unusable (see Table 9-5), and a much larger crystallizer will be required to reach zero discharge. The upper limit for feed concentration into a brine concentrator is determined by the need to keep enough seed material in suspension to prevent scaling as well as by the solubility of the more soluble salts. This usually occurs with a feed TDS of somewhere between

Table 9-4. Typical Brine Concentrator Process Conditions in Zero Discharge Applications

Feed TDS	2,000–20,000 ppm
Feed temperature	40–120°F
Preheater approach temperature	7–15°F
Concentration factor	8–120
Seed slurry concentration	1–10%
Boiling point rise	1–10°F
Compression ratio	1.20–2.0
Recirculation pump rate	20–40x feed
TDS of waste brine, weight %	15–22%
Total solids waste brine	15–30%
Overall power/1,000 ga	l75–110 kWh
Distillate TDS	5–25 ppm

Table 9-5. Effect of Concentration Factor (CF) on Calcium Sulfate Seed Concentrations[a]

Ion/Species	Feed	5 CF	10 CF	20 CF	50 CF
Calcium	400	2,000	4,000	8,000	20,000
Sodium	1,100	5,500	11,000	22,000	55,000
Chloride	550	2,750	5,550	11,000	27,500
Sulfate	2,500	12,500	25,000	50,000	125,000
TDS	4,550	17,510	33,460	65,360	161,060
Total solids	4,550	22,750	45,500	91,000	227,500
Suspended solids (calcium sulfate)		5,240	12,040	25,640	66,440

[a] All values in ppm.

---

[1] During operation, 85 kilowatt-hours (kWh)/1,000 gallons (gal) evaporated is routinely obtained; 85 × kWh 3,418 BTU/kWh/(1,000 gal × 8.3 lb/gal) = 35 BTU/lb.

25,000 and 40,000 ppm, a range easily reached with membranes.

Boiling point elevation (BPE), which increases with concentration, also limits the economical concentration factor. The BPE is the difference in boiling temperature at constant pressure between pure water and a solution. In an MVR system, the compressor must first overcome the BPE before any evaporative energy can be recovered. Typical commercial systems operate with BPEs ranging from 1.0° to 4.0°F. Above about 5° to 7°F, energy requirements begin to rise significantly.

Seeding

A brine concentrator cannot concentrate into the precipitation region of soluble salts such as sodium sulfate, sodium chloride, or the mixed salt glauberite, $CaNa_2(SO_4)_2$. This upper limit of concentration is typically reached with brackish waters at the 20- to 25-percent TDS level. Because most brine concentrators must be seeded to prevent scaling, however, sufficient seed material must be maintained in suspension in the system. The seed material most often used is calcium sulfate because it is the primary scaling component of most brackish wastewaters and is, therefore, usually present.

If insufficient seed is maintained in suspension, scaling of the heat transfer surfaces promptly results due to the deposition of calcium sulfate, silica, or both. Scale removal is expensive and time-consuming, and generally cannot be tolerated on a regular basis. Operating experience has shown that calcium sulfate seed crystals can effectively seed for themselves, for other sulfates such as barium and strontium, and for silica. In most applications, the seed material is formed in situ from dissolved calcium sulfate present in the feedwater.[2]

To precipitate most of the calcium sulfate and maintain a useful seed concentration in the brine concentrator, the feedwater must be concentrated many times. As a practical minimum, the operating brine concentrator must maintain a seed suspended solids concentration of at least 1 to 1.5 percent by weight calcium sulfate or 10,000 to 15,000 ppm suspended solids. With a feed completely saturated in calcium sulfate, the minimum concentration factor that will result in enough seed material, even with a seed recovery system, is about eight concentrations, or 87.5 percent water recovery. Fifteen or more concentrations are preferred, but operation at eight concentration factors is possible if a seed recovery system is used.

The seed recovery system recoups seed material in the discharge from the brine concentrator and recycles the naturally produced seed material. The apparatus generally consists of a hydrocyclone bank or settling tank that recovers suspended solids from the waste brine blowdown and recycles them back into the system. For example, consider a feedwater with the characteristics shown in Table 9-5, where calcium sulfate is close to saturation. The data in Table 9-5 show the effect of concentrating the feedwater to various degrees on suspended solids and TDS. Even with a feedwater near saturation with calcium sulfate, 20 concentration factors (20 CF) are needed to reach 2.5 percent or 25,000 ppm seed concentration in the evaporator if no seed recycle is used.

This example illustrates why it is necessary to achieve the maximum cycles of concentration in the brine concentrator. Because the brine concentrator is practically limited to a maximum TDS of about 200,000 ppm and needs at least ten concentration factors to reach moderate seed concentrations, the upper feed TDS limit for a brine concentrator is in the range of 25,000 to 40,000 ppm TDS. For example, concentrating a feedwater of 25,000 ppm for eight cycles will result in total solids of 200,000 ppm, near the maximum. With a 35,000-ppm feedwater, eight cycles results in 280,000 ppm, beyond the limit of most brine concentrators. Yet eight cycles are required to develop sufficient seed material to prevent scaling without an external supply of seed. Thus, a feedwater with 35,000 ppm TDS may be a candidate for a crystallizer rather than a brine concentrator.

When silica is the primary scaling component, or when calcium sulfate is not near saturation, addition of exogenous calcium sulfate might be required. This can be accomplished by adding calcium chloride and sodium sulfate or sulfuric acid and lime. Conceivably, anhydrous calcium sulfate could be added directly to the evaporator, although this has not been done commercially.

Another reason to design brine concentrators at the maximum concentration factor obtainable is that sensible heat loss in the concentrated waste discharge stream becomes a consideration as recovery decreases. Waste is hot and filled with slurry, making conventional heat exchangers difficult to operate. The additional cost required to recover the heat in the waste is usually not worth the cost of providing a larger compressor and accepting the energy loss penalty. At a high discharge rate (e.g., a waste flow equal to 20 percent of the feed flow), however, the compressor can have difficulty making up the heat loss, and makeup steam might be required.

In summary, brine concentrator designs with greater than 87.5 percent recovery (8 concentration factors) can usually be made to function in the normal operating range. Below 87.5 percent recovery, special considerations are required to maintain both proper seeding and a balanced heat input.

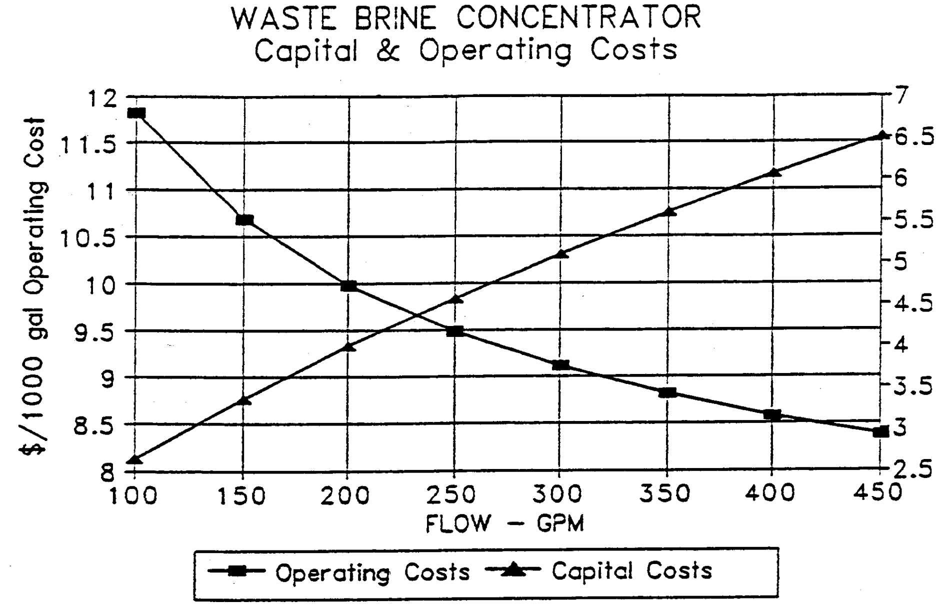
---

[2] At startup, of course, calcium sulfate is added directly to the brine concentrator to form the initial charge of seed material.

Design Considerations: Major Components

As shown in Figure 9-9, the main components of a brine concentrator are:

- *Distillate heat feed heat exchanger:* Usually a titanium plate type heat exchanger, this recovers heat from the hot distillate into the feedwater. Typical approach temperature for the feedwater is 10°F below the boiling point.
- *Deaerator:* Feedwater deaeration is required to prevent calcium carbonate scaling, to eliminate noncondensible gases that would interfere with heat transfer, and to remove dissolved oxygen for corrosion control. Ordinarily, the deaerator is a packed column (316L shell and internals, ceramic or 316L packing) with steam supplied from a hot distillate flash tank or a compressor bleed stream.
- *Main heater:* Wastewater is inside the tubes, and steam is outside. A falling film vertical tube unit is shown. Typically, 2-inch outside diameter tubes are used. Vertical lengths range from 30 to 55 ft. Tube material is usually titanium grade 2, with a 0.028- to 0.035-inch wall. Noncondensible venting occurs at approximately 1 to 3 percent of steam flow. Design heat transfer coefficients range from 400 to 600 BTU/hr/ft^2/°F for vertical tubes. Horizontal tube units are usually operated with the wastewater sprayed on the outside of the tubes. Typical tube diameter is 1 inch.
- *Vapor body:* The main vessel holding the circulating waste water also provides disengagement of steam from the liquid. The vapor body can be integral with the heater or a separate vessel. Typical volumes based on feed retention time range from 30 to 60 minutes of feed flow. The vapor disengagement area usually sets the minimum diameter of the vapor body. Materials used depend on the equilibrium concentrations of chloride and magnesium; 316L and 317L have been used with success in relatively low chloride applications with very good deaeration. Inconel 625 and SMO 254 or equivalent alloys have seen more use since the late 1980s.
- *Demisters/Entrainment separators:* Mesh pads or chevron-style entrainment separators are used to clean the vapor stream of entrained droplets of very salty water. Correct design is very important in MVR applications to prevent corrosion and buildup in the compressor. Entrainment separators can be external to the vapor body and used with vapor scrubbers if very high purity distillate is required. Chevrons and mesh pads are usually fabricated in the same material as the vapor body.
- *Recirculation pump:* Circulation of the concentrated waste stream through the tubes is required. Typical circulation rates in brine concentrators are 20 to 40 times the evaporation rate.
- *Compressors:* With low BPEs (e.g., below 4° to 5°F), single-stage centrifugal compressors are widely used. Typical compression ratios for single-stage units are 1.2 to 1.4. For higher BPEs, lobe-type compressors or multiple-stage centrifugals are needed. Compressors as large as 5,000 horsepower have been used in MVR installations. Guidevanes or variable speed motors are required to provide efficient operation over a wide range of inlet flow rates. Com-



Power: $0.045/kwh
Operating Labor: $20.00/hr
Capital Recovery: 10%/yr
Water Recovery: 98%
Power: 85 kwh/1000 gal
H2SO4: 75 ppm

Figure 9-9. Brine concentrator capital and operating costs.

pressor wheels are fabricated in stainless steel or cast iron. Housings are typically cast iron. Good entrainment separation is critical to prevent corrosion and/or wheel buildup.

- *pH control:* pH adjustment is usually needed to convert carbonate and bicarbonate in the feedwater to carbon dioxide before removal in the deaerator.
- *Scale control:* Scale control compounds are injected just before the feedwater preheater. This delays the precipitation of calcium sulfate at the higher temperatures found in the preheater and the deaerator.
- *Startup boiler:* In an MVR system, it is necessary to raise the circulating water temperature to near the boiling point before engaging the compressor. The required startup steam is furnished by the plant or by a small electrically fired startup boiler. For example, with a 200-gallon per minute system, a 200 kw startup boiler could bring the system to operating temperature in 10 to 15 hours.

Design Considerations: Process

For the desalting engineer, the most important process consideration is a system design that delivers feedwater within the known operating constraints of the brine concentrator system. This typically requires:

- A calcium sulfate concentration greater than 50 percent of saturation in feedwater. This assures that enough will be available to produce seed crystals. With less than 50 percent of saturation (about 750 ppm calcium sulfate) and less than 15 concentration factors, external seed supplies may be necessary.
- A TDS level low enough to permit high concentration factors in the brine concentrator without precipitation of soluble salts. Eight to 10 concentration factors are the minimum required for normal operation.

Volatile components of the feedwater will distill over in the brine concentrator and contaminate the recovered water. Typical volatile components encountered include:

- *Ammonia:* Carryover is a function of pH, but even at pH 6 to 7, ammonia can be found in the distillate.
- *Boron/Boric acid:* Boron carries over less than ammonia and is less sensitive to pH conditions.
- *Organic acids:* These are highly sensitive to pH and species. Higher pH results in less carryover as the ionized forms dominate.
- *Trihalomethanes, etc.:* The degree of carryover must be determined by laboratory or pilot testing.
- *Hydrogen sulfide:* Carryover is a function of pH. Oxidation to sulfate might be necessary.

Capital and Operating Costs

The graphs shown in Figures 9-9 and 9-10 provide a rough estimate of the capital and operating costs of typical brine concentrator systems. For smaller systems below 100-gallons per minute capacity, the equipment is generally skidded. Larger systems require some field fabrication of the vapor bodies and assembly of the structural steel. The costs shown in the graphs are based on installed outdoor systems with foundations. Control rooms are assumed to be located within the central plant facilities.

9.3.1.3 Waste Crystallizers

Crystallization technology for zero discharge grew out of the chemical process industries. It was extended to the nuclear power industry with the development of mixed salt waste crystallizers for ion exchange regeneration wastes. There it was necessary to produce a nearly dry solid from the low-level wastes. The forced circulation crystallizer has now become the primary tool for production of dry or nearly dry waste salts for zero discharge.

A basic waste crystallizer flow scheme is shown in Figure 9-11. In this illustration, an MVR cycle is shown, but steam can be used instead. Figure 9-12 shows a simplified steam system with a barometric condenser. Because the crystallizer circulates a slurry, some method of dewatering is required. The drawings show a centrifuge, but a filter press, rotary filter, or belt filter could also be used.

Crystallizers do not have the same feed supply limitations as brine concentrators; streams with widely varying composition can be effectively processed. Because the crystallizer does not generate a liquid blowdown, concentration factor is not the appropriate term to describe system operation. The crystallizer simply evaporates water, leaving a circulating crystal slurry. The slurry is dewatered at a rate that maintains a slurry concentration in the crystallizer of about 25 percent suspended solids by weight. The TDS level eventually reaches an equilibrium point, determined largely by the exact composition of the feed stream.

Residual water is entrained in the dewatered crystals, but the amount is small compared to the inlet flow. Centrifuges produce solids with 10 to 20 percent entrained liquid, while filters typically leave 40 to 50 percent entrained liquid with the solids. The entrained liquid can be as high as 50 percent TDS; thus, in many applications, the hot crystals can absorb all the residual water into water of hydration, leaving a dry-to-the-touch solid at room temperature. For example, sodium sulfate exists as the anhydrous form at typical atmospheric boiling temperatures, yet forms the decahydrate (Na_2SO_4-$10H_2O$) at room temperature, absorbing more than its weight in water.

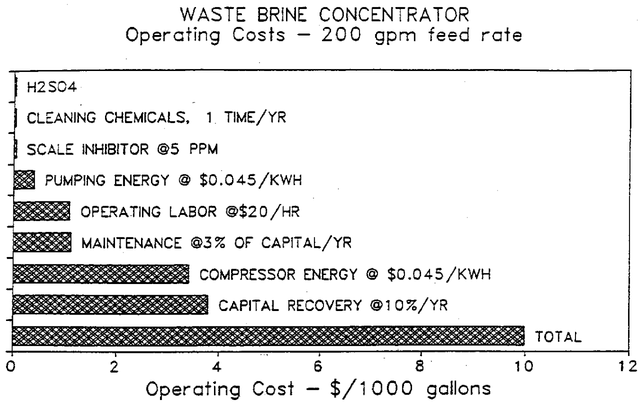


Figure 9-10. Brine concentrator cost components.

Table 9-6. Typical Brine Crystallizer Process Conditions

Feed TDS	>20,000 ppm
Feed temperature	40–220°F
Concentration factor	N/A
Boiling point rise	1–50°F
Compression ratio (MVR)	2.0–3.0
Recirculation pumping rate	>200x feed
Solids, % free liquid hot	15–50%
Solids, % free liquid cool	0–10%
kWh/1,000 gal (MVR)	150–200 kWh
BTU/1,000 gal (steam)	10 mm BTU
kWh/1,000 gal (steam)	2–4 kWh
Cooling water (steam)	50x feed
Distillate TDS	15–50 ppm

Design Considerations: Process

Typical operating conditions for a forced-circulation waste crystallizer are shown in Table 9-6. With regard to the feed stream, all of the dissolved and suspended material will remain in the solids. The overall system concentration factor for the desalting process, brine concentrator, and crystallizer can be very high. In some applications, it may be possible to concentrate relatively low levels of metals or other components in the feed to levels that create a concern. As in a brine concentrator, volatile materials will transfer to the distillate.

Steam is generally required if the waste solution has a high percentage of magnesium chloride or nitrates. These compounds are highly soluble and exhibit high BPEs. At equilibrium, the solution boiling point might well exceed the compression ratio available in typical MVR systems. Steam at 25 to 30 psig or higher will be needed. A condensing water supply equal to about 40 to 50 times the crystallizer feed rate will also be required. In many cases, it should be possible to use the brine concentrator and the membrane system product water to condense the crystallizer vapors in a direct contact or barometric condenser.

By necessity, the MVR waste crystallizer will require a high lift compressor. A multistage or positive displacement compressor will also be required. Above a compression ratio of about 2.0 to 3.0, steam begins to be favored from an economic viewpoint in almost all cases.

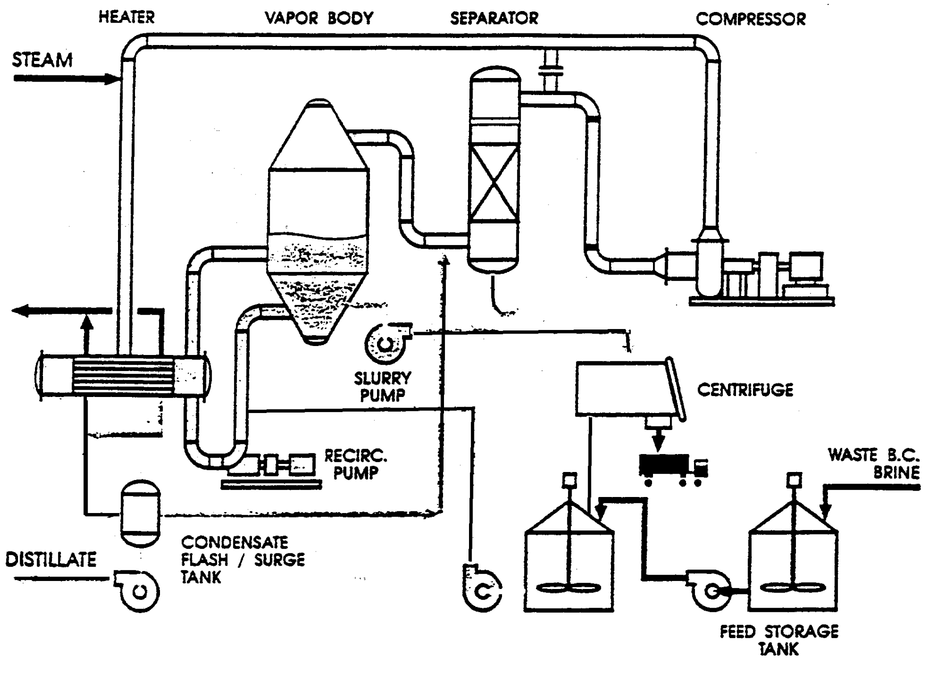


Figure 9-11. Waste crystallizer system (HPD, 1994).

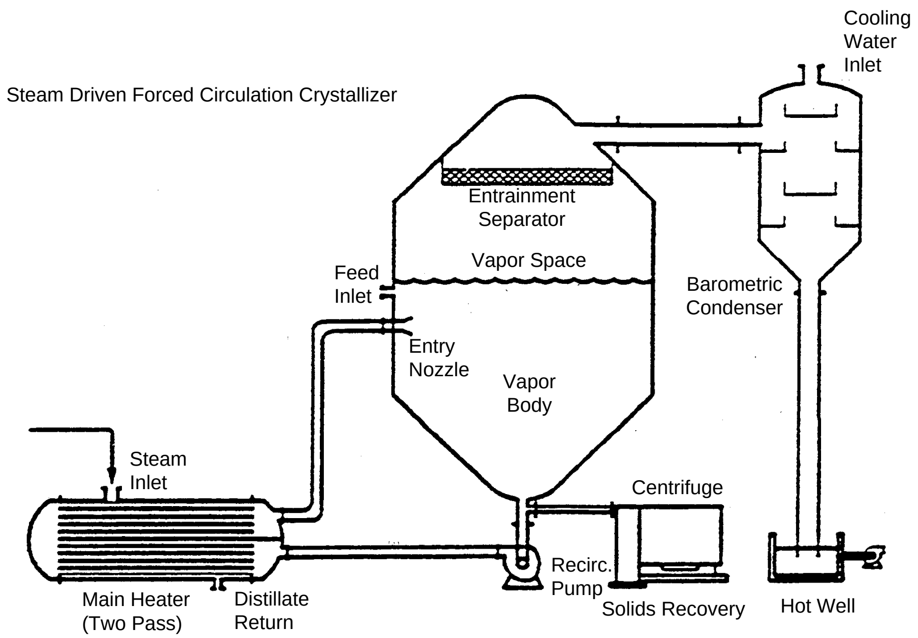


Figure 9-12. Steam driven circulation crystallizer.

Because the waste crystallizer usually has a relatively small capacity, steam might be preferred for simplicity of control and maintenance, even in cases with low BPEs.

For large capacity systems (100 gallons per minute or greater), the designer or specifier can consider the use of multiple-effect systems to lower steam usage. Multiple-effect systems use less steam than single-effect units, but have higher capital costs.

Separating solids from the crystallizer circulating liquor can be difficult. Centrifuges are almost always effective, but they are expensive to buy and maintain. In order of solids dewatering effectiveness and cost, options for solids removal include:

- Centrifuge (highest cost, lowest residual water).
- Belt filter (automatic operation, 40 to 50 percent retained liquid).
- Filter press (manual or semiautomatic, 40 to 60 percent retained liquid).
- Hydrocyclone (lowest cost, no moving parts, 50 to 70 percent retained liquid).

If solids meeting the “no free water rule” are required, testing of the filter options and hydrocyclone might be needed to demonstrate sufficient absorption of the free water.

Design Considerations: Major Components

Vapor release and entrainment separation determine the minimum size of the vapor body. A minimum of 6 feet of straight side above the liquid level is required for proper disengagement. Maximum vapor release velocities can be estimated using the following equation:

$$\text{Velocity (ft/sec)} = k[(D_l - D_v)/D_v]^{0.5}$$

where
D_l = density of the liquid in lb/ft^3
D_v = density of the vapor in lb/ft^3
k = constant ranging from 0.10 to 0.04 (dimensionless)

The minimum diameter can easily be calculated. For example, a 50-gallon per minute crystallizer operating at atmospheric pressure with a 20-degree BPE will require a minimum diameter of 10.3 ft with k set at 0.05. Vapor body materials must be suitable for harsh service. Inconel 625 is used for rad waste units, while titanium, Inconel 825, and SMO254 or FRP units have been used in commercial service.

The recirculation pump must overcome the heater pressure loss and the lift to the surface in the vapor body. The vapor body must be located enough above the heat exchanger to suppress boiling in the heat exchanger and to provide sufficient NPSH. Pump heads, in net TDH, typically run at about 15 to 25 feet. The circulating liquor specific gravity is greater than 1.2. Axial flow pumps are usually specified, but low (lower than 1,100) revolutions-per-minute centrifugal pumps can also be used.

Cooling water or air cooling can be used in the condenser (which is not required for MVR). The cooling requirement, in BTUs/hr, is approximately equal to the steam BTUs/hr. Direct contact condensation using the

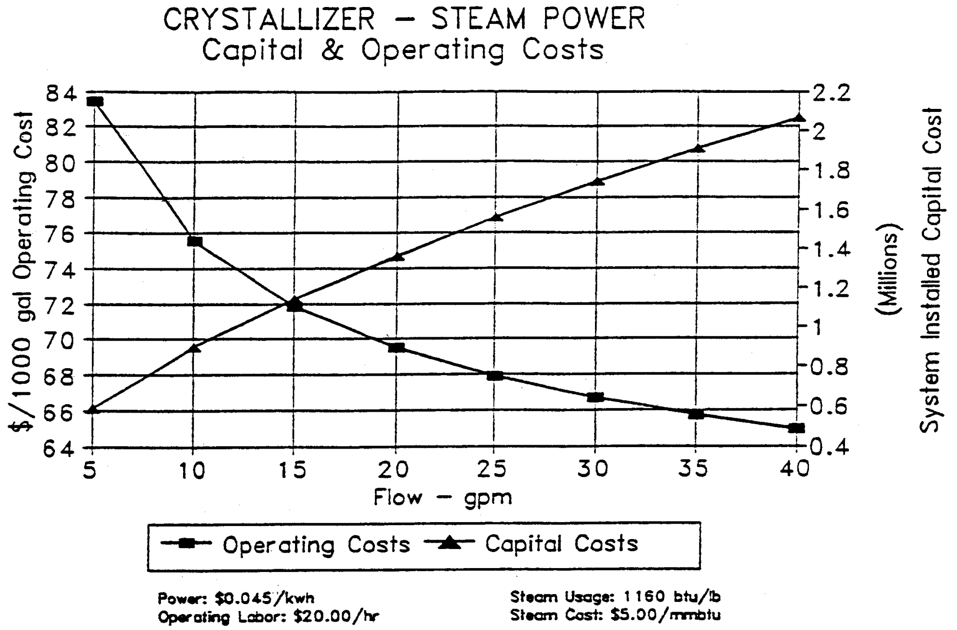


Figure 9-13. Steam power crystallizer capital and operating costs.

desalted product can be considered as well. Tubes are usually 304 or 316 ss.

Tubes in shell units are typically used for the heat exchanger. Its orientation can be vertical or horizontal. Boiling inside the tubes should be suppressed with sufficient head above the exchanger and tube velocities must be sufficient to avoid settling of the solids; 6 to 8 ft/sec is typical. Temperature rise per pass through the heat exchanger is kept low to avoid violent flashing in the vapor body. Titanium tubes are standard. Differential temperatures (steam to liquor) of 20°F can be used. Heat transfer coefficients are a function of temperature and concentration. For preliminary sizing at atmospheric boiling, an overall coefficient of 400 BTU/°F/ft^2 can be used.

Capital and Operating Costs

Figures 9-13 to 9-16 provide ways of estimating the capital and operating costs of crystallizers. The costs are based on single unit systems at 100 percent of design capacity. Single-effect steam driven systems are assumed. The values provided range roughly an order of magnitude for installed systems.

9.3.2 Evaporation Ponds

The suitability of evaporation ponds for concentrate disposal depends on ideal climatological conditions. The principal factors that affect the evaporation rate are relative humidity, wind velocity, barometric pressure, air and water temperature, and the salinity of the water.

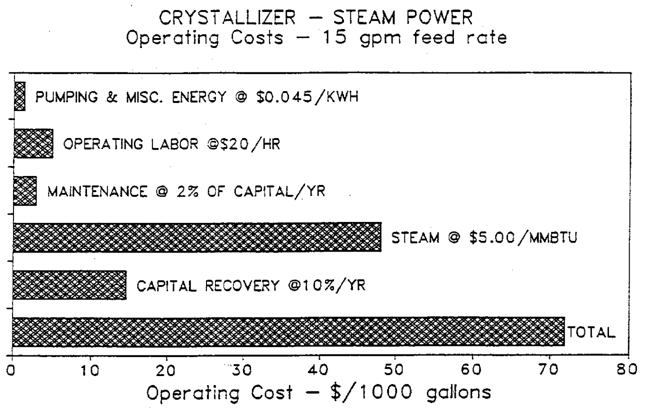


Figure 9-14. Steam power crystallizer cost components.

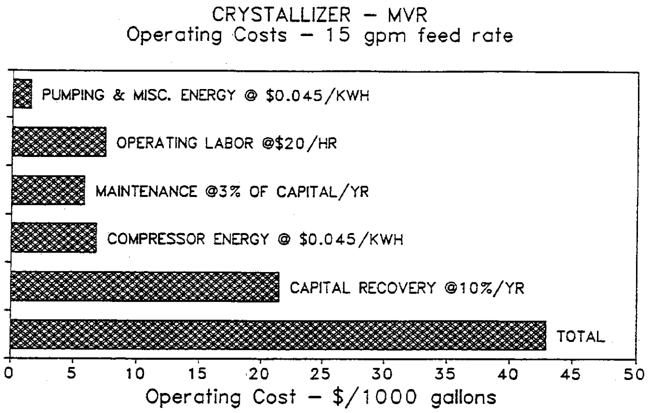


Figure 9-15. MVR crystallizer capital and operating costs.

The need for dry, arid conditions where evaporation losses are much greater than the amount of rainfall limits use of evaporation ponds to just a few areas in the United States. In addition, the amount of land required is an economic consideration. Because evaporation losses are directly proportional to area, large areas of land must be available for this option to be viable. A backup system is necessary, since an unusual weather event could affect the evaporation rate. Some large brackish water RO plants in the Middle East have employed this method of concentrate disposal successfully or have used it in combination with percolation onto the desert.

9.3.3 Emerging Technologies

Currently, no identifiable new technologies are predicted to be used in disposing of brines. Refinements, increased efficiencies, and combinations of existing disposal technologies might lower the cost of disposal, but no new options are anticipated.

9.4 Costs Associated With Brine Waste Disposal

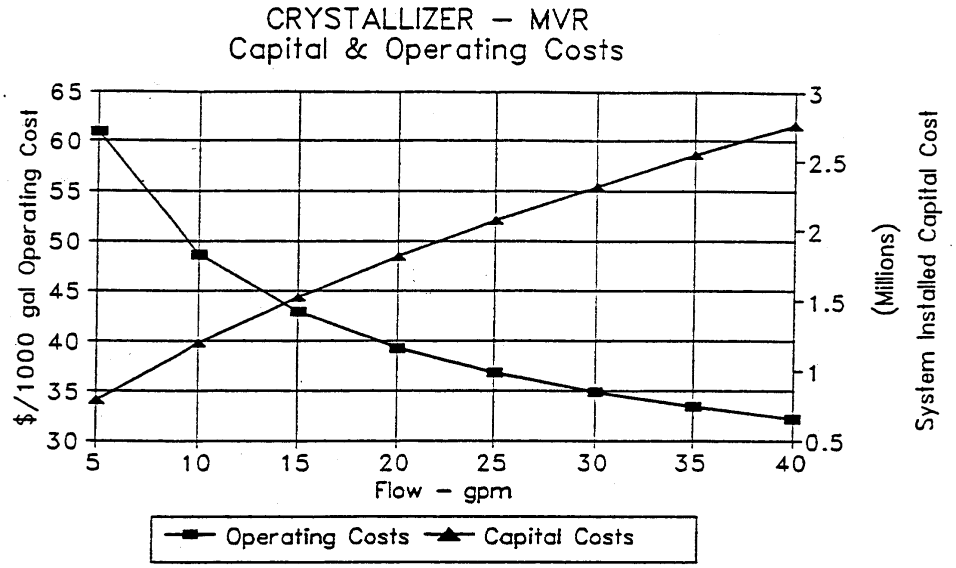
Capital and O&M costs for concentrate disposal will be specific to a given site. Cost variables include:

- Quality of concentrate water.
- Quantity of concentrate water.
- Disposal method.
- Distance from disposal point (and the length of the conveyance system).
- Pretreatment required, if any.
- Regulatory requirements for method of disposal.
- Permitting.
- Use of energy recovery.
- Requirement for backup or redundant disposal method.
- Climatological conditions.
- Residual pressure of the concentrate stream exiting the membrane process plant.

9.5 Conclusion

Concentrates from membrane process plants have been disposed of for over two decades without incident. Membrane processes are important treatment technologies that will assist the water treatment industry in meeting present and future drinking water regulations. Membrane technologies probably will be the best available technology in terms of the most organic and inorganic contaminant removal for the amount of capital invested. Safe methods of concentrate disposal as well as fair and pragmatic regulations will be necessary for membrane processes to continue. It is the responsibility of all those involved in the membrane process industry to work together to:

- Prevent damage to the environment that could occur from improper disposal of concentrate wastes.
- Ensure that proper engineering judgment is used in interpreting agency rules and regulations.
- Guarantee that future regulations do not unduly limit the use of membrane process technology.
- Secure new methods for concentrate disposal through research and development.



Power: $0.045/kwh
Operating Labor: $20.00/hr

MVR Compressor: 150 kwh/1000 gal feed
Capital Recovery: 10%/yr

Figure 9-16. MVR crystallizer cost components.

Chapter 10
Radioactive Waste Disposal

Nearly all of the radionuclides found in drinking water supplies are naturally occurring and are members of one of three natural radioactive decay series: the uranium series, the thorium series, and the actinium series. In 1976, the U.S. Environmental Protection Agency (EPA) promulgated interim drinking water regulations for radionuclides that established limits for gross alpha activity, gross beta and photon activity, and the specific elements of radium-226 (Ra-226) and radium-228 (Ra-228) (40 CFR Part 141).

On July 18, 1991, the Agency proposed new regulations that would increase radium maximum contaminant levels (MCLs) and add limits for uranium and radon (40 CFR Parts 141 and 142). The final regulation was scheduled to be promulgated April 30, 1995; as of this printing, however, this promulgation date is postponed until an undetermined time after December 1995. EPA is considering separating the other radionuclides from radon because regulation of radon is under a congressional freeze.

10.1 Background

A variety of treatment processes exist to remove radioactive contaminants from drinking water, such as conventional coagulation/filtration, ion exchange, lime softening, reverse osmosis, and granular activated carbon (GAC) adsorption (U.S. EPA, 1976b; Brinck et al., 1978; U.S. EPA, 1977; Lassovszky and Hathaway, 1983; Sorg et al., 1980; and Lowry and Brandow, 1985). These processes separate the contaminants from drinking water and concentrate them in the waste streams. Because these processes are also commonly used to remove other nonradioactive contaminants, they could potentially concentrate significant levels of radioactivity in waste treatment residuals even if the treatment was not originally designed or intended to remove radioactivity.

In addition to the waste streams containing the concentrated contaminants, some materials used to treat drinking water adsorb radioactive contaminants and permanently retain them. One good example is sand used in filtration processes. Sand adsorbs and retains radium on its surfaces. Although the radium does not interfere with the treatment process, at the end of its useful life the sand must be replaced and disposed of.

The handling and disposal of the water treatment plant (WTP) residuals containing naturally occurring radionuclides pose significant concerns to water suppliers, local and state governments, and the public. The unresolved question is do these wastes require special handling and disposal, or can they be disposed of by the same methods used to dispose of conventional, nonradioactive wastes? Presently, the federal government does not regulate the disposal of naturally occurring radioactive materials (NORM) waste from drinking water treatment processes. The U.S. Nuclear Regulatory Commission (NRC) has the authority to regulate the handling and disposal of all licensed radioactive materials, but this does not include NORM in drinking water. Licensed materials include high-level radioactive wastes, such as spent nuclear reactor fuel rods and wastes from nuclear weapons processing centers and low-level radioactive wastes such as those used in building materials, machinery, and clothing from nuclear power plants, as well as spent nuclear medicines.

Although federal agencies do not classify NORM wastes as radioactive wastes, policies could change, and definitions to characterize these wastes as radioactive could be developed in the future. Because of current concern for the proper disposal of these wastes, in July 1990 EPA's Office of Drinking Water published *Suggested Guidelines for the Disposal of Drinking Water Treatment Wastes Containing Naturally Occurring Radionuclides* (U.S. EPA, 1990c; Parrotta, 1991). These guidelines were intended to help WTPs and state and local regulators achieve safe and responsible waste management practices for water treatment residuals containing radionuclides at concentrations in excess of background levels.

The EPA guidelines clearly state that no federal regulations specifically address the disposal of wastes concentrated by water treatment processes on the basis of their naturally occurring radioactive content. The document further states that the guidelines developed were taken from a review of the regulations and from guidelines originating in other industries and programs. For example, one source of information for the suggested guidelines for radium wastes was disposal criteria for WTP residuals that the states of Illinois and Wisconsin (Illinois, 1984; Wisconsin, 1985) developed. Some states have also established rules for the disposal of wastes that contain radioactivity. EPA emphasizes in its guidelines, therefore, that where radioactivity in water treatment residuals is anticipated, state agencies should always be consulted.

A review of the current literature shows that almost all information on the disposal of WTP residuals containing radionuclides comes from the EPA guidance document. The guidelines are being revised, but no major changes are anticipated. Until formal regulations are developed, states having no specific rules or regulations will likely cite and use this document as general policy. Consequently, the sections on waste disposal in this chapter have been summarized from EPA's revised 1994 draft guideline document (U.S. EPA, 1994b). The revised guidelines will not be issued to the public until the regulations are promulgated, some time after December 1995.

10.2 Waste Disposal Practices

A list of residuals generated from common water treatment practices appears in Table 10-1. Except for disposal at a radioactive waste disposal site, current disposal methods are basically the same as those used for nonradioactive wastes. These methods are shown in Table 10-2.

Federal, state, and local regulations exist that govern and limit the discharge of nonradioactive WTP residuals into the environment. These regulations also apply to residuals containing radionuclides. The radioactivity of these residuals causes additional concern with disposal and thus may narrow disposal options or place additional requirements on the options. In selecting the disposal option, the concentration of radioactive contaminants in the residuals is the governing factor. Unfortunately, no current federally established levels of radionuclides have been developed to define low or high radioactive wastes or dictate the acceptable disposal method.

Table 10-1. Summary of Residuals Produced From Water Treatment Processes

Water Treatment Process	Waste Streams/Material
Conventional coagulation/filtration	Filter backwash water Sludge (alum or iron) Filter material
Lime softening	Filter backwash water Sludge (lime) Filter material
Ion exchange	Reuse and backwash water Regeneration liquid (brine, caustic, acid) Resin
Reverse osmosis/electrodialysis	Reject water Membrane/Material
Greensand filtration	Filter backwash water Sludge Greensand media
Selective sorbents (GAC, resins activated alumina)	Sorbent media

Table 10-2. Water Treatment Methods for Residuals Containing Radionuclides

Residuals Type	Disposal Method
Liquids	Direct discharge to surface water
	Direct discharge to sanitary sewer
	Deep well injection
	Irrigation
	Lagooning/Evaporation ponds
Sludge	Lagooning (temporary)
	Landfill disposal • No pretreatment • With prior lagooning • With mechanical dewatering
	Land disposal
	Licensed low-level radioactive waste disposal site
Solids	Landfill disposal
	Licensed low-level radioactive waste disposal facility

10.3 Waste Disposal Guidelines

The following information on disposal of liquids, solids, and sludges has been summarized from EPA's revised 1994 draft guideline document (U.S. EPA, 1994b).

10.3.1 Liquid Disposal

10.3.1.1 Discharge to Surface Waters

The Clean Water Act requires that dischargers of pollutants to navigable waters obtain National Pollutant Discharge Elimination System (NPDES) permits containing, at a minimum, technology-based effluent limitations that reflect various levels of wastewater treatment and, where necessary, more stringent limitations necessary to ensure attainment and maintenance of state water quality standards. EPA has not promulgated any rule establishing technology-based effluent limitations applicable to WTPs nationwide.

In the absence of such a categorical standard, limitations are established on a case-by-case basis, using the best professional judgment. After determining minimum technology-based requirements, the effect(s) of discharge on the receiving water is also determined; if necessary, more stringent limitations are established to protect the receiving water quality (see Chapter 5). In certain states, water quality standards may include specific criteria for radionuclides. State water quality standards that require more stringent discharge limitations must be reflected in any NPDES permit.

The NPDES permitting process should include study of the flow and geometry of receiving storm sewers and surface waters, as well as the potential uses of the surface waters (e.g., drinking water, agriculture). Based on site conditions, a state or other regulatory agency may use its discretion in determining a limiting concentration (e.g., a drinking water MCL) or a percentage of the background concentration(s) of radionuclides (e.g., 10 percent) that would limit the increase of radionuclides in the water body and/or the sediments due to the discharge of WTP residuals.

The NRC has established levels of radionuclides that its licensees are allowed to release into unrestricted areas of the environment (10 CFR Section 20.1302(b)(2)(i) and Sections 20.1001-20.2401). The referenced NRC limits are: 60 pCi/L for Ra-226 and Ra-228 and 300 pCi/L for natural uranium. If no state or local standards are in place, the NRC standards may be used as guidelines for surface water discharge. Some states and local authorities have promulgated conservative limits of 10 percent of the NRC levels under 10 CFR 20 for release of radionuclides into the environment.

If the conditions of flow and geometry are not adequate to prevent a buildup of radionuclides in surface water or sediments to within the limit set by the regulator, then other solutions need to be studied. These may include additional waste treatment, waste storage, and controlled discharge measures to produce a waste stream that can meet in-stream requirements. Otherwise, discharge to surface waters should not be allowed, and other options need to be considered.

10.3.1.2 Discharge Into Sanitary Sewers

Federal regulations prohibit discharges to sewers that would cause a municipal waste water treatment plant (WWTP) to violate an NPDES permit or that would interfere with wastewater treatment operations or sludge disposal. In addition, states or localities may establish more stringent limitations on the discharge of wastes from WTPs into a sanitary sewer, or may require pretreatment of the waste prior to its release into the sanitary sewer. State or local regulations limiting the discharge of WTP residuals that contain naturally occurring radionuclides into sanitary sewers would govern those discharges.

The NRC limits the discharge by its licensees of wastes containing radioactive materials into sanitary sewers. For NRC licensees, the monthly quantity of soluble Ra-226, Ra-228, and natural uranium, diluted by the average monthly quantity of total WTP residuals released into a sewer, should not exceed 600 pCi/L, 600 pCi/L, and 3,000 pCi/L, respectively. Also, the gross quantity of all radioactive material (excluding tritium and carbon-14) that a facility releases into sanitary sewer should not exceed 1 curie per year, according to the same NRC standards. EPA puts forth these standards for consideration in this context.

The NRC standards correspond to an individual radiation dose of 50 millirem per year. Although high, the reasonable dilution that a receiving waterway provides may reduce this dose to levels below any applicable drinking water standards. This should be examined, however, on a case-by-case basis.

If accumulation of radioactivity in the sanitary sewer distribution system, sewage treatment facility or publicly owned treatment works (POTW) is observed, discharging radioactive residuals into a sanitary sewer should be discontinued until radiation exposures and possible hazards to personnel repairing sewage pipelines are evaluated. Discharge of residuals containing radionuclides into a sanitary sewer results in the accumulation of radionuclides in the sludge that the POTW produces. Affected sewage sludge should be monitored for high levels of radioactivity and disposed of properly.

10.3.1.3 Deep Well Injection

Under Part C of the Safe Drinking Water Act (SDWA), EPA is required to promulgate minimum requirements for effective underground injection control (UIC) programs to prevent endangering underground sources of drinking water by subsurface emplacement of fluids through wells. Regulations may be implemented by states that have adopted requirements at least as stringent as the federal requirements and have been given primary enforcement responsibility for the UIC program. For states that do not have primacy, EPA has promulgated state-specific regulations that EPA regional offices implement.

A WTP interested in discharging residuals containing radionuclides into an injection well in a primacy state should first consult with the appropriate state agency. State regulations may be more stringent than federal requirements and may actually ban such a practice. The WTPs in other states should consult with the appropriate UIC regional branch office of EPA before deciding to dispose of residuals containing radionuclides into an injection well in accordance with a method recommended below.

Under the federal requirements, regulation of WTP residuals containing radionuclides depends on the concentrations of radionuclides present. Furthermore, requirements are specified for shallow and for deep well injection. Shallow wells are defined as those above or in an underground source of drinking water (USDW).

USDWs and injection wells are defined very broadly. A USDW is an aquifer or its portion that 1) supplies any public water system or that contains sufficient quantity of water to supply a public water system and 2) supplies drinking water for human consumption, or contains fewer than 10,000 mg/L TDS. A well is any bored, drilled, or driven hole where the depth of the hole is greater than the largest surface dimension. This definition includes septic systems and cesspools used for disposal of wastes.

Radioactive wastes are treated differently from nonradioactive wastes under the UIC program. As defined in 40 CFR Section 144.3, radioactive waste means any waste that contains radioactive concentrations that exceed those listed in 10 CFR 20, Appendix B, Table 11, Column 2. The concentration for Ra-226 and Ra-228 is currently listed as 60 pCi/L, while the radioactive concentration for natural uranium is 300 pCi/L.

Radioactive wastes as defined by the UIC program (i.e., wastes containing greater than 60 pCi/L Ra-226 or Ra-228, and/or containing greater than 300 pCi/L uranium) would not be disposed of in a shallow well, as defined above. Shallow injection of radioactive wastes (e.g., injection above or into a USDW) is a banned practice under the UIC program. The definitions of USDW and shallow injection would virtually eliminate any shallow disposal of radioactive waste.

Well disposal of radioactive waste below a USDW is currently considered a Class V well injection and is under study by EPA as part of the Class V regulatory development effort. At this time, EPA is not prepared to make any recommendations regarding these wells. The following is suggested guidance for nonradioactive wastes as defined by the UIC program (i.e., wastes containing less than 60 pCi/L Ra-226 or Ra-228, or less than less than 300 pCi/L uranium).

Well injection of nonradioactive WTP residuals beneath the lowest USDW is classified as a Class I nonhazardous practice, provided the waste contains no other hazardous components. EPA recommends disposal of nonradioactive waste through a Class I well because these wells must be permitted, and current permitting requirements for these wells ensure adequate protection of USDWs and human health.

10.3.1.4 Other Options

If, due to the properties of a residual, or local regulatory restrictions, a residual containing NORM cannot be processed using one of the above methods, then the treatment operator may choose from another treatment or disposal option such as evaporation of liquid wastes, sand drying or lagooning, chemical precipitation of contaminants, or other solids separation techniques. Other state regulations may apply to these practices. At a minimum, no practice should be less environmentally protective than the options mentioned above.

For instance, lagooning of radioactive wastes would be analogous to shallow well injection, if practiced in an unlined unit. Lagoons or other impoundments should, at a minimum, be lined to prevent infiltration. If WTPs intend to evaporate radioactive waste, the evaporation unit should be designed and operated properly to ensure isolation of the waste from the watertable.

10.3.2 Solids and Sludge Disposal

The following disposal guidance for radioactive WTP residuals is summarized from the EPA guidelines. The guidance is based solely on the radioactivity of these wastes and does not address other potentially hazardous substances.

10.3.2.1 Landfill Disposal

Wastes containing less than 3 pCi/g (dry weight) of radium and less than 50 g/g of uranium may be disposed of in a municipal landfill without the need for long-term institutional controls if the wastes are first dewatered and then spread and mixed with other materials when they are emplaced. The total contribution of radioactive wastes to the landfill should constitute only a small fraction (less than 10 percent of the volume) of the material in the landfill.

Wastes containing 3 to 50 pCi/g (dry weight) of radium should be disposed of using a physical barrier (i.e., a cover) that protects against radon release and isolates the wastes. Institutional controls designed to avoid inappropriate uses of the disposal site should also be provided. A physical barrier consisting of 10 feet of cover earth or nonradioactive waste, properly designed for long-term stability of the waste, should suffice.

Sludge should be dewatered prior to disposal to minimize migration of contaminants. Consideration should be given to the hydrogeology of the site and other factors affecting long-term stability of the wastes. Sites that fully comply with EPA's Subtitle D regulations and guidance under the Resource Conservation and Recovery Act (RCRA) would be appropriate for disposal of these wastes.

A jurisdiction may choose to ensure ground-water protection by specifying RCRA hazardous waste requirements (e.g., properly lined waste unit or sludge stabilization), to prevent seepage of contaminants from the landfill. The degree of additional protection a jurisdiction wants to provide against intrusion and misuse may vary from site to site but should be determined before waste disposal.

Disposal of solid wastes containing 50 to 2,000 pCi/g (dry weight) of radium should be determined on a case-by-case basis. Methods that comply with EPA's standards for disposal of uranium mill tailings should be considered (40 CFR 192). A decision not to employ such methods fully should be based on a demonstration of significant differences between the quantity and potential for migration of uranium mill tailings versus water treatment residuals. The disposal method should be augmented by long-term institutional controls to avoid future misuse of disposal sites; such controls are not normally already in place at sanitary landfills.

At a minimum, disposal in RCRA-permitted hazardous waste units should be considered. In states where disposal is licensed or permitted, disposal at these sites should be considered for radium-bearing solid wastes. At concentrations approaching 2,000 pCi/g, waste disposal within a licensed low-level radioactive waste disposal facility or a facility that is permitted by EPA or a state to dispose of discrete wastes, should be considered.

In states where lower concentration waste disposal is licensed or permitted, that option should be considered for disposal of solids containing 50 to 500 g/g (dry weight). Recovery of the uranium resource (e.g., at a uranium milling site) may be considered. NRC may require licensing of the recovered material (greater than 0.05 percent), however, as a source material under the provision of the Atomic Energy Act. Disposal at a licensed low-level radioactive waste facility should also be considered.

Solid wastes containing more than 2,000 pCi/g (dry weight) of radium or more than 500 %g/g (dry weight) uranium should be disposed of in a low-level radioactive waste disposal facility or at a facility that is permitted by EPA or a state to dispose of NORM wastes. Recovery of the uranium resource (e.g., at a uranium milling site) may be considered. NRC may require licensing of the material containing uranium (if greater than 0.05 percent uranium) as a source material under the provisions of the Atomic Energy Act. Also, when radium occurs with uranium, it is regulated as part of that source material.

10.3.2.2 Land Disposal

EPA does not recommend applying, mixing, or otherwise spreading NORM wastes onto open land (e.g., farm land, pasture, orchard or forestry lands, construction sites, roadbeds) for several reasons:

- Data relating to plant, animal, and human uptake and the potential exposure that may result from land-applied NORM wastes need to be collected and analyzed. Preliminary assessments suggest that human health risks from radon inhalation significantly increase in buildings constructed on such areas.
- The long-term control of and monitoring at a site that may contain higher than background levels of radionuclides (which sometimes are very long-lived) cannot be assured.
- Diluting wastes runs counter to EPA's general policy of concentrating wastes before safe disposal.
- EPA has not collected or reviewed any data on the status of surface runoff from sites that host land application of WTP residuals. Preliminary risk assessments using conservative assumptions, however, indicate that runoff and surface water contamination may pose a significant risk to the general population.
- Although certain types of sludge have been found to have beneficial properties as amendments to agricultural soils, EPA has not decided that the beneficial results outweigh the potential adverse results, such as food-chain contamination, future misuse of sites for building, and impacts on surface and groundwater quality.

10.4 Recordkeeping

EPA guidelines recommend that water treatment facilities keep records of the amount and composition of radioactive wastes (solid and liquid) they generate and the manner and location of their disposal. Repositories of wastes containing more than 50 pCi/g (dry weight) should be permanently marked to ensure long-term protection against future misuse of the site and/or its materials. This guideline is not meant to cause an increase in the federal recording requirements for water treatment facilities.

Chapter 11
Economics

Any water utility manager considering a new residuals management option for a water treatment plant (WTP) will want to estimate as closely as possible the cost of the residuals management option. This chapter provides cost equations that can be used to estimate the total annual costs associated with several different residuals treatment and disposal options. The cost equations provided here were developed using computer cost models, published cost information, and quotes provided by vendors. All costs presented in this chapter are in 1992 dollars.

These equations should be used for relative comparison purposes only. Certainly, more accurate, site-specific estimates should be developed for facility planning and budgeting purposes. The specific factors and treatment requirements will likely drive the costs up.

Final project costs have been found to be higher than those costs presented herein. This is primarily due to site-specific factors such as physical and chemical characteristics of the residuals, site constraints, operational costs, and differing state and local regulatory requirements.

11.1 Cost Assumptions

11.1.1 Capital Cost Assumptions

Table 11-1 presents cost factors and unit costs used to calculate the capital costs presented in this chapter. The land, buildings, and piping components are discussed in detail below. Other capital cost items (e.g., tanks, lagoons, and pumps) are calculated individually.

Table 11-1. Capital Cost Factors and Selected Unit Costs for WTP Facility Planning (U.S. EPA, 1993c)

Component	Factor/Unit Cost
Land	\$10,000/acre
Buildings	\$33.00/ft^2
Piping	5% of installed equipment[a]
Pipe fittings	20% of piping costs[b]
Electrical	1% of installed equipment
Instrumentation	1–2% of installed equipment
Engineering fee	15% of direct capital
Contingency, bonding, and mobilization	20% of direct capital
Contractor’s overhead and profit	12% of direct capital

[a] Piping costs are calculated directly when piping is a significant cost (e.g., for direct discharge).
[b] Factor is used when piping costs are calculated directly.

11.1.1.1 Land

Land prices vary considerably across the country and depend on proximity to metropolitan areas, the state of land development (i.e., improved or unimproved), current use, and scarcity of land. A cost of \$10,000 per acre is assumed for the cost estimates included in this chapter. Costs for suburban or industrial unimproved land average \$10,000 per acre and range from approximately \$4,000 to \$350,000 per acre depending on location. Costs for unimproved agricultural or rural land vary from approximately \$150 per acre in states with large tracts of undeveloped land, to \$2,200 per acre in states with small tracts of undeveloped land. An average cost for rural, unimproved land is \$1,000 per acre.

Costs for purchasing land are included in the capital cost equations provided here for all treatment and disposal processes that use surface impoundments. For example, assume an impoundment is used for liquid residuals storage prior to land application. The land costs are presented as a separate cost in the capital cost equation to allow the user to delete the cost for land if no purchase is required or to vary the cost of the land if the unit cost of land for a given area is available. A minimum land purchase of a 1/2 acre is used in developing land costs.

Land costs are not included for direct discharge and discharge to a publicly owned treatment works (POTW). It is assumed that the water treatment authority is able to obtain an easement for placement of a sewer or discharge line.

11.1.1.2 Buildings

Building costs also vary considerably based on geographic location. The building cost of \$33.00 per square foot, listed in Table 11-1, comes from *1992 Means Building Construction Cost Data* (R.S. Means Co., Inc., 1991). This cost is for a basic warehouse or storage building and includes electrical, foundations, heating, ventilation, air conditioning, and plumbing costs. It includes a 10 percent surcharge for structures of less than 25,000 square feet. This cost is the median cost for basic warehouse and storage buildings. The lower quartile cost is $21.30 per square foot and the upper quartile cost is $45.65 per square foot. Different warehouse or building configurations will change this unit cost. An aesthetically pleasing architecturally designed building or a complex building with several levels may cost substantially more than the building costs included in this chapter.

11.1.1.3 Piping and Pipe Fittings

Piping costs are calculated using two different methodologies that depend on the magnitude of the piping costs. For management systems in which the piping constitutes a significant portion of the capital cost (e.g., direct discharge and discharge to a POTW), piping costs are calculated by sizing individual pipe lengths and determining installation charges. For these management systems, pipe fittings are assumed to be 20 percent of the installed piping costs. For management systems where piping is not a significant cost (e.g., chemical precipitation and mechanical dewatering), piping and pipe fittings are assumed to be 5 percent of the installed equipment costs.

11.1.2 Operation and Maintenance Assumptions

Table 11-2 presents cost factors and unit costs used to calculate the operation and maintenance costs presented in this chapter.

11.1.2.1 Labor

The labor rate is based on a 20-city average for union scale laborers. The hourly rate of $28.00 is based on an average salary of $18.74 per hour including fringe benefits and a 50 percent labor overhead cost. Average salaries, including fringe benefits, by region range from $11.17 per hour in the southeast to $26.23 in the western United States.

Table 11-2. Operation and Maintenance Cost Factors and Unit Costs for WTP Facility Planning (U.S. EPA, 1993c)

Component	Factor/Unit Cost
Labor	$28.00/hour
Supervision	$42.00/hour
Supervision factor	10% of labor hours
Insurance and general and administrative expenses	2% of direct capital, excluding land and buildings
Maintenance	2 to 3% of direct capital, excluding land and buildings
Electricity	$0.086/kilowatt-hour

11.1.2.2 Supervision

The supervision rate is based on an estimated salary of $45,000 per year for a water system treatment manager. The $42.00 per hour assumes a 30 percent fringe benefit and a 50 percent labor overhead cost. The supervisor's salary is based on 1992 job postings in *Public Works* magazine (Public Works, 1992).

11.1.3 Total Annual Cost Assumptions

Total annual costs are equal to the sum of the yearly operation and maintenance costs plus the annualized capital costs. Annualized capital costs are calculated based on a 20-year operating life and an interest rate of 10 percent. The capital recovery factor for a 20-year operating life at a 10 percent interest rate is 0.1175. Alternate capital recovery factors can be calculated using the formula presented below.

$$\text{Capital recovery factor} = \frac{(1+i)^N\,(i)}{(1+i)^N - 1}$$

where
i = interest rate
N = number of years

11.1.4 Cost Components Excluded

The following cost components are not included in the cost estimates presented in this chapter. Other excluded costs, specific to each management option, are highlighted under their respective technology cost sections.

11.1.4.1 Sample Collection and Laboratory Analysis

Costs for sample collection and analysis to determine solids content, free liquids, toxicity characteristics, and other parameters are not included in the costs presented in this chapter. Sampling frequency is highly variable. Depending on the residuals management method selected, sampling requirements could be minimal or extensive.

11.1.4.2 Permits and Other Regulatory Requirements

Costs for permits and other regulatory requirements are not included in this chapter. Requirements vary considerably from state to state for a given management option. Permitting costs will vary based on the size and complexity of a unit and the local governing jurisdiction. Management methods that may require permits include landfills, land application, evaporation ponds, and storage lagoons. In addition, generators of hazardous waste are required to comply with RCRA generator requirements (40 CFR Part 262).

11.1.4.3 Mechanical Standby Equipment

Costs for standby equipment, or redundancy, are not included in these cost estimates.

11.1.5 Cost Equations

The cost equations for the capital components and for operation and maintenance were developed by estimating the costs for different residuals flow rates. Equipment sizes, retention times, installed equipment costs, operation and maintenance expenses, commercial disposal fees, component cost factors, and unit costs were developed based on published literature, (e.g., AWWA reports, technical journal articles, EPA documents), vendor contacts, and best engineering judgment. Capacities for major equipment components are presented in each cost section. Each cost equation covers the range of expected residuals flow rates unless a specific technology is not applicable to the entire range. For example, some technologies are not feasible for very high flow rates because of excessive land requirements or equipment limitations.

The limits of the cost equations presented for each residuals management method may be extended slightly beyond the ranges presented in this chapter. The upper or lower end for some equations, however, may represent the largest or smallest practical limits of a specific technology. Consequently, the design assumptions should be carefully reviewed before extending the range of the equations. Best engineering judgment should be used when extending the equations beyond their specified limits.

The capital and operation and maintenance equations encompass the residuals volumes generated by WTPs ranging from a design flow rate of 1.8 million gallons per day (mgd) (average operating flow rate, 0.7 mgd) to 430 mgd (average operating flow rate, 270 mgd). One option is to size the equipment (i.e., determine the capital cost) for the residuals produced from the water treatment system design flow rate, while calculating the operating cost using the average residuals flow rate. Thus, the equipment is sized for peak flow periods, but the actual operating costs may be more closely estimated using an average daily flow rate.

Typically, the design flow rate for a WTP may be two to three times the average flow rate for the facility. When calculating the costs for capital intensive management technologies such as chemical precipitation and mechanical dewatering, calculate both the capital and operation and maintenance costs using the average flow rate. Table 11-3 indicates whether the water treatment design flow rate or average flow rate should be used to calculate the capital and operation and maintenance costs. The design flow rate can be used to calculate costs, but this calculation may result in substantial overestimation of the cost to treat WTP residuals generated by an actual facility.

Table 11-3. Flow Rate Use in Calculating Facility Costs (U.S. EPA, 1993c)

Technology	Capital Cost	Operation & Maintenance Cost
Gravity thickening	Average	Average
Chemical precipitation	Average	Average
Mechanical dewatering	Average	Average
Nonmechanical dewatering	Design	Average
Evaporation ponds	Design	Average
POTW discharge	Design	Average
Direct discharge	Design	Average
French drain	Design	Average
Land application	Design	Average
Nonhazardous waste disposal	Average	Average
Hazardous waste disposal	N/A	N/A
Radioactive waste disposal	N/A	N/A

All costs presented in this chapter are in 1992 dollars. To update cost results obtained from either the cost equations or cost curves specified in the following sections, the *Builders' Construction Cost Indexes* are recommended and available in quarterly "cost report" issues of *Engineering News Record (ENR).* The ENR 20-city construction cost index or the *Means Construction Cost Index* are good general-purpose cost updating indexes. Their January 1992 values are 455.08 and 97.90, respectively.

11.1.6 Cost Curves

The cost curves in this chapter are the direct graphical representations of the cost equations. A break in the curve indicates that two equations covered two different ranges of the data. In these instances, the two curves typically do not meet at the same point at the overlap.

11.1.7 Calculating Residuals Management Costs

WTP residuals management costs are calculated using the capital, operating and maintenance and total annual cost equations presented in the following sections. Known residual flow rates from an operating facility may be used or residuals volumes can be estimated using the information presented in Chapter 3. The annual residuals management costs can be determined by calculating a total annual cost for each required treatment or disposal technology and then summing the costs associated with all WTP residuals management processes at the facility.

11.2 Gravity Thickening

The following section presents the design assumptions, capital components, and the operation and maintenance components used to estimate the costs for gravity thickening of filter backwash streams from chemical coagulation, or lime softening residuals streams and/or their sludge waste streams.

11.2.1 Design Assumptions

- Gravity thickening is used to pretreat filter backwash or sludges from chemical coagulation and lime softening.
- The waste streams flow by gravity from the treatment plant to settling tank.
- The backwash volume ranges from 7,000 to 2,500,000 gallons per day.
- The total backwash volume is 0.5 to 2 percent of the design flow rate.
- The initial solids concentration of the backwash is 0.1 percent and the discharge concentration is equal to 1 percent.
- The backwash is thickened in a tank.
- The volume of backwash is reduced by 90 percent.
- The supernatant is pumped to the head of the treatment plant.
- The thickened sludges are discharged to another treatment for further dewatering.

11.2.2 Capital Components

The capital components for each filter backwash system consist of the following items: holding tank, piping and fittings, pump, trenching, electrical, and instrumentation.

Table 11-4 indicates the holding tank capacities used to develop the capital cost equation for gravity thickening.

Table 11-4. Holding Tank Capacities, Gravity Thickening (U.S. EPA, 1993c)

Waste Stream Volume (gpd)	Number of Tanks	Waste Stream Settling Tank Capacity (gal)
7,000	1	30,000
13,300	1	50,000
36,000	1	30,000
96,000	1	30,000
165,000	1	50,000
270,000	1	50,000
510,000	1	50,000
1,050,000	2	50,000
2,150,000	2	50,000

11.2.3 Operation and Maintenance Components

The operation and maintenance components for each gravity thickening system include: electricity, labor and supervision, maintenance labor and materials, insurance, and general and administration.

11.2.4 Cost Components Excluded

The following cost components are not included in the cost equations for gravity thickening systems:

- Supernatant disposal, if the supernatant cannot be pumped to the head of the treatment plant.
- Thickened sludge disposal.
- Costs for supernatant disposal, if the supernatant cannot be pumped to the head of the treatment plant, and treatment of the thickened sludge can be determined using other sections of this chapter.

11.2.5 Gravity Thickening Cost Equations and Cost Curves

The cost equations for the capital components and the operation and maintenance components for gravity thickening were developed by estimating the costs for nine different filter backwash flow rates. The filter backwash and sludge flow rates (X) used in the equations below are the average daily volumes generated (e.g., for filter backwash, 2.5 percent of the daily treated water flow rate). The capital cost equation calculates the total capital cost (i.e., installed capital plus indirect capital costs). The total annual cost is calculated based on the capital and operation and maintenance costs obtained using the equations presented below. Capital costs for gravity thickening are presented graphically in Figure 11-1 and operation and maintenance costs are shown in Figure 11-2.

Capital Costs

$$Y = 32{,}400 + [41.7(X^{0.50})]$$

where
Y = $
X = gallons of filter backwash/sludge per day
Range: 7,000 gpd ≤ X ≤ 270,000 gpd

$$Y = 21{,}700 + [66.5(X^{0.50})]$$

where
Y = $
X = gallons of filter backwash/sludge per day

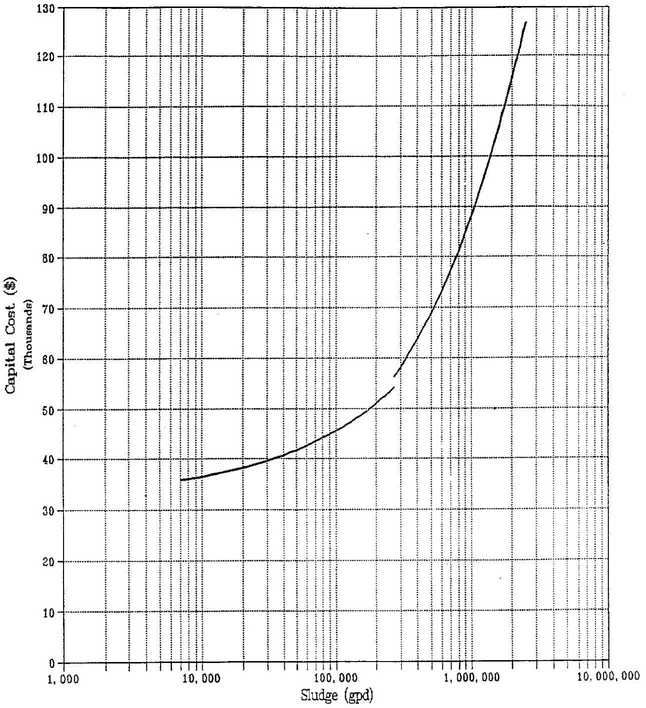


Figure 11-1. Capital costs for gravity thickening.

Range: 270,000 gpd < X ≤ 2,500,000 gpd

Operation and Maintenance

$$Y = 5{,}800 + [8.04(X^{0.50})]$$

where
Y = $/year
X = gallons of filter backwash/sludge per day
Range: 7,000 gpd ≤ X ≤ 2,500,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = $/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.3 Chemical Thickening

This section presents the design assumptions, capital components, and the operation and maintenance components used to estimate the costs for chemical thickening of residuals streams such as ion exchange backwash and reverse osmosis brines.

11.3.1 Design Assumptions

- The chemical precipitation system consists of mixing and holding tanks for the lime solution, a precipitation tank, a clarifier, agitators, and pumps.

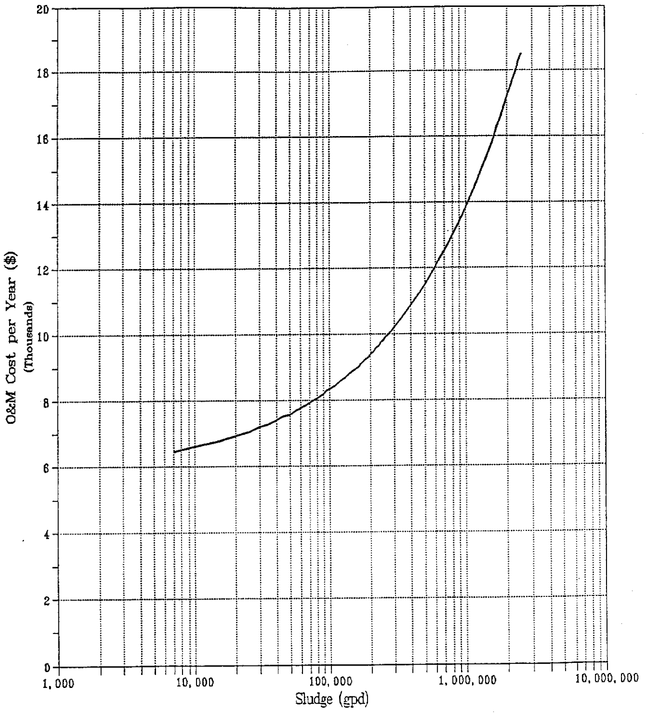


Figure 11-2. O&M costs for gravity thickening.

- The precipitation tank has a 1/2-hour retention time and a 5 percent overdesign.
- The clarifier (settling tank) has a 1- to 2-hour retention time.
- Waste brines flow to the treatment system under pressure from the ion exchange or reverse osmosis system.
- The chemical precipitation equipment is located in the water treatment building.
- Sludge from the clarifier may require additional dewatering prior to disposal.
- The sludge volumes generated by chemical precipitation are 5 and 2 percent of the influent volumes for reverse osmosis and ion exchange brines, respectively.

11.3.2 Capital Components

The capital components for each chemical precipitation system consist of: carbon steel mixing tank, carbon steel holding tank, carbon steel precipitation tank, clarifier, agitators, sludge pumps, building, piping, electrical and instrumentation.

Table 11-5 indicates the brine volumes, tank capacities, and clarifier capacities used to develop the capital cost equation for chemical precipitation.

Table 11-5. Capital Cost Equation Determinants, Chemical Precipitation (U.S. EPA, 1993c)

Residuals Flow Rate (gpd)	Precipitation Tank Size (gal)	Mix Tank Capacity (gal)	Clarifier Capacity (gal)
27,400	650	50	2,500
67,500	2,000	75	8,000
274,000	6,000	200	24,000
500,000	12,000	600	40,000
1,000,000	23,000	1,200	50,000
2,740,000	60,000	1,500	40,000 x 2

11.3.3 Operation and Maintenance Components

The operation and maintenance components for each chemical precipitation system include: lime, electricity, labor, maintenance labor and materials, insurance, general and administration, and water.

11.3.4 Cost Components Excluded

The following cost components are not included in the cost equations for chemical precipitation systems since they are accounted for elsewhere:

- Sludge dewatering (if needed for the selected disposal option).
- Sludge disposal.
- Clarifier overflow disposal, if the overflow cannot be pumped to the head of the treatment works.

Costs for sludge dewatering and sludge disposal techniques are included in other sections of this chapter. Costs for clarifier overflow disposal, if the overflow cannot be pumped to the head of the treatment works, can be determined using other sections of this chapter.

11.3.5 Chemical Precipitation Cost Equations and Cost Curves

The cost equations for the capital components and operation and maintenance were developed by estimating the costs for six different brine flow rates. The capital cost equation calculates the total capital cost (i.e., installed capital plus indirect capital costs). Equipment costs and building costs are separate components in the capital cost equation to enable the user to exclude the cost for a building addition if it is not necessary. The total annual cost is calculated based on the capital and operation and maintenance costs obtained using the equations presented below.

A capital cost curve for chemical precipitation is shown in Figure 11-3. Operation and maintenance costs for chemical precipitation are shown in Figure 11-4.

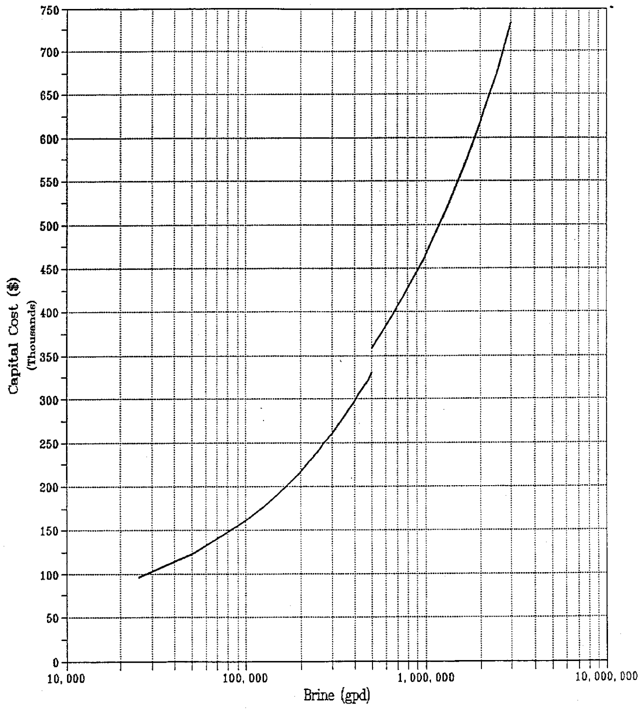


Figure 11-3. Capital costs for chemical precipitation.

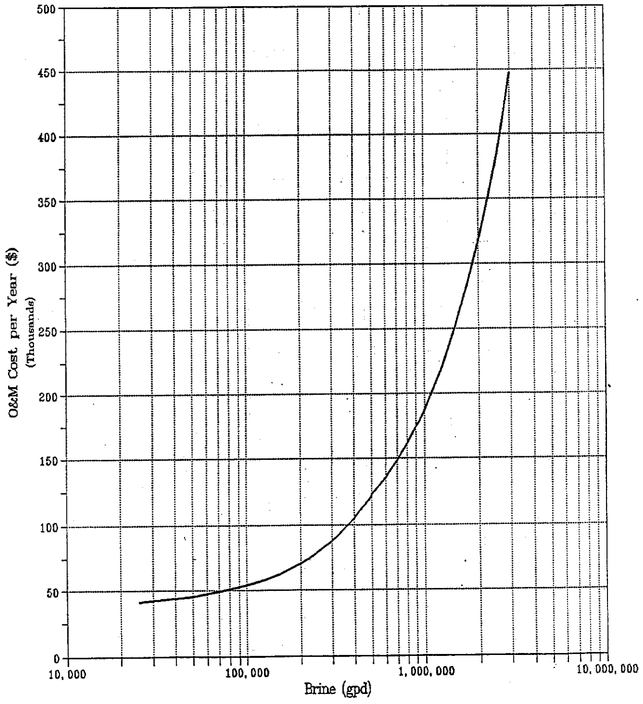


Figure 11-4. O&M costs for chemical precipitation.

Capital Costs

$$Y = [34{,}600 + 206(X^{0.50})] + [110(X^{0.55})]$$

$$Y = [\text{equipment cost}] + [\text{building cost}]$$

where
Y = $
X = gallons of brine per day
Range: 25,000 gpd ≤ X ≤ 500,000 gpd

$$Y = [34{,}600 + 206(X^{0.50})] + [65{,}000 + 160(X^{0.50})]$$

$$Y = [\text{equipment cost}] + [\text{building cost}]$$

where
Y = $
X = gallons of brine per day
Range: 500,000 gpd < X ≤ 3,000,000 gpd

Operation and Maintenance

$$Y = 37{,}300 + 0.17(X)$$

where
Y = $/year
X = gallons of brine per day
Range: 25,000 gpd ≤ X ≤ 500,000 gpd

$$Y = 57{,}500 + 0.13(X)$$

where
Y = $/year
X = gallons of brine per day
Range: 500,000 gpd < X ≤ 3,000,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = $/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.4 Mechanical Sludge Dewatering

11.4.1 Pressure Filter Press Cost Components

This section presents the design assumptions, capital components, and the operation and maintenance components used to estimate the costs for a pressure filter, or plate and frame, press for mechanical sludge dewatering.

11.4.1.1 Design Assumptions

- Pressure filter presses are effective for residuals flow rates greater than 100 gallons per day.
- The polymer feed system consists of a polymer storage tank, a polymer pump, a polymer/sludge contact tank, and a 20- to 30-minute holding tank.
- A positive displacement pump delivers residuals to the filter press.
- The filter press operates on a batch basis.
- Filtrate collects in a filtrate holding tank before being pumped to the head of the treatment plant.
- In smaller systems, the filter cake collects in a small, wheeled container, which is emptied into a larger solids bin as necessary.
- In larger systems, the filter cake drops into rolloff bins located beneath the filter press.
- Accumulated solids require disposal on a periodic basis.
- The dewatered residuals volume is 0.03 and 10 percent of the initial volume for alum and lime residuals, respectively.

11.4.1.2 Capital Components

The capital components for each pressure filter press system consist of the following items: Schedule 40 steel piping, polymer feed system, positive displacement pump(s), pressure filter press(es), steel filtrate tank and pump, filter cake storage bin, building, piping, electrical, and instrumentation.

Table 11-6 indicates the sludge volumes and filter press capacities used to develop the capital cost equation for pressure filter presses.

11.4.1.3 Operation and Maintenance Components

The operation and maintenance components for each pressure filter press system include: electricity, polymer, labor, maintenance labor and materials, insurance, and general and administration.

11.4.1.4 Cost Components Excluded

The following cost components are not included in the cost equations for pressure filter press systems:

- Dewatered residuals disposal.
- Filtrate disposal, if the filtrate cannot be pumped to the head of the treatment plant.
- Acid wash system, if required.

Costs for residuals disposal techniques are included in other sections of this chapter. Costs for filtrate disposal, if the overflow cannot be pumped to the head of the treatment works, can be determined using other sections of this chapter.

Table 11-6. Capital Cost Equation Determinants, Pressure Filter Presses (U.S. EPA, 1993c)

Sludge Flow Rate (gpd)	Filter Press Capacity (ft^3)
500	2.0
2,000	10
5,000	24
50,000	50
250,000	130 x 2
600,000	160 x 3

11.4.1.5 Pressure Filter Press Cost Equations and Cost Curves

The cost equations for the capital components and operation and maintenance were developed by estimating the costs for six different residuals flow rates. The capital cost equation calculates the total capital cost (i.e., installed capital plus indirect capital costs). Equipment costs and building costs are separate components in the capital cost equation to enable the user to exclude the cost for a building addition if it is not necessary. The total annual cost is calculated based on the capital and operation and maintenance costs obtained using the equations presented below. A capital cost curve for pressure filter press is shown in Figure 11-5. An operation and maintenance cost curve is shown in Figure 11-6.

Capital Cost

$$Y = [15{,}600 + 1{,}520(X^{0.50})] + [4{,}600 + 3.0(X)]$$

$$Y = [\text{equipment cost}] + [\text{building cost}]$$

where
Y = \$
X = gallons of sludge per day
Range: 500 gpd $\leq X \leq$ 50,000 gpd

$$Y = [4{,}181(X^{0.50}) - 579{,}900] + [22{,}300 + (9.8 \times 10^{-2})(X)]$$

$$Y = [\text{equipment cost}] + [\text{building cost}]$$

where
Y = \$
X = gallons of sludge per day
Range: 50,000 gpd $< X \leq$ 600,000 gpd

Operation and Maintenance

$$Y = 13{,}400 + 0.92(X)$$

where
Y = \$/year
X = gallons of sludge per day
Range: 500 gpd $\leq X \leq$ 50,000 gpd

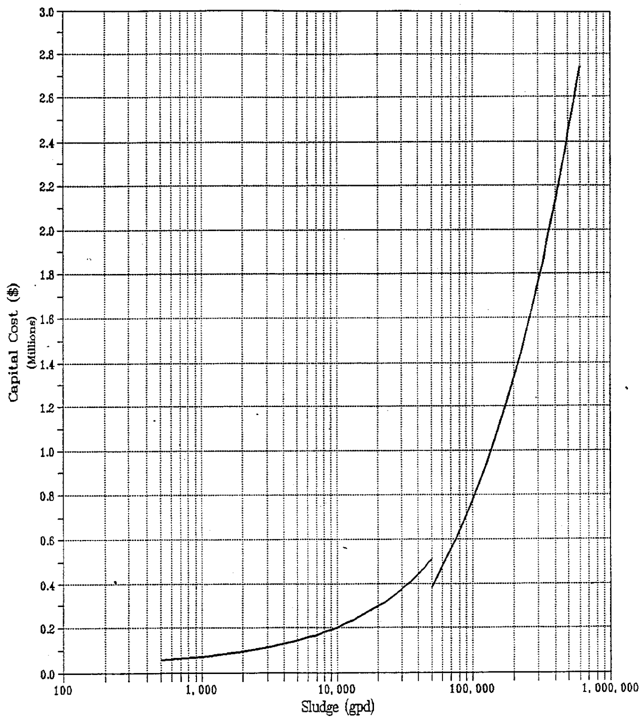


Figure 11-5. Capital costs for pressure filter press.

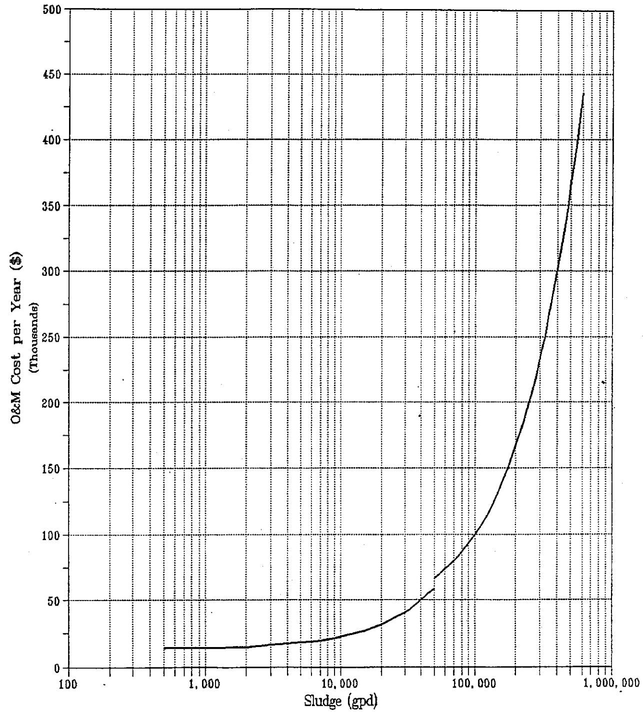


Figure 11-6. O&M costs for pressure filter press.

$$Y = 32{,}900 + 0.67(X)$$

where
Y = \$/year
X = gallons of sludge per day
Range: 50,000 gpd $< X \leq$ 600,000 gpd

Total Annual Cost

Y = [CAP(CRF)] + O&M

where
Y = $/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O& = operation and maintenance cost

11.4.2 Scroll Centrifuge Cost Components

This section presents the design assumptions, capital components, and the operation and maintenance components used to prepare the costs for the solid-bowl, or scroll centrifuge, used for mechanical sludge dewatering.

11.4.2.1 Design Assumptions

- The polymer feed system consists of polymer storage tank, a polymer pump, and a polymer/sludge contact tank, and a 20- to 30-minute holding tank.
- A positive displacement pump feeds conditioned sludge to the scroll centrifuge.
- The scroll centrifuge operates 8 to 24 hours per day, depending on the size of the water system.
- Centrate collects in a centrate holding tank before being pumped to the head of the treatment works.
- Dewatered residuals collect in a small, wheeled container that is emptied into a larger solids bin as necessary.
- Large systems are located on the second floor and the dewatered residuals drop into sludge containers located below the centrifuge.
- Accumulated solids require disposal on a periodic basis.
- The dewatered residuals volume is 0.04 of the initial residuals volume.

11.4.2.2 Capital Components

The capital components for each scroll centrifuge system are: Schedule 40 steel piping, polymer feed system, positive displacement pump, scroll centrifuge with backdrive, steel centrate tank and pump, dewatered sludge storage bin, building, piping, electrical, and instrumentation.

Table 11-7 indicates the sludge flow rates and centrifuge motor sizes used to develop the capital cost equation for the scroll centrifuge system.

Table 11-7. Capital Cost Equation Determinants, Scroll Centrifuge (U.S. EPA, 1993c)

Sludge Flow Rate (gpd)	Centrifuge Motor Size
2,000	7 1/2 hp
5,000	7 1/2 hp
50,000	10 hp
250,000	25 hp
600,000	40 hp
2,500,000	80 hp x 2
5,200,000	80 hp x 4

11.4.2.3 Operation and Maintenance Components

The operation and maintenance components for each scroll centrifuge system include: electricity, polymer, labor, maintenance labor and materials, insurance, and general and administration.

11.4.2.4 Cost Components Excluded

The following cost components are not included in the cost equations for scroll centrifuge systems: scroll cleaning apparatus, dewatered residuals disposal, and centrifuge overflow disposal, if the overflow cannot be pumped to the head of the treatment plant.

Costs for residuals disposal techniques are included in other sections of this chapter. Costs for centrifuge overflow disposal, if the overflow cannot be pumped to the head of the treatment works, can be determined using other sections of this chapter.

11.4.2.5 Scroll Centrifuge Cost Equations and Cost Curves

The cost equations for the capital components and operation and maintenance were developed by estimating the costs for seven different sludge flow rates. The capital cost equation calculates the total capital cost (i.e., installed capital plus indirect capital costs). Equipment costs and building costs are separate components in the capital cost equation to enable the user to exclude the cost for a building addition if it is not necessary. The total annual cost is calculated based on the capital and operation and maintenance costs obtained using the equations presented below. A capital cost curve for scroll centrifuge is shown in Figure 11-7 and operation and maintenance costs are shown in Figure 11-8.

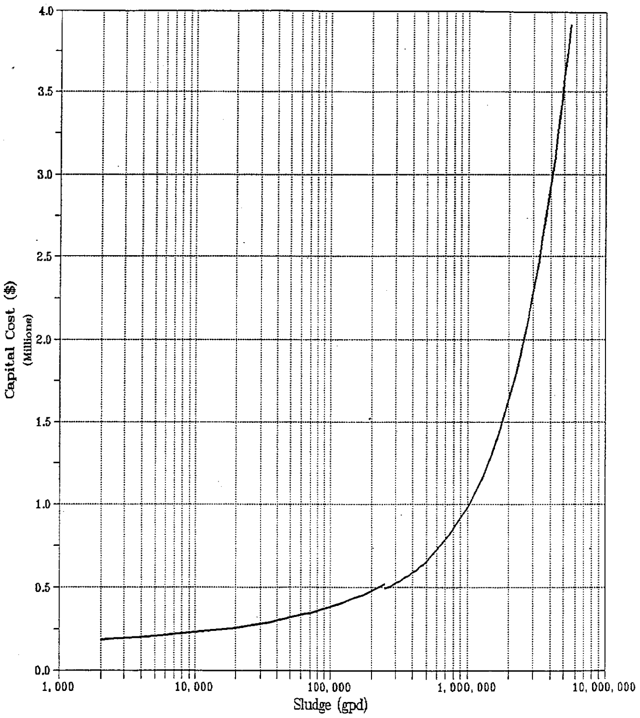


Figure 11-7. Capital costs for scroll centrifuge.

Capital Cost

$$Y = [138{,}400 + 666(X^{0.50})] + [15{,}400 + 59(X^{0.50})]$$

$$Y = [\text{equipment cost}] + [\text{building cost}]$$

where
Y = \$
X = gallons of sludge per day
Range: 2,000 gpd $\leq X <$ 250,000 gpd

$$Y = [287{,}000 + 0.63(X)] + [40{,}400 + (2.26 \times 10^{-2})(X)]$$

$$Y = [\text{equipment cost}] + [\text{building cost}]$$

where
Y = \$
X = gallons of sludge per day
Range: 250,000 gpd $\leq X \leq$ 5,500,000 gpd

Operation and Maintenance

$$Y = 17{,}500 + 311(X^{0.50})$$

where
Y = \$/year
X = gallons of sludge per day
Range: 2,000 gpd $\leq X <$ 250,000 gpd

$$Y = 72{,}000 + 0.43(X)$$

where
Y = \$/year
X = gallons of sludge per day
Range: 250,000 gpd $\leq X \leq$ 5,500,000 gpd

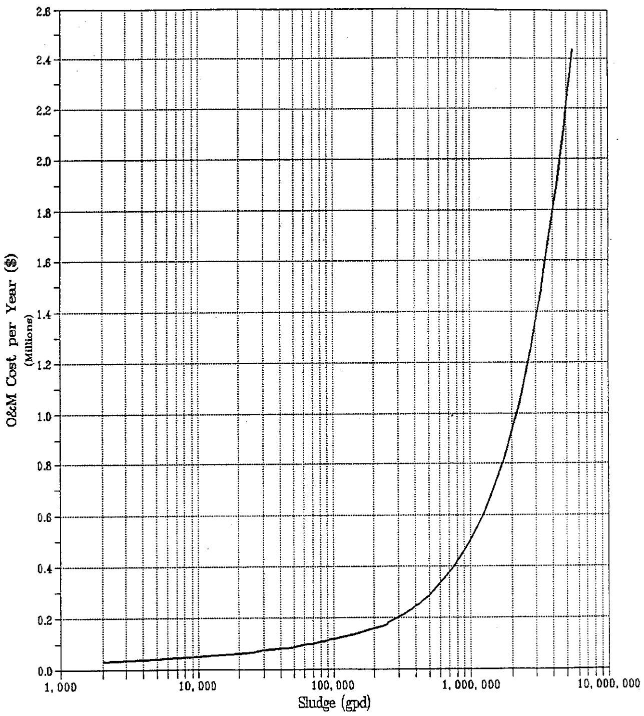


Figure 11-8. O&M costs for scroll centrifuge.

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = \$/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.5 Nonmechanical Sludge Dewatering

11.5.1 Storage Lagoons Cost Components

This section presents the design assumptions, capital components, and the operation and maintenance components used to estimate the costs for residuals dewatering using storage lagoons.

11.5.1.1 Design Assumptions

- Storage lagoons are used to dewater alum residuals and are periodically dredged.
- Permanent lagoons are used to dewater lime residuals; each storage lagoon has a 10-year capacity.
- The lagoons are earthen basins lined with a synthetic membrane and a geotextile support fabric. They have no underdrains, but have a variable height outlet structure to discharge supernatant. Permanent lagoons are 6- to 12-feet deep and storage lagoons range from 3- to 5-feet deep.

- Decanted supernatant from the lagoon is collected and pumped to the head of the treatment plant.

- Residuals flow from the treatment plant to the lagoon by gravity via 1,250 to 4,000 feet of 2- to 6-inch diameter PVC piping. Pipes are laid 4 feet below grade.

- Alum treatment plants operate two lagoons. Each lagoon has a 6-month storage capacity. After 6 months, the second lagoon is used while free liquid from the idle lagoon is allowed to evaporate. The remaining free liquid is decanted and returned to the head of the treatment plant. Sludge is dredged from the lagoon on an annual basis.

- Lime treatment plants operate two or four lagoons to provide a total of 20 years of permanent storage. Each lagoon has a 6- or 12-month storage capacity prior to drying and decanting. Free liquid is continuously decanted from the lagoon while it is accepting sludge and returned to the head of the treatment plant. After the 6- or 12-month period, another lagoon is used while free liquid from the idle lagoon is decanted and evaporated. The remaining sludge is allowed to dry prior to the next sludge application.

- Approximately 1.5 cubic feet of solids per 1,000 gallons of initial alum residuals volume will require final disposal from the storage lagoons.

11.5.1.2 Capital Components

The capital components for each storage lagoon consist of: two or four lined earthen lagoons, piping and fittings, trenching, decant collection pump, electrical, instrumentation, and land clearing.

Tables 11-8 and 11-9 indicate the sludge volumes, number of ponds, and total lagoon surface area used to develop the capital cost equation for lime softening and alum storage lagoons, respectively.

Table 11-8. Capital Cost Equation Determinants, Lime Softening Storage Lagoons (U.S. EPA, 1993c)

Lime Softening Sludge Volume (gpd)	Number of Ponds	Total Lagoon Surface Area (acres)
7,000	2	5
20,000	2	7
50,000	2	18
100,000	2	37
500,000	4	112
1,000,000	4	224

Table 11-9. Capital Cost Equation Determinants, Alum Storage Lagoons (U.S. EPA, 1993c)

Alum Sludge Volume (gpd)	Number of Ponds	Total Surface Area (acres)
250	2	0.1
1,000	2	0.4
5,000	2	2
10,000	2	4
50,000	2	19
100,000	2	22
200,000	2	45
500,000	2	112

11.5.1.3 Operation and Maintenance Components

The operation and maintenance components for each storage lagoon include: labor and supervision, maintenance labor and materials, electricity, insurance, general and administration, and annual sludge removal (alum sludge only).

11.5.1.4 Cost Components Excluded

The following cost components are not included in the cost equations for storage lagoons: sludge dewatering equipment, dewatered sludge disposal, and supernatant disposal, if the supernatant cannot be pumped to the head of the treatment plant.

Costs for sludge disposal techniques are included in other sections of this chapter. Costs for supernatant disposal, if the supernatant cannot be pumped to the head of the treatment plant, can be determined using other sections of this chapter.

11.5.1.5 Lime Softening Storage Lagoon Cost Equations and Cost Curves

The cost equations for the capital components and operation and maintenance were developed by estimating the costs for six different sludge flow rates. The capital cost equation calculates the total capital cost (i.e., installed capital plus indirect capital costs). Equipment costs and land costs are separate components in the capital cost equation to enable the user to exclude the cost for purchasing land if it is not necessary. The total annual cost is calculated from the capital and operation and maintenance costs obtained using the equations presented below. A capital cost curve for lime softening storage lagoon is shown in Figure 11-9. An operation and maintenance cost curve is shown in Figure 11-10.

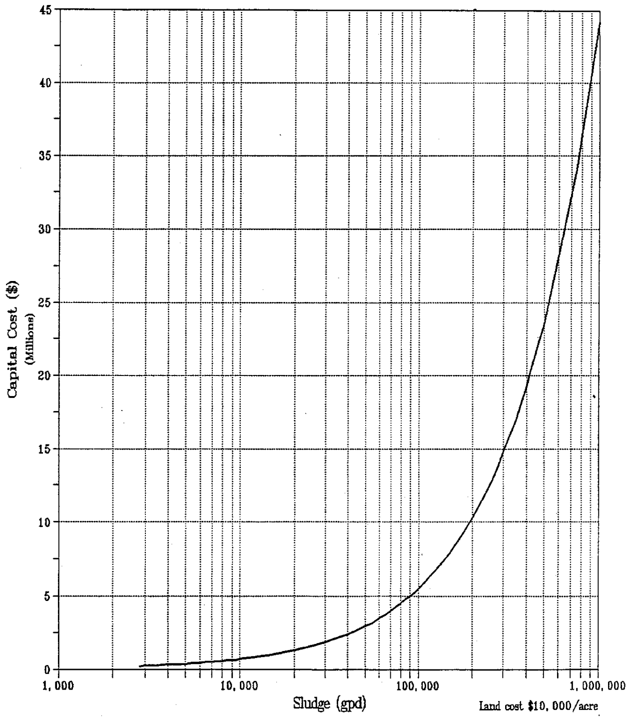


Figure 11-9. Capital costs for lime softening storage lagoon.

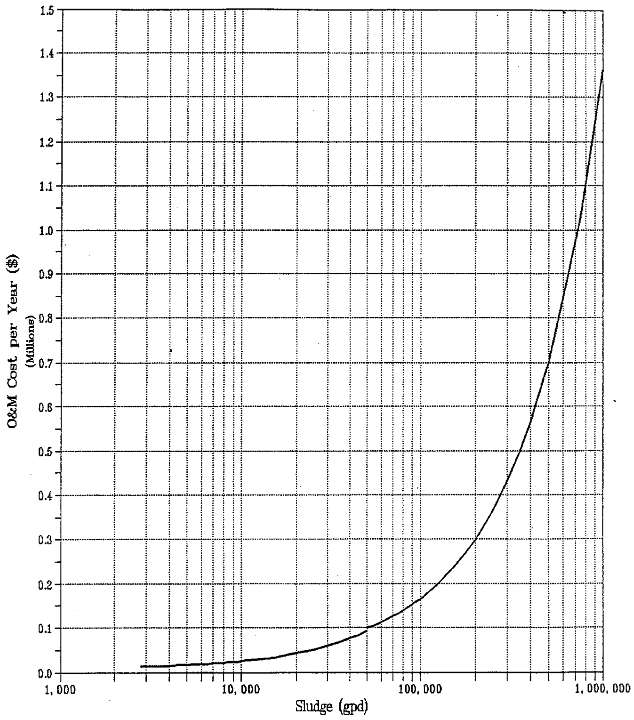


Figure 11-10. O&M costs for lime softening storage lagoon.

Capital Costs

$$Y = [144(X^{0.91})] + \{[(1.57 \times 10^{-3})(X^{0.87})](Z)\}$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = $
X = gallons of sludge per day
Z = land cost in $/acre (e.g., $10,000/acre)
Range: 2,800 gpd ≤ X ≤ 1,000,000 gpd

Operation and Maintenance

$$Y = 7{,}700 + 1.69(X)$$

where
Y = $
X = gallons of sludge per day
Range: 2,800 gpd ≤ X ≤ 50,000 gpd

$$Y = 33{,}100 + 1.33(X)$$

where
Y = $/year
X = gallons of sludge per day
Range: 50,000 gpd < X ≤ 1,000,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = $/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.5.1.6 Alum Sludge Storage Lagoons Cost Equations and Cost Curves

The cost equations for the capital components and operation and maintenance were developed by estimating the costs for eight different sludge flow rates. The capital cost equation calculates the total capital cost (i.e., installed capital plus indirect capital costs). Equipment costs and land costs are separate components in the capital cost equation to enable the user to exclude the cost for purchasing land if it is not necessary. The total annual cost is calculated from the capital and operation and maintenance costs obtained using the equations presented below. A capital cost curve for alum sludge storage lagoons is shown in Figure 11-11. An operation and maintenance cost curve is shown in Figure 11-12.

Capital Costs

$$Y = [13{,}700 + 60.8(X)] + \{[0.5 + (3.44 \times 10^{-4})(X)](Z)\}$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = $
X = gallons of sludge per day
Z = land cost in $/acre (e.g., $10,000/acre)
Range: 250 gpd ≤ X ≤ 10,000 gpd

$$Y = [326{,}500 + 28.6(X)] + \{[4.2 + (2.35 \times 10^{-4})(X)](Z)\}$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = $

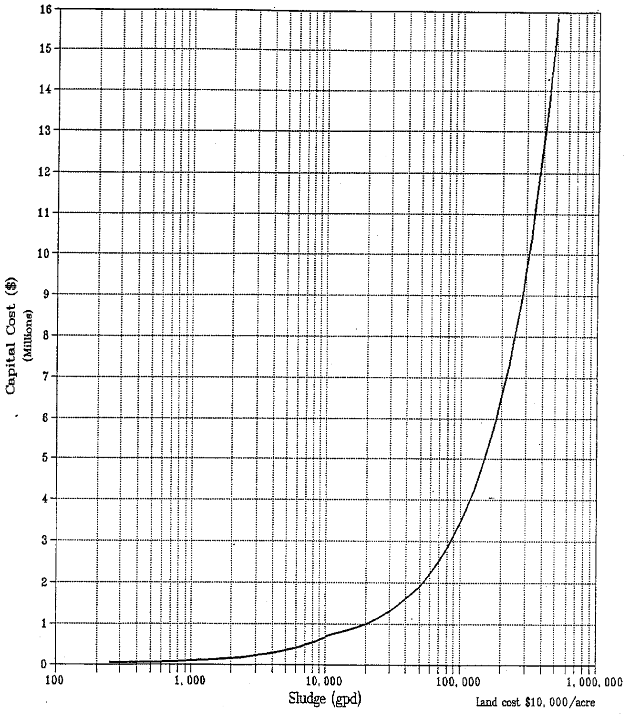


Figure 11-11. Capital costs for alum sludge storage lagoon.

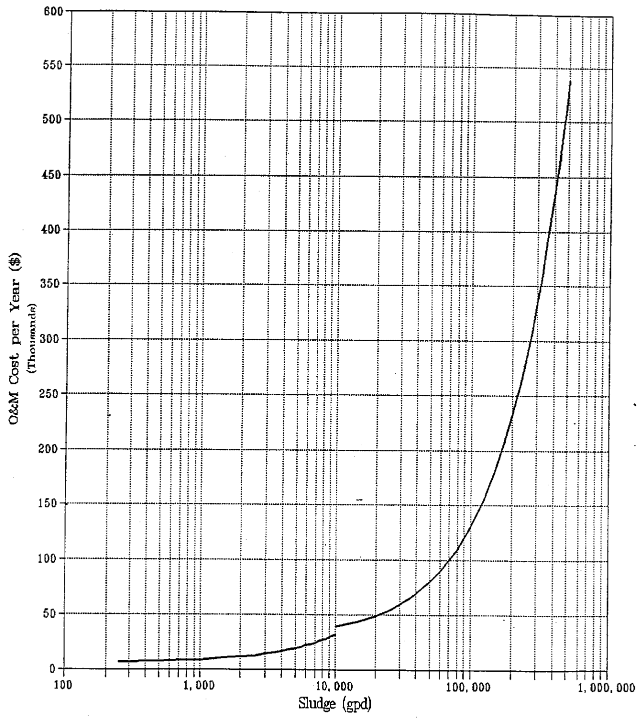


Figure 11-12. O&M costs for alum sludge storage lagoon.

X = gallons of sludge per day
Z = land cost in $/acre (e.g., $10,000/acre)
Range: 10,000 gpd ≤ X ≤ 500,000 gpd

Operation and Maintenance

$$Y = 6{,}100 + 2.56(X)$$

where
Y = $/year
X = gallons of sludge per day

Range: 250 gpd ≤ X ≤ 10,000 gpd

$$Y = 28{,}500 + 1.02(X)$$

where
Y = $/year
X = gallons of sludge per day
Range: 10,000 gpd ≤ X ≤ 500,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = $/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.5.2 Evaporation Ponds Cost Components

The following section presents the design assumptions, capital components, and the operation and maintenance components used to prepare the costs for evaporation ponds.

11.5.2.1 Design Assumptions

- The evaporation pond is used for treatment of IX or RO brines. The influent solids concentration ranges from 1.5 to 3.5 percent by weight.
- RO systems with flow rates of 4.8 mgd and greater, and IX systems with flow rates of 51 mgd and greater are not considered applicable for evaporation ponds since over 150 acres is required to evaporate the daily generation volumes.
- Waste brines flow from the treatment plant to the evaporation pond by gravity or by the pressure from the treatment system via 1,250 to 4,000 feet of 4- to 8-inch diameter PVC piping. Pipes are laid 4 feet below grade.
- Waste brine flow rates range from 31,000 to 500,000 gallons per day.
- The pond is designed for a geographical region with a net annual evaporation rate of at least 45 inches per year.
- The pond has no outlet.
- The pond sides and earthen berm are engineered to have 2.5 to 1 side slopes and 2 feet of free board.
- Soils cut from the excavation of the basin are used to construct the berm.

- The pond is constructed with a synthetic membrane liner and a geotextile support fabric; 1 foot of sand is placed on top of the liner.
- Piping from the treatment process is sized to discharge the waste brines to the ponds as they are generated. The treatment process is assumed to run 8 to 16 hours per day.
- The evaporation ponds are sized with sufficient surface area to evaporate the average daily flow. The pond depth is 2 feet, which provides solids storage volume and accommodates peak flows.

11.5.2.2 Capital Components

The capital components for each evaporation pond are: evaporation ponds, piping and fittings, pumps, land clearing, and instrumentation.

Table 11-10 indicates the brine volumes, number of ponds, and the total required surface area used to develop the capital cost equation for evaporation ponds.

Table 11-10. Capital Cost Equation Determinants, Evaporation Ponds (U.S. EPA, 1993c)

Brine Volume (gpd)	Number of Ponds	Total Surface Area (acres)
31,000	2	10
50,000	2	15
100,000	2	30
200,000	4	60
400,000	6	120
500,000	3	150

11.5.2.3 Operation and Maintenance Components

The operation and maintenance components for each evaporation pond include: mowing, labor and supervision, maintenance labor and materials, insurance, and general and administration.

11.5.2.4 Cost Components Excluded

The cost component not included in the cost equations for evaporation ponds is dewatered sludge disposal. Costs for sludge disposal are included in other sections of this chapter.

11.5.2.5 Evaporation Ponds Cost Equations and Cost Curves

The cost equations for the capital components and for operation and maintenance were developed by estimating the costs for six different brine flow rates. The capital cost equation calculates the total capital cost (i.e., installed capital plus indirect capital cost). The total annual cost is calculated from the capital and operation and maintenance costs obtained using the equations presented below. A capital cost curve for evaporation ponds is shown in Figure 11-13. An operation and maintenance cost curve is shown in Figure 11-14.

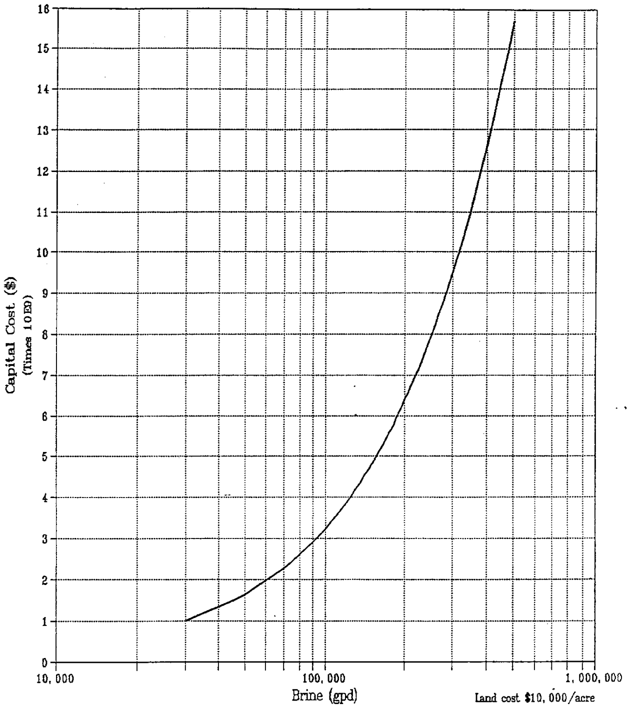


Figure 11-13. Capital costs for evaporation ponds.

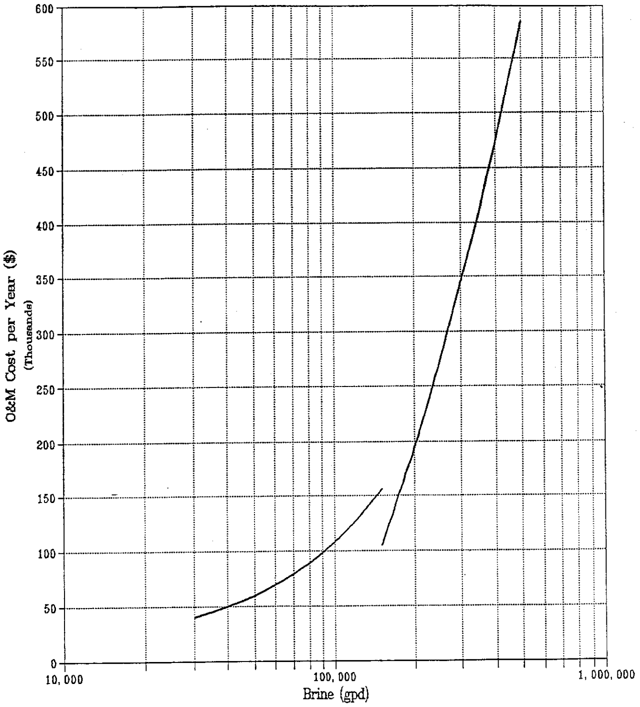


Figure 11-14. O&M costs for evaporation ponds.

Capital Cost

$$Y = [193{,}600 + 28.8(X)] + [4.08(X^{0.98})(Z)]$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = $
X = gallons of brine per day
Z = land cost in $/acre (e.g., $10,000/acre)
Range: 30,000 gpd ≤ X ≤ 150,000 gpd

$$Y = [44{,}820(X^{0.5}) - (1.43 \times 10^{7})] + [4.08(X^{0.98})(Z)]$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = $
X = gallons of brine per day
Z = land cost in $/acre (e.g., $10,000/acre)
Range: 150,000 gpd < X ≤ 500,000 gpd

Operation and Maintenance

$$Y = 11{,}600 + 0.96(X)$$

where
Y = $/year
X = gallons of brine per day
Range: 30,000 gpd ≤ X ≤ 150,000 gpd

$$Y = 1{,}500(X^{0.50}) - 476{,}000$$

where
Y = $/year
X = gallons of brine per day
Range: 150,000 gpd < X ≤ 500,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = $/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.6 Discharge to Publicly Owned Treatment Works

This section presents the design assumptions, capital components, and the operation and maintenance components used to estimate the costs for discharging to a POTW.

11.6.1 Design Assumptions

- Residuals flow rates range from 2,000 to 25 million gallons per day, depending on the water treatment technology.
- The sewer line connection is 500 feet or 1,000 feet from the water treatment system.
- The residuals stream flows by gravity or under pressure from the treatment process to the sewer line.
- Two-inch minimum diameter pipe is used to prevent clogging. Pipe diameter ranges from 2 to 24 inches.
- PVC or reinforced concrete piping is used.
- A 1- to 2-day storage lagoon is optional.

11.6.2 Capital Components

The capital components of system for discharge to a POTW consist of piping and fittings, trenching, and land clearing.

In addition, a pump, storage lagoon, electrical, and instrumentation are added for those facilities electing to store residuals prior to discharge to a POTW.

Table 11-11 indicates the brine volumes, pipe diameters, and lagoon capacities used to develop the capital cost equation for discharge to POTW systems.

Table 11-11. Capital Cost Equation Determinants, Discharge to POTW (U.S. EPA, 1993c)

Residuals Flow Rate (gpd)	Pipe Diameter (in.)	Lagoon Capacity (gal)
30,000	2	50,000
67,500	2	115,000
162,500	3	330,000
500,000	4	750,000
750,000	6	1,125,000
1,000,000	6	1,500,000
10,000,000	24	15,000,000
25,000,000	36	37,500,000

11.6.3 Operation and Maintenance Components

The operation and maintenance components of each system for discharge to a POTW include: labor and supervision, maintenance labor and materials, basic POTW charges, total suspended solids (TSS) surcharges, electricity, insurance, and general and administration.

11.6.4 Cost Components Excluded

The cost components not included in the cost equations for systems for discharge to a POTW are any fees charged by the POTW for the initial connection and land cost.

It is assumed the water treatment facility would be able to gain access for piping via an easement.

11.6.5 Cost Equations and Cost Curves for Discharge to Publicly Owned Treatment Works

The cost equations for the capital components and operation and maintenance were developed by estimating the costs for eight different brine flow rates. The capital cost equation calculates the total capital cost (i.e., installed capital plus indirect capital costs). The total annual cost is calculated from the capital and operation and maintenance costs obtained using the equations presented below.

11.6.5.1 500 Feet of Discharge Pipe

Capital costs for 500 feet of discharge pipe are shown in Figure 11-15. Operation and maintenance costs are shown in Figure 11-16.

Capital Costs

$$Y = 4{,}500$$

where
Y = \$
X = gallons of brine per day
Range: 2,000 gpd $\le X <$ 150,000 gpd

$$Y = 4{,}600 + (1.9 \times 10^{-3})(X)$$

where
Y = \$
X = gallons of brine per day
Range: 150,000 gpd $\le X \le$ 25,000,000 gpd

Operation and Maintenance

$$Y = 1{,}000 + [(1.83 \times 10^{-3})(365)(X)] + [(0.12)(365)(Z)]$$

$$Y = [\text{operating cost}] + [\text{POTW volume charge}] + [\text{POTW strength charge}]$$

where
Y = \$/year
X = gallons of brine per day
Z = TSS pounds per day in excess of 300 ppm
(1.83×10^{-3} = POTW volume charge)
(0.12 = TSS surcharge)
Range: 2,000 gpd $\le X \le$ 25,000,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = \$/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.6.5.2 1,000 Feet of Discharge Pipe

Capital costs for 1,000 feet of discharge piping are shown in Figure 11-17 and operation and maintenance costs are shown in Figure 11-18.

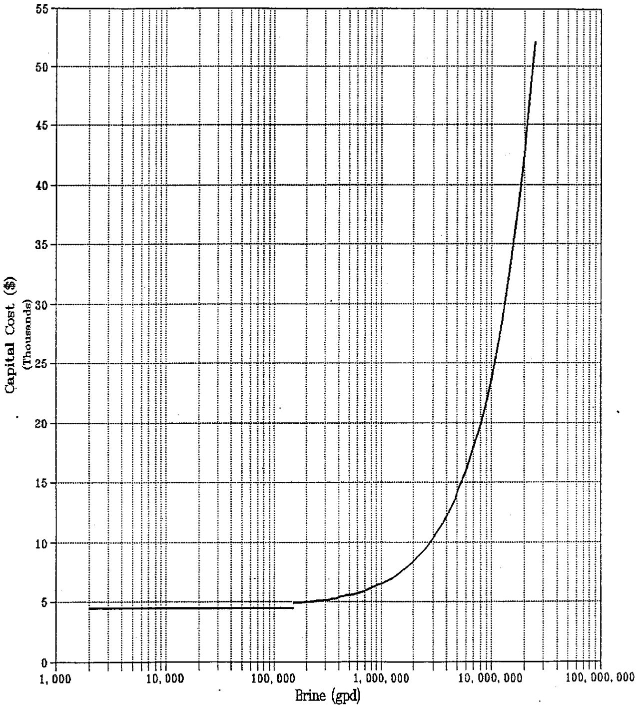


Figure 11-15. Capital costs for 500 feet of discharge pipe.

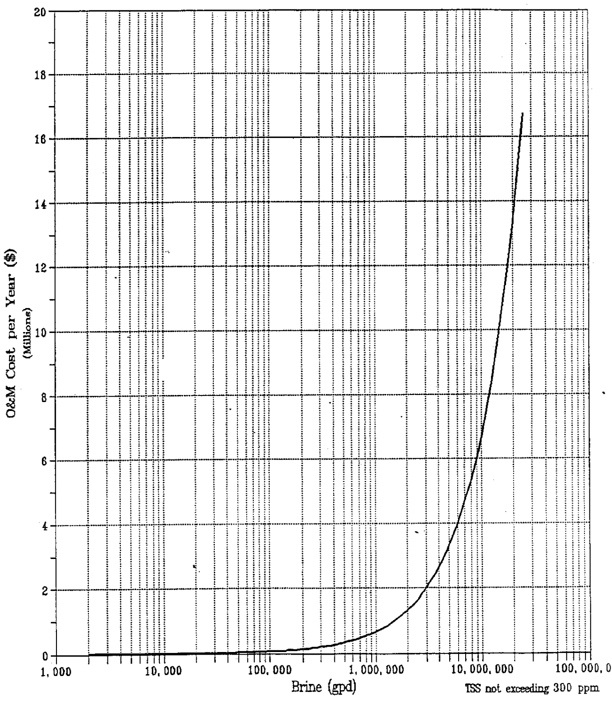


Figure 11-16. O&M costs for 500 feet of discharge pipe.

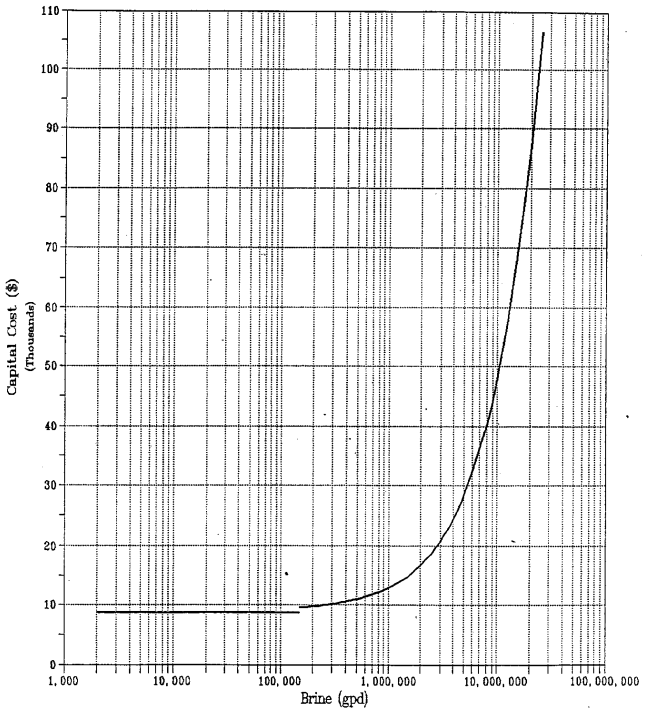


Figure 11-17. Capital costs for 1,000 feet of discharge pipe.

Capital Costs

$$Y = 8{,}700$$

where
Y = $
X = gallons of brine per day
Range: 2,000 gpd ≤ X < 150,000 gpd

$$Y = 9{,}000 + (3.9 \times 10^{-3})(X)$$

where
Y = $
X = gallons of brine per day
Range: 150,000 gpd ≤ X ≤ 25,000,000 gpd

Operation and Maintenance

$$Y = 1{,}000 + [(1.83 \times 10^{-3})(365)(X)] + [(0.12)(365)(Z)]$$

$$Y = [\text{operating cost}] + [\text{POTW volume charge}] + [\text{POTW strength charge}]$$

where
Y = $/year
X = gallons of brine per day
Z = TSS pounds per day in excess of 300 ppm
(1.83×10^{-3} = POTW volume charge)
(0.12 = TSS surcharge)
Range: 2,000 gpd ≤ X ≤ 25,000,000 gpd

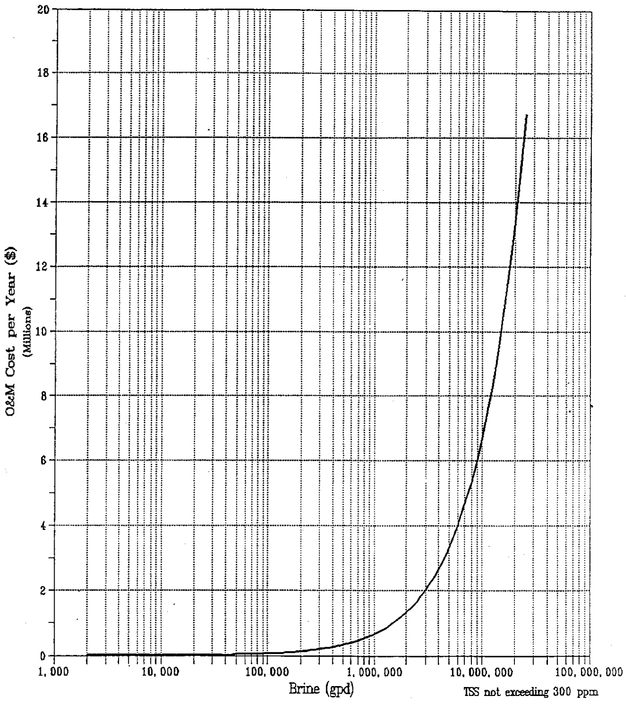


Figure 11-18. O&M costs for 1,000 feet of discharge pipe.

Total Annual Costs

$$Y = [CAP(CRF)] + O\&M$$

where
Y = $/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.6.5.3 500 Feet of Discharge Pipe With a Storage Lagoon

Capital and operation and maintenance cost curves for 500 feet of discharge pipe with a storage lagoon are shown in Figures 11-19 and 11-20.

Capital Costs

$$Y = [44.3(X^{0.56})] + [0.5(Z)]$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = $
X = gallons of brine per day
Z = land cost in dollars per acre (e.g., $10,000/acre)
Range: 25,000 gpd ≤ X ≤ 750,000 gpd

$$Y = [1.13(X^{0.84})] + [(2.47 \times 10^{-5})(X^{0.74})(Z)]$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = $
X = gallons of brine per day

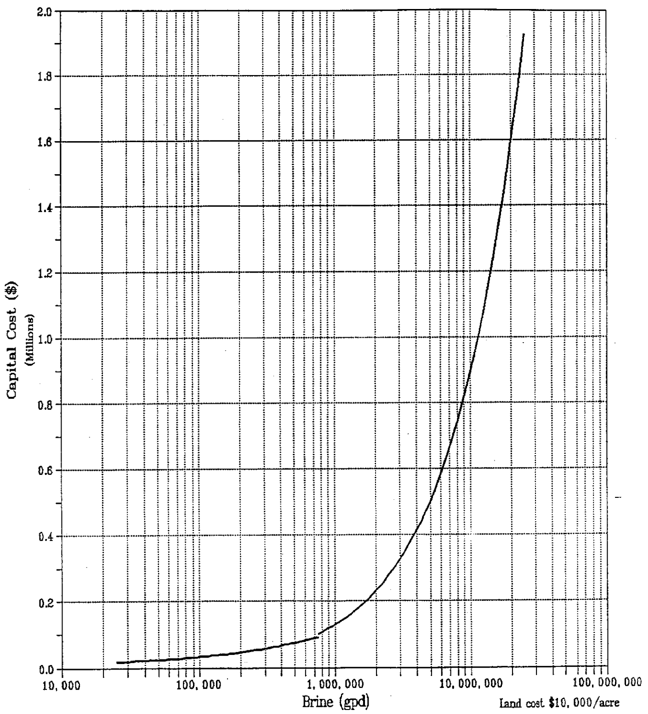


Figure 11-19. Capital costs for 500 feet of discharge pipe with storage lagoon.

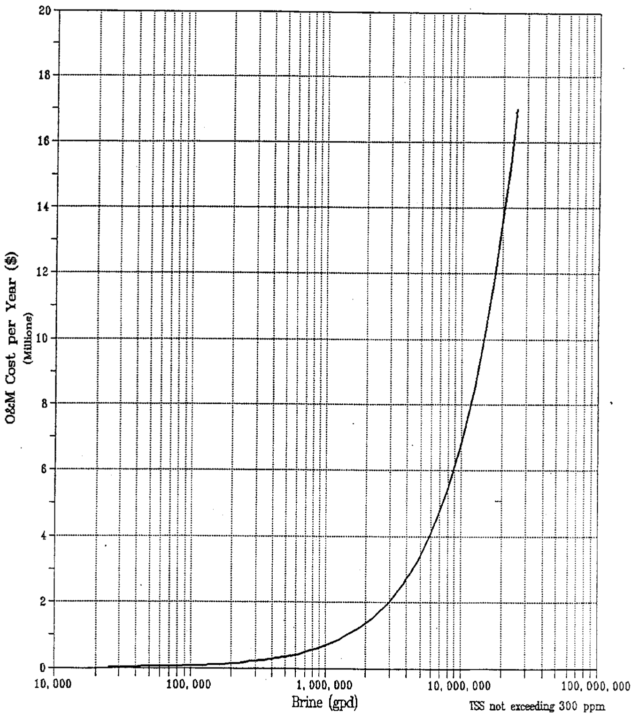


Figure 11-20. O&M costs for 500 feet of discharge pipe with storage lagoon.

Z = land cost in dollars per acre (e.g., $10,000/acre)
Range: 750,000 gpd < X ≤ 25,000,000 gpd

Operation and Maintenance

$$Y = [3{,}500 + (2.25 \times 10^{-2})(X)] + [(1.83 \times 10^{-3})(365)(X)] + [(0.12)(365)(Z)]$$

Y = [operating cost] + [POTW volume charge] + [POTW strength charge]

where
Y = $/year
X = gallons of brine per day
Z = TSS pounds per day in excess of 300 ppm
(1.83 x 10^{-3} = POTW volume charge)
(0.12 = TSS surcharge)
Range: 25,000 gpd ≤ X ≤ 500,000 gpd

$$Y = [0.30(X^{0.81})] + [(1.83 \times 10^{-3})(365)(X)] + [(0.12)(365)(Z)]$$

Y = [operating cost] + [POTW volume charge] + [POTW strength charge]

where
Y = $/year
X = gallons of brine per day
Z = TSS pounds per day in excess of 300 ppm
(1.83 x 10^{-3} = POTW volume charge)
(0.12 = TSS surcharge)
Range: 500,000 gpd < X ≤ 25,000,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = $/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.6.5.4 1,000 Feet of Discharge Pipe With a Storage Lagoon

Capital and operation and maintenance cost curves for 1,000 feet of discharge pipeline with a storage lagoon are shown in Figures 11-21 and 11-22.

Capital Cost

$$Y = [19{,}500 + (0.11)(X)] + [0.5(Z)]$$

Y = [equipment cost] + [land cost]

where
Y = $
X = gallons of brine per day
Z = land cost in dollars per acre (e.g., $10,000/acre)
Range: 25,000 gpd ≤ X ≤ 750,000 gpd

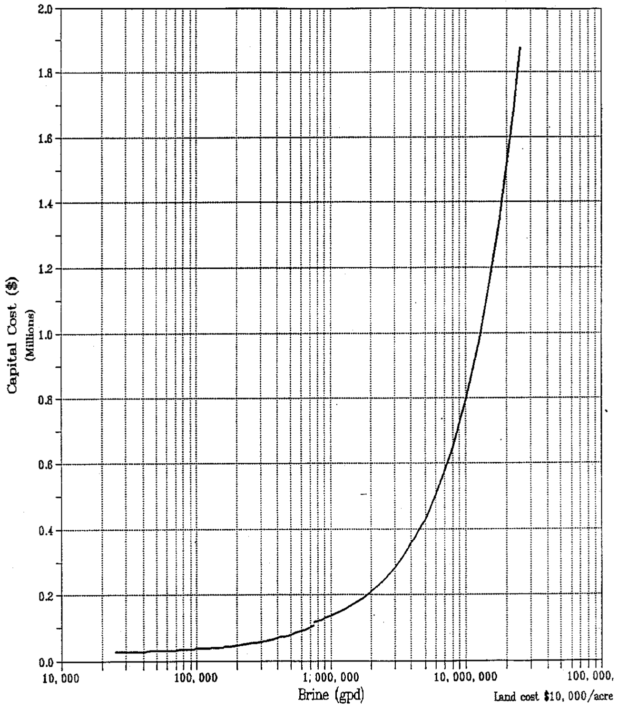


Figure 11-21. Capital costs for 1,000 feet of discharge pipe with storage lagoon.

$$Y = [60{,}200 + (6.97 \times 10^{-2})(X)] + [(2.47 \times 10^{-5})(X^{0.74})(Z)]$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = $
X = gallons of brine per day
Z = land cost in dollars per acre (e.g., $10,000/acre)
Range: 750,000 gpd < X ≤ 25,000,000 gpd

Operation and Maintenance

$$Y = [3{,}500 + (2.25 \times 10^{-2})(X)] + [(1.83 \times 10^{-3})(365)(X)] + [(0.12)(365)(Z)]$$

$$Y = [\text{operating cost}] + [\text{POTW volume charge}] + [\text{POTW strength charge}]$$

where
Y = $/year
X = gallons of brine per day
Z = TSS pounds per day in excess of 300 ppm
(1.83 x 10^{-3} = POTW volume charge)
(0.12 = TSS surcharge)
Range: 25,000 gpd ≤ X ≤ 500,000 gpd

$$Y = [0.30(X^{0.81})] + [(1.83 \times 10^{-3})(365)(X)] + [(0.12)(365)(Z)]$$

$$Y = [\text{operating cost}] + [\text{POTW volume charge}] + [\text{POTW strength charge}]$$

where
Y = $/year

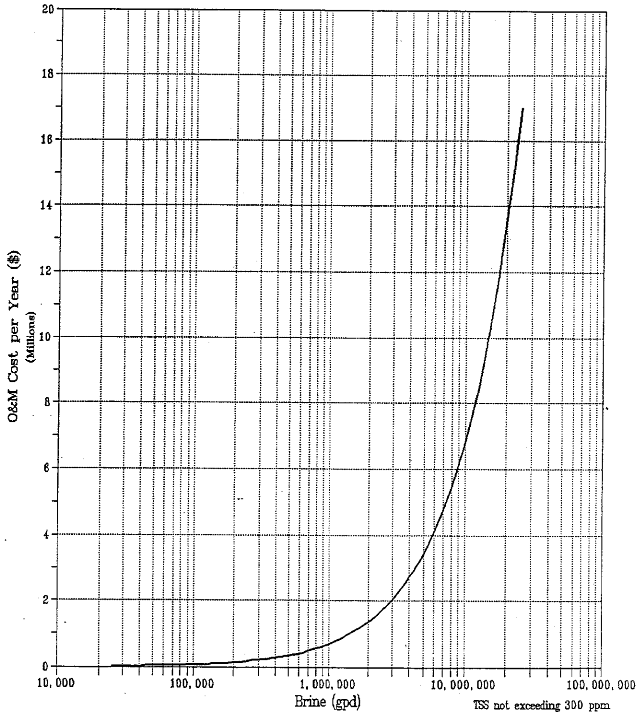


Figure 11-22. O&M costs for 1,000 feet of discharge pipe with storage lagoon.

X = gallons of brine per day
Z = TSS pounds per day in excess of 300 ppm
(1.83 x 10^{-3} = POTW volume charge)
(0.12 = TSS surcharge)
Range: 500,000 gpd < X ≤ 25,000,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = $/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.7 Direct Discharge

This section presents the design assumptions, capital components, and the operation and maintenance components used to estimate the costs for direct discharge into surface water.

11.7.1 Design Assumptions

- Residuals flow rates range from 2,000 to 25,000,000 gallons per day depending on the water treatment technology.
- The discharge point is 500 or 1,000 feet from the water treatment system.

- The residuals stream flows by gravity or under pressure from the treatment process to the discharge point.
- Two-inch minimum diameter pipe is used to prevent clogging. Pipe diameter ranges from 2 to 24 inches.
- PVC or reinforced concrete piping is used.
- A 1- to 2-day storage lagoon is optional.

11.7.2 Capital Components

The capital components for each direct discharge system are piping and fittings, trenching, and land clearing.

In addition, a pump, storage lagoon, electrical, and instrumentation are added for those facilities electing to store residuals prior to direct discharge.

Table 11-12 indicates the residuals flow rate, pipe diameters, and storage lagoon capacities used to develop the capital cost equation for direct discharge.

Table 11-12. Capital Cost Equation Determinants, Direct Discharge (U.S. EPA, 1993c)

Residuals Flow Rate (gpd)	Pipe Diameter (in.)	Lagoon Capacity (gal)
30,000	2	50,000
67,500	2	115,000
162,500	3	330,000
500,000	4	750,000
750,000	6	1,125,000
1,000,000	6	1,500,000
10,000,000	24	15,000,000
25,000,000	36	37,500,000

11.7.3 Operation and Maintenance Components

The operation and maintenance components for each direct discharge system include: labor and supervision, maintenance labor and materials, electricity, insurance, and general and administration.

11.7.4 Cost Components Excluded

The cost components not included in the cost equations for direct discharge systems are: NPDES permit application and monitoring costs, and land costs. It is assumed that the water treatment facility would be able to gain access for piping via an easement.

11.7.5 Cost Equations for Direct Discharge

The cost equations for the capital components and operation and maintenance were developed by estimating the costs for eight different residuals flow rates. The capital cost equation calculates the total capital cost (i.e., installed capital plus indirect capital costs). The total annual cost is calculated from the capital and operation and maintenance costs obtained using the equations presented below.

11.7.5.1 500 Feet of Discharge Pipe

Capital Costs

$$Y = 4{,}500$$

where
Y = $
X = gallons of brine per day
Range: 2,000 gpd ≤ X < 150,000 gpd

$$Y = 4{,}600 + [(1.9 \times 10^{-3})(X)]$$

where
Y = $
X = gallons of brine per day
Range: 150,000 gpd ≤ X ≤ 25,000,000 gpd

Operation and Maintenance

$$Y = 1{,}000$$

where
Y = $/year
X = gallons of brine per day
Range: 2,000 gpd ≤ X ≤ 25,000,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = $/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.7.5.2 1,000 Feet of Discharge Pipe

Capital Cost

$$Y = 8{,}700$$

where
Y = $
X = gallons of brine per day
Range: 2,000 gpd ≤ X < 150,000 gpd

$$Y = 9{,}000 + [(3.9 \times 10^{-3})(X)]$$

where
Y = $
X = gallons of brine per day
Range: 150,000 gpd ≤ X ≤ 25,000,000 gpd

Operation and Maintenance

$$Y = 1{,}000$$

where
Y = \$/year
X = gallons of brine per day
Range: 2,000 gpd ≤ X < 25,000,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = \$/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.7.5.3 500 Feet of Discharge Pipe With a Storage Lagoon

Capital Costs

$$Y = [44.3(X^{0.56})] + [0.5(Z)]$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = \$
X = gallons of brine per day
Z = land cost in dollars per acre (e.g., \$10,000/acre)
Range: 25,000 gpd ≤ X ≤ 750,000 gpd

$$Y = [1.13(X^{0.84})] + [(2.47 \times 10^{-5})(X^{0.74})(Z)]$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = \$
X = gallons of brine per day
Z = land cost in dollars per acre (e.g., \$10,000/acre)
Range: 750,000 gpd < X ≤ 25,000,000 gpd

Operation and Maintenance

$$Y = 3{,}500 + [(2.25 \times 10^{-2})(X)]$$

where
Y = \$/year
X = gallons of brine per day
Range: 25,000 gpd ≤ X ≤ 500,000 gpd

$$Y = 0.30(X^{0.81})$$

where
Y = \$/year
X = gallons of brine per day
Range: 500,000 gpd < X ≤ 25,000,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = \$/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.7.5.4 1,000 Feet of Discharge Pipe With a Storage Lagoon

Capital Cost

$$Y = [19{,}500 + (0.11)(X)] + [0.5(Z)]$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = \$
X = gallons of brine per day
Z = land cost in dollars per acre (e.g., \$10,000/acre)
Range: 25,000 gpd ≤ X ≤ 750,000 gpd

$$Y = [60{,}200 + (6.97 \times 10^{-2})(X)] + [(2.47 \times 10^{-5})(X^{0.74})(Z)]$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = \$
X = gallons of brine per day
Z = land cost in dollars per acre (e.g., \$10,000/acre)
Range: 750,000 gpd < X ≤ 25,000,000 gpd

Operation and Maintenance

$$Y = 3{,}500 + [(2.25 \times 10^{-2})(X)]$$

where
Y = \$/year
X = gallons of brine per day
Range: 25,000 gpd ≤ X ≤ 500,000 gpd

$$Y = 0.30(X^{0.81})$$

where
Y = \$/year
X = gallons of brine per day
Range: 500,000 gpd < X ≤ 25,000,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = \$/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.8 Land Application

11.8.1 Liquid Sludge Land Application Cost Components

This section presents the design assumptions, capital components, and the operation and maintenance components used to estimate the costs for the land application of liquid residuals.

11.8.1.1 Design Assumptions

- The storage lagoon is constructed with a synthetic membrane liner, overlaid by 12 inches of sand.
- The applied residuals consist of lime or alum residuals.
- The water treatment plant is 500 feet from the storage lagoon; the residuals are gravity-fed to the lagoon.
- The lagoons are sized for 6-month storage.
- The application field is cultivated farmland. The farmer uses a personally owned tractor and fuel to till the land following the residuals application.
  - *Application by truck:* The residuals are applied twice a year, spring and fall, in a 1-inch layer.
  - *Application by sprinkler:* The sprinkler system is a portable system with aboveground piping and a radius of 50 feet. The land application rate is 1/4 inch per day.

11.8.1.2 Capital Components

The capital components for land application consist of: storage lagoon, piping, land, electrical, and sprinkler system, when appropriate. Table 11-13 indicates the residuals volume and pond surface areas used to develop the capital cost equation for the land application systems.

Table 11-13. Capital Cost Equation Determinants, Liquid Residuals Land Application (U.S. EPA, 1993c)

Residuals Volume (gpd)	Pond Surface Area (acres)
2,000	0.27
10,000	0.92
20,000	1.26
60,000	2.64
100,000	3.72
500,000	10.97

11.8.1.3 Operation and Maintenance Components

The operation and maintenance components for land application include: electricity, labor and supervision, sludge removal from storage pond, maintenance labor and materials, insurance, general and administration, and transportation and land application fees, when appropriate.

11.8.1.4 Cost Components Excluded

Costs for incorporation of the liquid residuals into the soil, if necessary, are not included in the cost equations for liquid residuals.

11.8.1.5 Liquid Residuals Land Application Cost Equations and Cost Curves

The cost equations for the capital components and operation and maintenance were developed by estimating the costs for six different residuals flow rates. The capital cost equation calculates the total capital cost (i.e., installed capital plus indirect capital costs). Equipment costs and land costs are separate components in the capital cost equation to enable the user to exclude the cost for purchasing land if it is not necessary. The total annual cost is calculated based on the capital and operation and maintenance costs obtained using the equations presented below.

Sprinkler System

A capital cost curve for liquid residuals land application using a sprinkler system is shown in Figure 11-23. Operation and maintenance costs are shown in Figure 11-24.

Capital Cost

$$Y = [120(X^{0.78})] + [(2.06 \times 10^{-3})(X^{0.66})(Z)] + [(2.87 \times 10^{-4})(X)(Z)]$$

$$Y = [\text{equipment cost}] + [\text{pond land cost}] + [\text{application field cost}]$$

where
Y = \$
X = gallons of residuals per day
Z = land cost in \$/acres (e.g., \$10,000/acre)
Range: 2,000 gpd $\leq X \leq$ 500,000 gpd

Operation and Maintenance

$$Y = 81.7(X^{0.56})$$

where
Y = \$/year
X = gallons of residuals per day
Range: 2,000 gpd $\leq X \leq$ 60,000 gpd

$$Y = 8.63(X^{0.76})$$

where
Y = \$/year
X = gallons of residuals per day
Range: 60,000 gpd $< X \leq$ 500,000 gpd

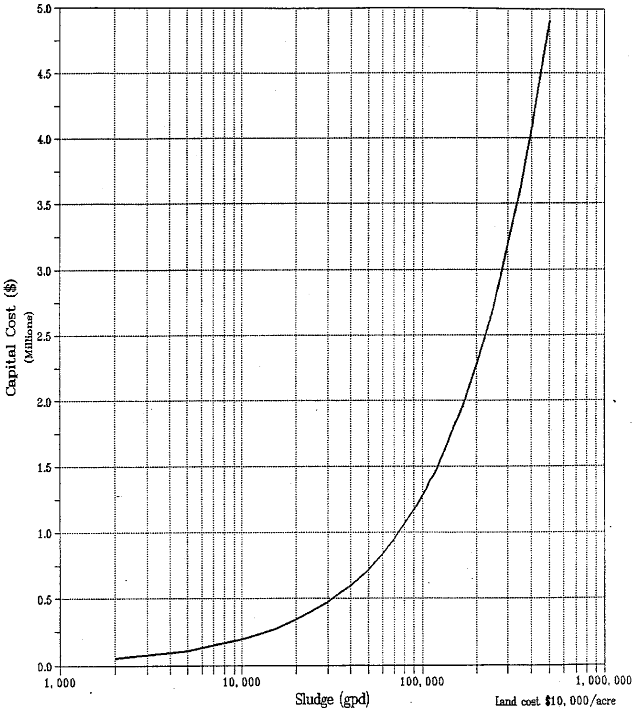


Figure 11-23. Capital costs for liquid sludge land application.

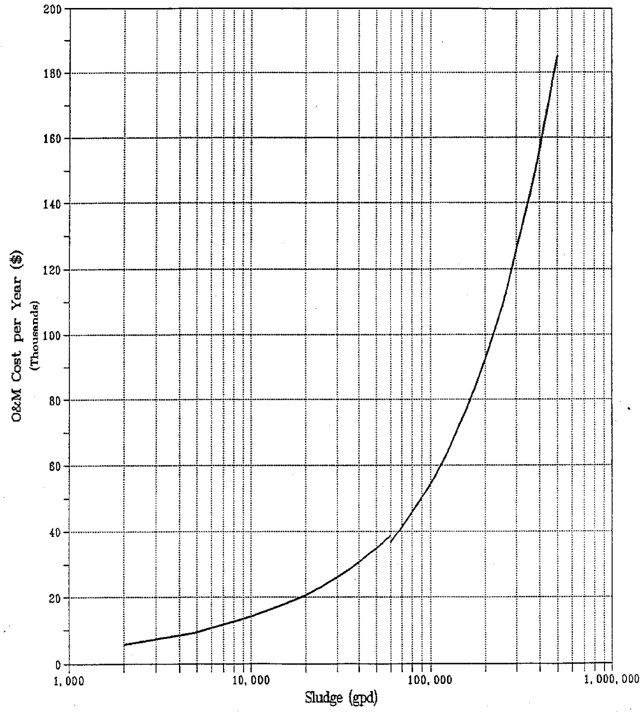


Figure 11-24. O&M costs for liquid sludge land application.

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = \$/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

Trucking System

Capital and operation and maintenance cost curves for liquid residuals land application with a residuals application truck are shown in Figures 11-25 and 11-26, respectively.

Capital Cost

$$Y = [104(X^{0.78})] + [(2.06 \times 10^{-3})(X^{0.66})(Z)]$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = \$
X = gallons of residuals per day
Z = land cost in \$/acre (e.g., \$10,000/acre)
Range: 2,000 gpd ≤ X ≤ 100,000 gpd

Operation and Maintenance

$$Y = 18.9(X^{0.96})$$

where
Y = \$/year
X = gallons of residuals per day
Range: 2,000 gpd ≤ X ≤ 100,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = \$/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

11.8.2 Dewatered Residuals Land Application Cost Components

11.8.2.1 Design Assumptions

- The residuals are stockpiled on site.
- The residuals are transported off site for land application.
- A front-end loader is used to load the residuals for transport. The front-end loader is owned by the WTP.
- The application field is within 25 miles of the plant.
- The application field is agricultural. The farmer growing crops on the field uses a personally owned tractor and fuel to till the residuals into the soil.
- The application rate is 2 dry tons of residuals per acre.

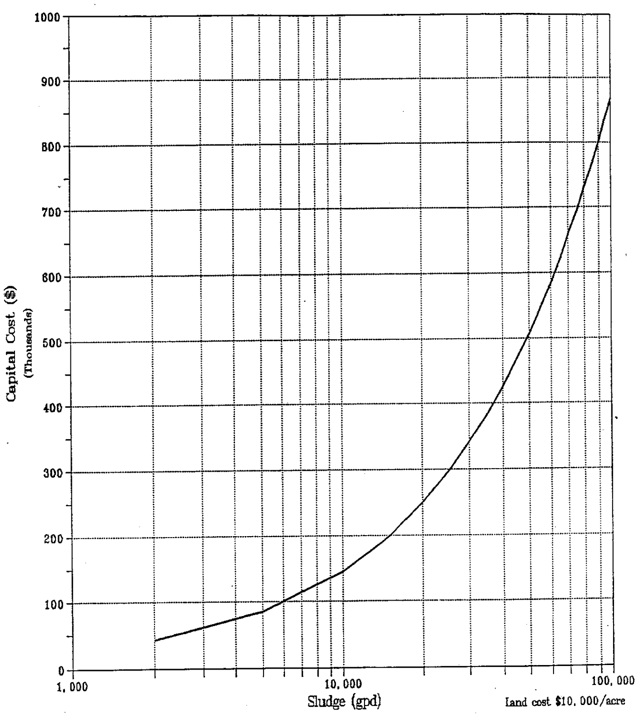


Figure 11-25. Capital costs for trucking system.

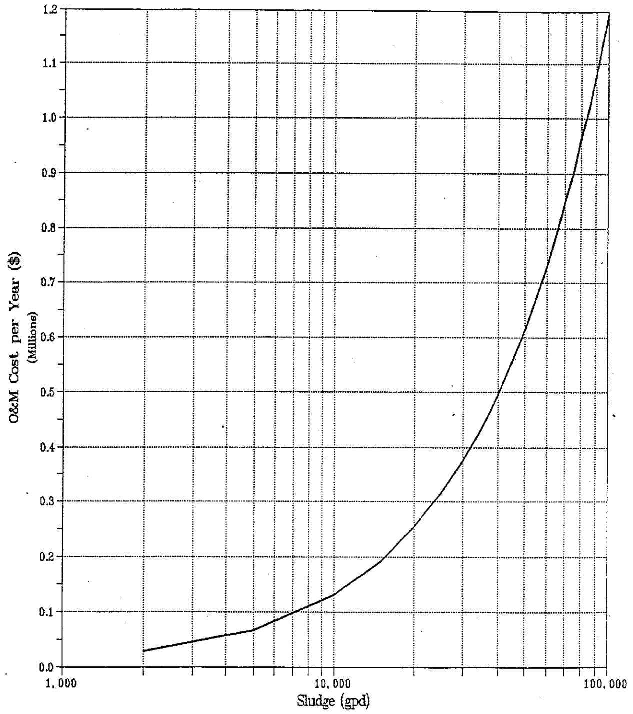


Figure 11-26. O&M costs for trucking system.

11.8.2.2 Capital Components

The capital components for dewatered residuals land application consist of: land for residuals stockpile, and land clearing. Table 11-14 indicates the residuals volume and storage pile surface area used to develop the cost equations for the land application of dewatered residuals.

Table 11-14. Capital Cost Equation Determinants, Dewatered Residuals Land Application (U.S. EPA, 1993c)

Residuals Volume (gpd)	Storage Pile Surface Area (acres)
2,000	0.34
10,000	2.2
20,000	4.5
60,000	9.2

11.8.2.3 Operation and Maintenance Components

The operation and maintenance components for each dewatered residuals land application include: labor and supervision, residuals loading, and transportation and land application fees.

11.8.2.4 Cost Components Excluded

Costs for incorporation of the dewatered residuals into the soil, if necessary, are not included in the cost equations for dewatered residuals.

11.8.2.5 Dewatered Residuals Land Application Cost Equations and Cost Curves

The cost equations for the capital components and operation and maintenance were developed by estimating the costs for four different residuals flow rates. The capital cost equation calculates the total capital cost (i.e., installed capital plus indirect capital costs). Equipment costs and land costs are separate components in the capital cost equation to enable the user to exclude the cost for purchasing land if it is not necessary. The total annual cost is calculated based on the capital and operation and maintenance costs obtained using the equations presented below. Capital and operation and maintenance cost curves for dewatered residuals land application are shown in Figures 11-27 and 11-28, respectively.

Capital Cost

$$Y = [(111)(X^{0.50}) - 4{,}700] + [0.5(Z)]$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = \$
X = gallons of residuals per day
Z = land cost in \$/acre (e.g., \$10,000/acre)
Range: 2,000 gpd $\leq X <$ 2,500 gpd

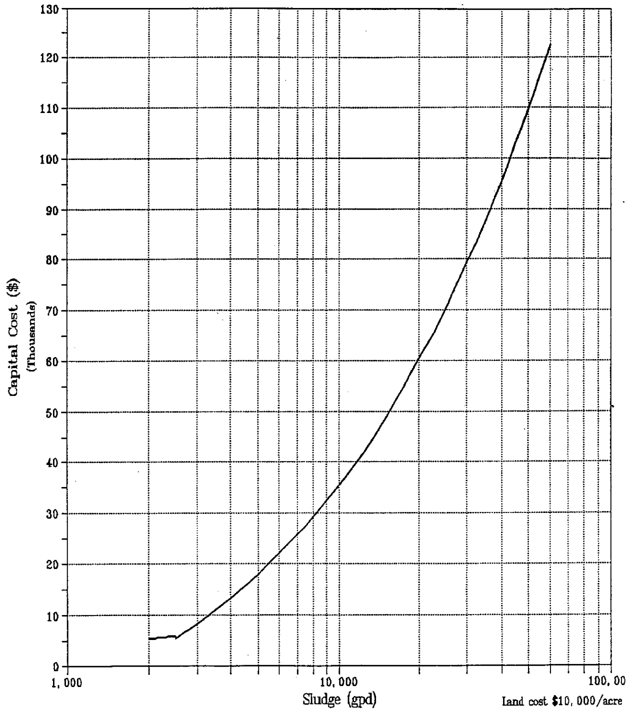


Figure 11-27. Capital costs for dewatered sludge land application.

$$Y = [(111)(X^{0.50}) - 4{,}700] + \{[(4.9 \times 10^{-2})(X^{0.50}) - 2](Z)\}$$

$$Y = [\text{equipment cost}] + [\text{land cost}]$$

where
Y = $
X = gallons of residuals per day
Z = land cost in $/acre (e.g., $10,000/acre)
Range: 2,500 gpd ≤ X < 60,000 gpd

Operation and Maintenance

$$Y = 800 + 28.2(X)$$

where
Y = $/year
X = gallons of residuals per day
Range: 2,000 gpd ≤ X ≤ 60,000 gpd

Total Annual Cost

$$Y = [CAP(CRF)] + O\&M$$

where
Y = $/year
CAP = capital cost
CRF = capital recovery factor (e.g., 0.1175)
O&M = operation and maintenance cost

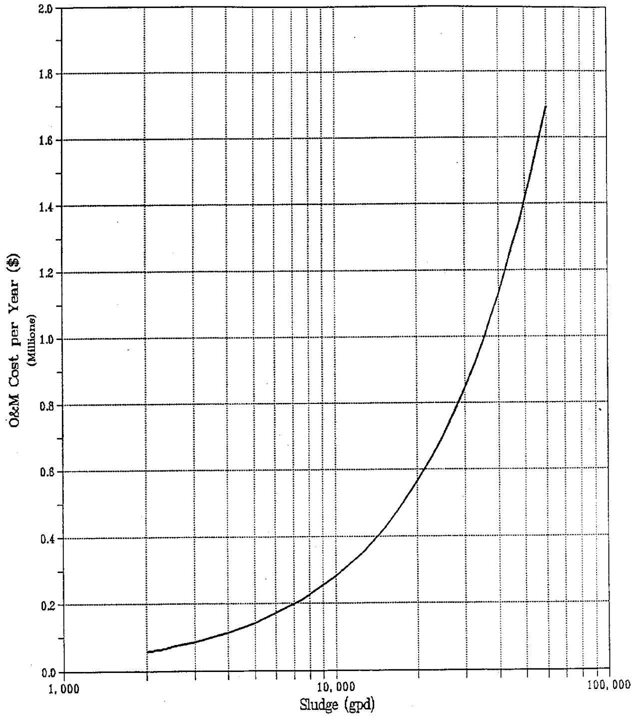


Figure 11-28. O&M costs for dewatered sludge land application.

11.9 Nonhazardous Waste Landfill

11.9.1 Off-Site Nonhazardous Waste Landfill Cost Components

This section presents the design assumptions used to estimate the costs for offsite nonhazardous waste landfills.

11.9.1.1 Design Assumptions

- A commercial nonhazardous waste landfill is used for residuals disposal.
- Transportation distance varies from 5 to 50 miles.
- There is no economy of scale for large waste volumes.
- All wastes pass the Paint Filter Liquids Test.
- The waste does not exhibit the characteristics of ignitability, corrosivity, reactivity, or toxicity.

11.9.1.2 Cost Components

The cost components for nonhazardous waste landfill consist of commercial nonhazardous waste landfill tipping fee and transportation fees.

11.9.1.3 Offsite Nonhazardous Waste Landfill Cost Equation and Cost Curve

The total operation and maintenance cost is based on the disposal and transportation costs shown in the equation presented below. An operation and maintenance cost curve for offsite nonhazardous waste landfill with transportation distances of 5 miles and 50 miles is shown in Figure 11-29.

Total Annual Cost

$$Y = [35(X)] + [(2.48 + 0.16Z)(X)]$$

$$Y = [\text{disposal}] + [\text{transportation}]$$

where
Y = \$/year
X = tons of residuals requiring disposal/year
Z = transportation distance from 5 to 50 miles

11.9.2 Onsite Nonhazardous Waste Landfill Cost Components

This section presents the design assumptions used to estimate the costs for onsite nonhazardous waste landfills, the capital cost components, and the operation and maintenance components.

11.9.2.1 Design Assumptions

- The landfill has a 20-year operating life.
- The landfill is a combination fill (i.e., a design combining below and above grade fills).
- The landfill containment system consists of 2 feet of clay, a 30 mil HDPE liner, 1 foot of sand with a leachate collection system, a geotextile filter fabric, and 1 foot of native soil fill.
- The intermediate cover consists of slope and earthfill soils.
- A ground-water monitoring system is installed and is sampled biannually.
- The final cover consists of 1 foot of native soil fill, a geotextile support fabric, a 30 mil PVC liner, 1 foot of sand with drain tiles, a geotextile filter fabric, and 1.5 feet of topsoil.
- The postclosure care period is 30 years.

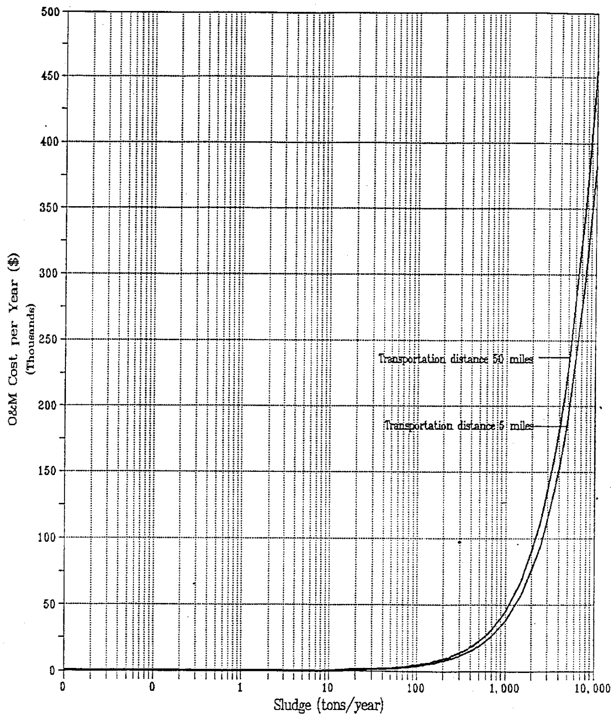


Figure 11-29. O&M costs for off-site nonhazardous waste landfill.

11.9.2.2 Capital Components

The capital components for each landfill consist of land, land clearing, landfill excavation, composite (clay and synthetic) liner, leachate collection system, ground-water monitoring system, equipment storage and maintenance buildings, visual screening berm, bulldozer, truck, and inspection, testing and quality assurance.

11.9.2.3 Operation and Maintenance Components

The operation and maintenance components for each landfill include labor and supervision, maintenance labor and materials, intermediate cover, ground-water monitoring, leachate collection and treatment, and utilities, including electricity and water.

11.9.2.4 Closure Components

The closure components for each landfill include final cover system (including drain tiles), revegetation, and inspection, testing, and quality assurance.

11.9.2.5 Postclosure Components

The postclosure components for each landfill include ground-water monitoring, leachate collection and treatment, landscape maintenance, slope maintenance, and annual inspection.

11.9.2.6 Cost Components Excluded

Siting and permitting costs are not included in the cost estimates. Regulatory requirements for RCRA-regulated Subtitle D landfills vary from state to state. Permitting costs will vary based on the size of the landfill and the local governing jurisdiction.

11.9.2.7 Onsite Nonhazardous Waste Landfill Cost Equations and Cost Curves

The capital cost, operation and maintenance cost, closure cost, and postclosure cost equations for onsite nonhazardous waste landfills are presented below. A capital cost curve is shown in Figure 11-30. An operation and maintenance cost curve is presented in Figure 11-31. Figure 11-32 shows a closure cost curve, and Figure 11-33 an annual postclosure cost curve.

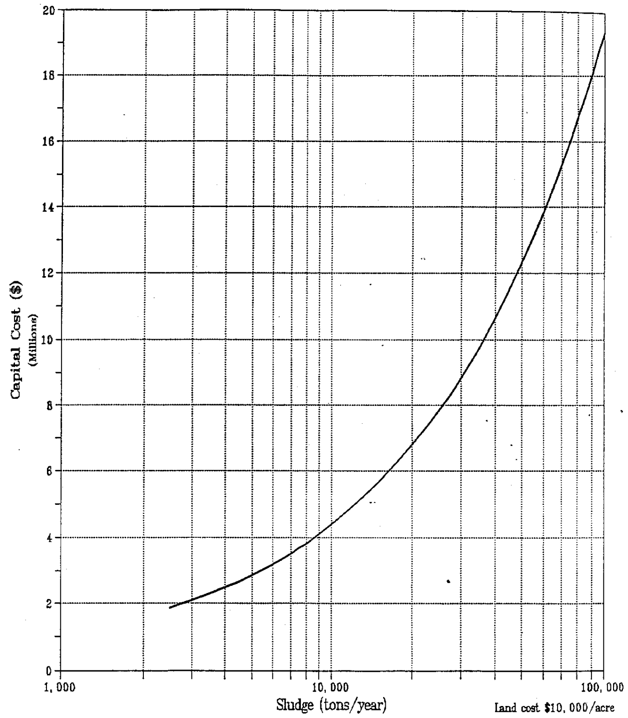


Figure 11-30. Capital costs for onsite nonhazardous waste landfill.

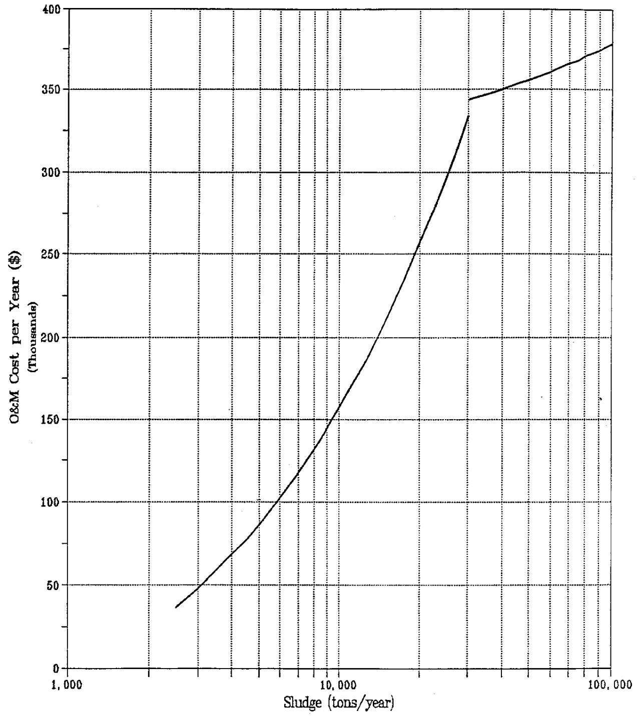


Figure 11-31. O&M costs for onsite nonhazardous waste landfill.

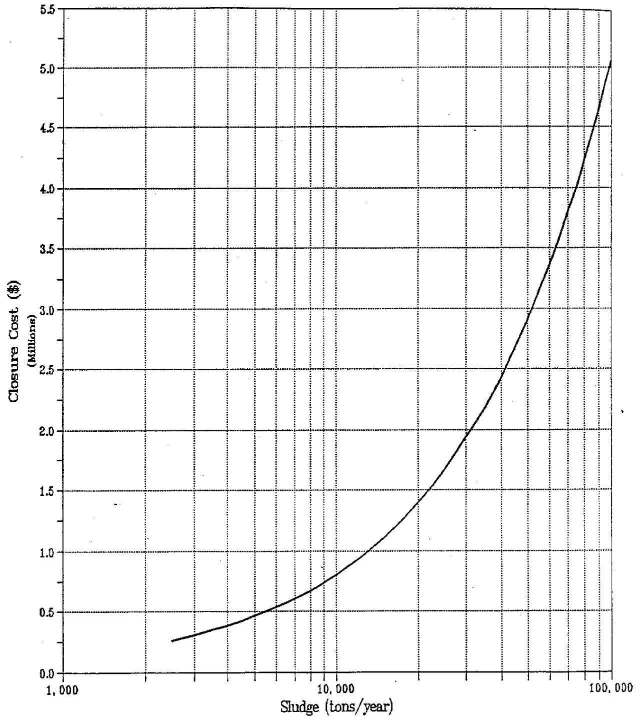


Figure 11-32. Closure costs for onsite nonhazardous waste landfill.

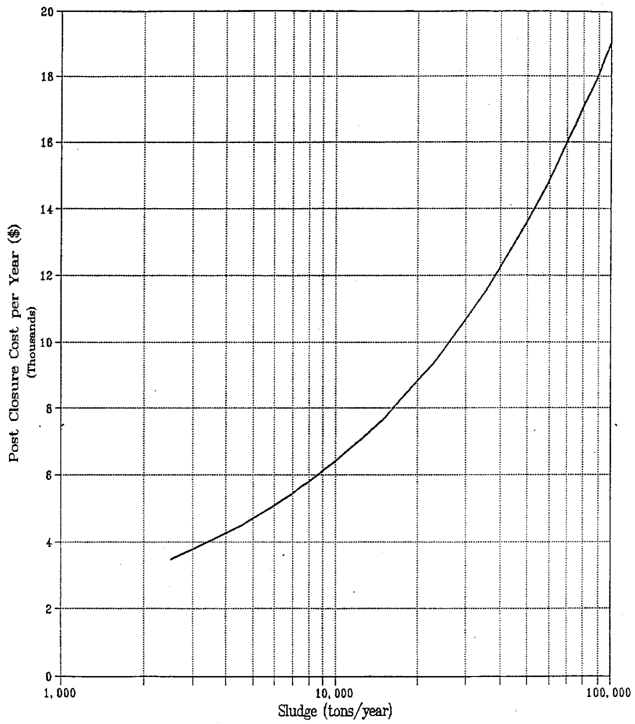


Figure 11-33. Postclosure costs for onsite nonhazardous waste landfill.

Capital Costs

$$Y = [8{,}079(X^{0.67})] + \{[20.4 + 0.33(X^{0.50})](Z)\}$$

$$Y = [\text{landfill cost}] + [\text{land cost}]$$

where
Y = \$
X = tons of residuals requiring disposal per year
Z = land cost in \$/acre (e.g., \$10,000/acre)
Range: 2,500 TPY $\leq X \leq$ 100,000 TPY

Operation and Maintenance

$$Y = [2{,}411(X^{0.50})] - 83{,}900$$

where
Y = \$/year
X = tons of residuals requiring disposal per year
Range: 2,500 TPY $\leq X <$ 30,000 TPY

$$Y = [246(X^{0.50})] + 300{,}900$$

where
Y = \$/year
X = tons of residuals requiring disposal per year
Range: 30,000 $\leq X \leq$ 100,000 TPY

Closure

$$Y = 506(X^{0.80})$$

where
Y = \$
X = tons of residuals requiring disposal per year
Range: 2,500 TPY $\leq X \leq$ 100,000 TPY

Postclosure

$$Y = 600 + 58.2(X^{0.5})$$

where
Y = \$/year
X = tons of residuals requiring disposal per year
Range: 2,500 TPY $\leq X \leq$ 100,000 TPY

Total Annual Cost

$$Y = [8{,}483(X^{0.50}) - 250{,}300] + \{[2.4 + 0.04(X^{0.50})](Z)\}$$

where
Y = \$/year
X = tons of residuals requiring disposal per year
Z = land cost in \$/acre (e.g., \$10,000/acre)
Range: 2,500 TPY $\leq X \leq$ 100,000 TPY

11.10 Hazardous Waste Landfill

This section presents the design assumptions, capital components, and the operation and maintenance components used to estimate the costs for hazardous waste landfills.

11.10.1 Design Assumptions

- A commercial hazardous waste landfill is used for residuals disposal.
- Transportation distance varies from 200 to 500 miles.
- There is no economy of scale for large waste volumes.
- All wastes pass the Paint Filter Liquids Test or, if they fail it, they are stabilized prior to disposal.
- The waste fails the TCLP test.
- Some wastes require stabilization prior to disposal to meet the Land Disposal Restrictions Treatment Standards for toxicity characteristic (TC) wastes.

11.10.2 Cost Components

The cost components for hazardous waste landfill consist of: hazardous waste landfill charges, stabilization charges, and transportation fees.

11.10.3 Cost Components Excluded

The cost components not included in the cost equations for hazardous waste landfills are: generator notification requirements, manifest requirements, and other RCRA requirements, as applicable.

11.10.4 Total Annual Cost Equation and Cost Curves

Operation and maintenance cost equations for hazardous waste disposal and for stabilization and hazardous waste disposal are presented below.

11.10.4.1 Hazardous Waste Disposal

An operation and maintenance cost curve for hazardous waste disposal is shown in Figure 11-34, based on transportation distances of 200 and 500 miles.

$$Y = [200\,X] + \{[7.9 + 0.22(Z)](X)\}$$

$$Y = [\text{disposal}] + [\text{transportation}]$$

where
Y = \$/year
X = tons of residuals requiring disposal per year
Z = transportation distance between 200 and 500 miles

11.10.4.2 Stabilization and Hazardous Waste Disposal

An operation and maintenance cost curve for stabilization and hazardous waste disposal is shown in Figure 11-35.

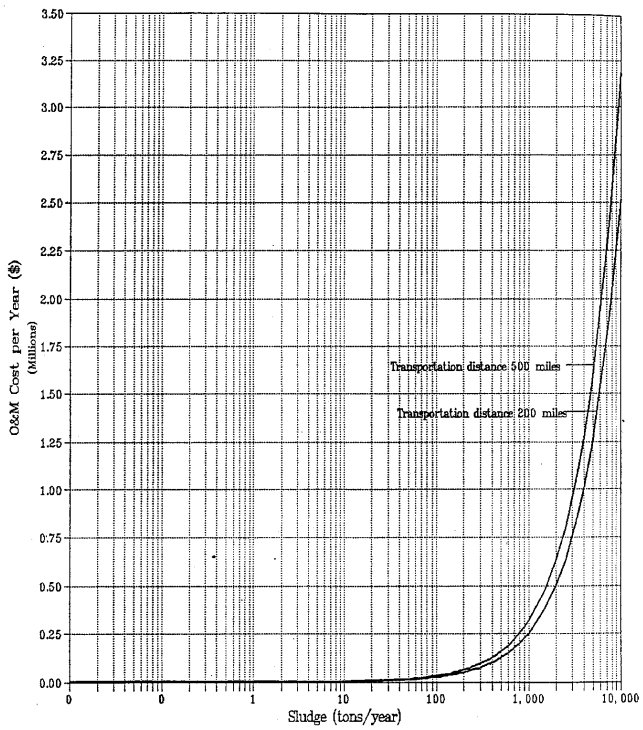


Figure 11-34. O&M costs for hazardous waste disposal.

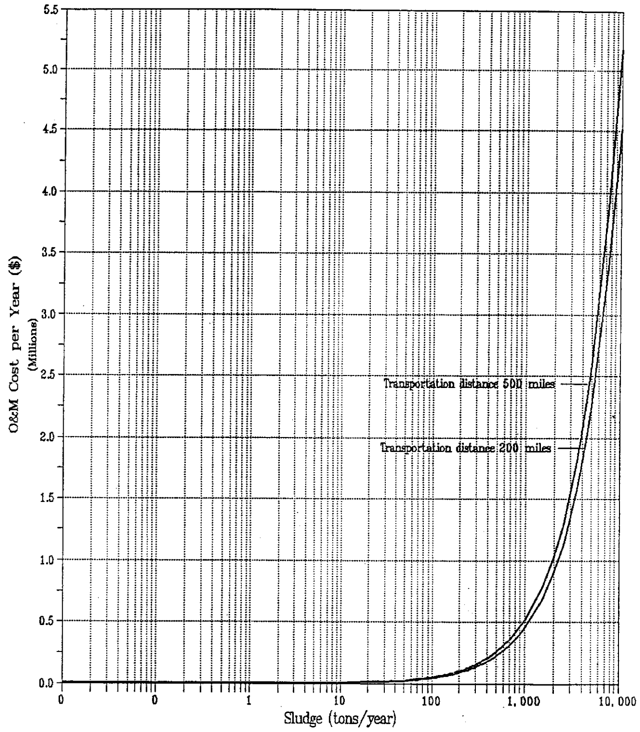


Figure 11-35. O&M costs for stabilization and hazardous waste disposal.

$$Y = [400(X)] + \{[7.9 + 0.22(Z)](X)\}$$

$$Y = [\text{stabilization/disposal}] + [\text{transportation}]$$

where
Y = $/year
X = tons of residuals requiring disposal per year
Z = transportation distance between 200 and 500 miles

11.11 Radioactive Waste Disposal

11.11.1 Low-Level Radioactive Waste Disposal

There are three low-level radioactive waste disposal sites operating in Nevada, South Carolina, and Washington, and one naturally occurring radioactive material (NORM) disposal site operating in Utah. Each low-level site bases waste disposal costs on several factors including state of origin of the waste, type of waste, and measured radioactivity at the container surface. All disposal facilities are part of a regional pricing compact. The Low-Level Radioactive Waste Policy Amendments Act (LLRWPAA) of 1985 authorized the formation of regional compacts and a system of incentives and penalties to ensure that states and compacts will be responsible for their own waste after January 1, 1993. States that are part of a compact with an operational waste disposal facility pay the lowest disposal costs while states located outside of the compacts with disposal facilities pay a disposal surcharge of $40 or $120 per cubic foot (approximately $300 to $900 per 55-gallon drum). The NORM disposal site is not part of a pricing compact and does not add surcharges.

11.11.2 Cost Information for Radioactive Waste Disposal

Radioactive waste disposal costs are not included within this chapter. These costs may be obtained from EPA's guideline document, *Suggested Guidelines for the Disposal of Drinking Water Treatment Wastes Containing Naturally Occurring Radionuclides* (U.S. EPA, 1990c). These guidelines provide cost-related information, recommendations, and a summary of existing regulations and criteria used by EPA and other agencies in addressing the disposal of radionuclides.

Chapter 12
Case Studies

12.1 Case Study 1: Disposal of Water Treatment Residuals From Pine Valley Water Treatment Plant, Colorado Springs, Colorado

The Pine Valley Water Treatment Plant, located on the southwest boundary of the U.S. Air Force Academy in Colorado Springs, Colorado, was originally constructed as a 42 million gallons per day (mgd) direct filtration plant and placed into service in July 1969. The location of the plant is shown in Figure 12-1. Expansion and improvements over the years have increased design and overload capacities to 56 and 84 mgd, respectively. The addition of sedimentation basins in 1987 converted the plant to a conventional treatment plant. Figure 12-2 illustrates the present site plan and facilities.

12.1.1 Facility Information

12.1.1.1 Treatment Processes

The Pine Valley Treatment Plant is a conventional facility using the following processes: chemical addition with in-line static mixing, coagulation-flocculation, sedimentation, and dual-media filtration, using two parallel treatment trains. An underground reservoir adjacent to the plant provides 10 mgd of treated water storage. Two of the eight filters are multimedia, i.e., containing coal, filter sand, and garnet sand. The sedimentation basins contain stainless steel lamella plates. The rapid mix and slow mix units, numbers 6 and 10 on Figure 12-3, have been taken out of service. Figure 12-3 is a schematic showing the direction of flow through the facility.

The chemicals applied at the Pine Valley Plant include chlorine, aluminum sulfate, sodium aluminate, coagulant aid (Nalco 8100), filter aid (Nalco 8170), and soda ash. Monthly average dosages for the chemicals used from 1987 through 1992 are shown in Appendix D.

12.1.1.2 Raw Water Supply

The Pine Valley Plant was constructed primarily to treat the Homestake Project water supply stored in Rampart Reservoir. Subsequent acquisition of additional western slope water supplies has provided other sources needing storage in the Rampart Reservoir. A 3.5-mile pressurized pipeline supplies raw water from the reservoir to the plant. Water quality data for the plant's influent raw water for 1987 through 1992 are shown in Appendix D.

12.1.1.3 Plant Facility and Flow

Raw water is conveyed to the plant through a pipeline that reduces from 42 to 36 to 30 inches (see Figure 12-2). Modulating valves are in place to control the flow of water to one of two parallel treatment trains designated as "A" or "B." Water enters a static mixer, continues through the flocculation and sedimentation basins, and then flows onto the filters, as shown in this schematic. Treated water leaving the plant is stored in an adjacent 10 million gallon (MG) covered reservoir until needed in the system. Filter backwash water is discharged into the washwater recovery basin. Sedimentation basin sludge and washdown water are discharged to the sediment collection pond (see Figure 12-4) and/or washwater recovery basin. Water from the recovery basin is systematically recycled to the plant headworks. Annual figures for the quantities of waste streams generated are presented in Table 12-1. Decant water from the sedimentation/collection pond will not be recycled until a pump station is constructed; one is currently scheduled for installation in 1996.

12.1.2 Residuals Management

The original plan for residuals management at the Pine Valley WTP included the use of sediment lagoons (sediment drying beds), decanting, and landfilling. Decanting operations were discontinued when the 1973 Colorado Water Quality Control Act was promulgated. Pine Valley, in service since 1969, did experience labor intensive operations and limited capacity to store their annual sediment production. In 1980, additional sediment handling facilities were installed to include sediment removal and transfer equipment in the washwater recovery basin and two sediment drying beds. The new facilities did not perform as planned, resulting in residuals containing too much water and incapable of being removed with loaders and dump trucks (see Figure 12-5).

Because the facilities did not produce the desired results, a long-range plan was developed for ultimate disposal. The plan, considered the most suitable and

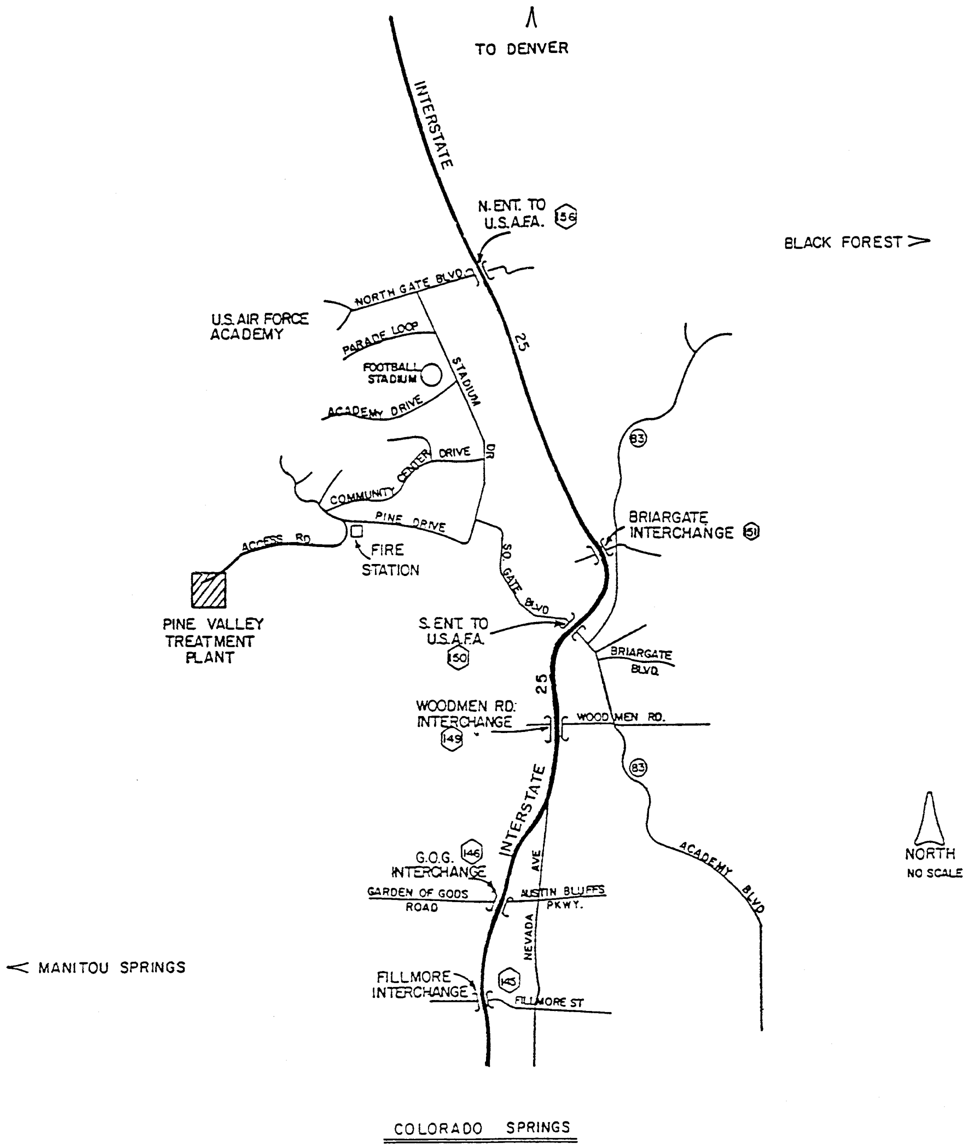


Figure 12-1. Locus map of Pine Valley WTP, Colorado Springs, CO (Colorado Springs Utilities, 1994).

economical method of sediment disposal, was to obtain a site just east of Pine Valley and construct new disposal facilities. The new sediment disposal facilities would receive sediment from the Pine Valley WTP and another proposed 150-mgd WTP. The new facilities, including a 175 acre-foot (acre-ft) sediment collection pond and two 3.5-acre-ft sediment drying ponds, were constructed and placed into service in 1991. The new 150-mgd WTP is scheduled for completion in 1996.

12.1.3 Residuals Handling Facilities and Operations

12.1.3.1 Facilities

Current residuals disposal facilities at Pine Valley consist of a 1.5-MG washwater recovery basin with sediment collection and transfer equipment, four sediment drying ponds (2 MG), and an offsite designated landfill (Hanna Ranch). Additional facilities include sharing of the 17 million-ft^3 sediment collection/holding basin, and the two 3.5-acre-ft sediment drying ponds. Equipment used to transfer sediment from the sediment drying ponds to the landfill area include a sludge pump, tractor, and 9,000-gallon tank trailer. Additional equipment is used at the landfill site for monitoring, landscaping, and erosion, dust, weed, and pest control.

12.1.3.2 Operations

Filter backwash water is discharged into the 1.5-MG washwater recovery basin. Water containing sediment, generated by washing down the flocculation and sedimentation basins, is transferred to the 128-MG sediment collection basin (see Figures 12-2 and 12-4). Decant from the recovery basin is pumped back to the plant

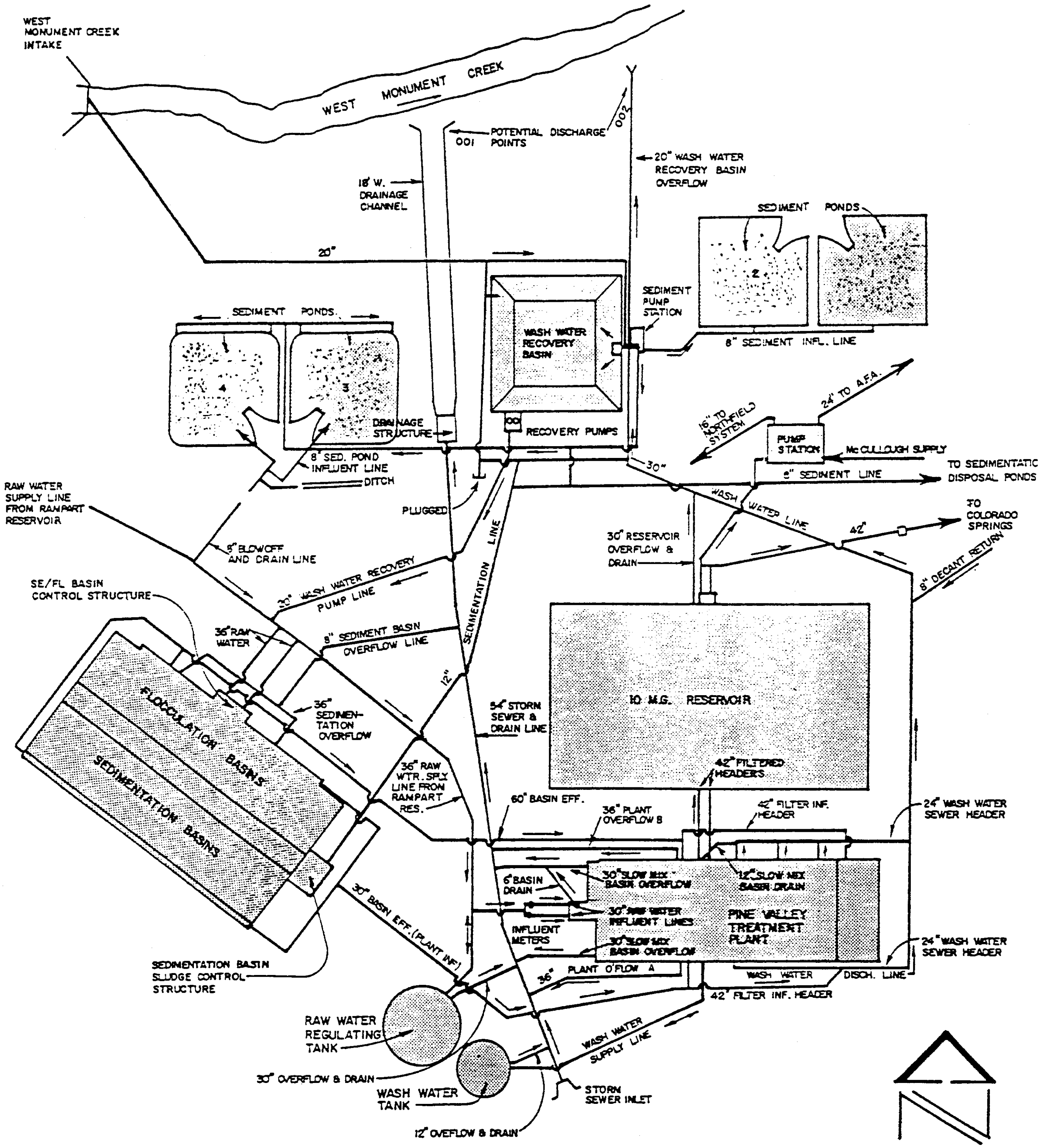


Figure 12-2. Schematic of Pine Valley WTP, Colorado Springs, CO (Colorado Springs Utilities, 1994).

Pine Valley Purification Plant - Flow Chart

1. Diversion dam (raw water intake)
2. 980,000 Gallon Regulating Tank
3. 36-Inch Raw Water Supply Line
4. Plant Divides Into A (south) and B (north) Halves
5. Control Valves
6. Rapid Mixers
7. Alum Feeders
8. Soda Ash Feeders
9. Chlorinators
10. Slow Mix Basins
11. Filter Influent Headers and Valves
12. Central Control Console
13. Diagrammatic Section Rapid Sand Filter
14. Cutaway View of Filter Bed
15. 42-Inch Filtered Water Header
16. 10 Million Gallon Filtered Water Reservoir (underground)
17. 48-Inch Transmission Main to City
18. Washwater Tank Supply
19. 400,000 Gallon Washwater Storage Tank
20. 24-Inch Washwater Waste Pipe
21. Washwater Recovery Basin and Pump Station
22. 20-Inch Recovery Basin Discharge Pipe (returns wastewater to raw water system)
23. Sludge Drying Beds

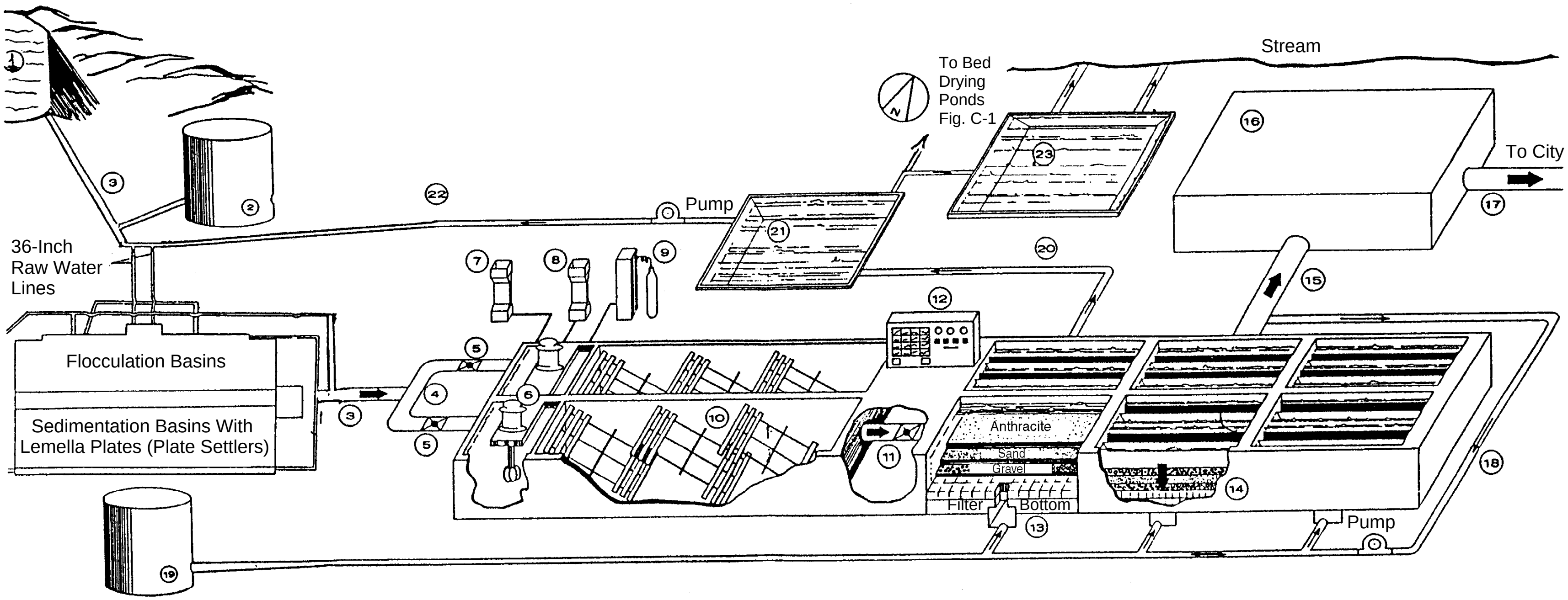


Figure 12-3. Flow chart of Pine Valley WTP, Colorado Springs, CO (Colorado Springs Utilities, 1994).

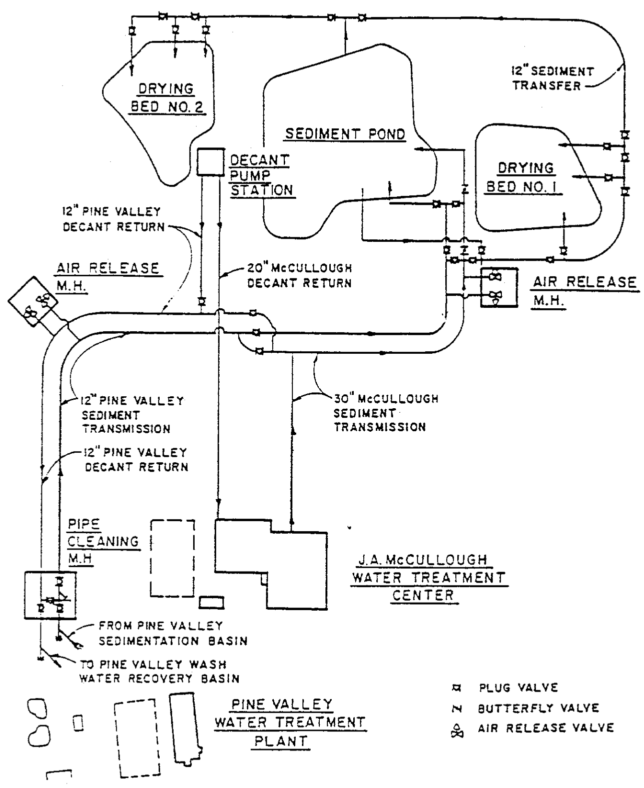


Figure 12-4. **Sediment and decant piping schematic, Pine Valley WTP, Colorado Springs, CO (Colorado Springs Utilities, 1994).**

Table 12-1. Water Treated and Used, Pine Valley WTP, Colorado Springs, CO (Pine Valley WTP, 1994)

	Water Treated	Wash-water	Percent Wash-water	Mainten-ance Water	Percent W/W & M/W	Recycled Water
1987	9,805.310	165.185	1.71	12.420	1.83	361.0409
1988	11,636.362	192.959	1.66	18.687	1.82	224.1929
1989	11,820.405	142.346	1.20	26.609	1.43	227.6053
1990	10,112.920	115.852	1.13	17.552	1.32	211.0325
1991	10,082.701	117.096	1.16	18.890	1.35	234.9224
1992	11,010.706	131.890	1.20	15.995	1.34	261.4732
Averages	10,744.743	144.555		18.359		253.3778

Note: Figures in million gallons.

headworks and retreated. The supernatant from the sediment collection basin will be pumped either to the recovery basin at Pine Valley or to the new WTP, when operational. The sediment that is retained in the recovery basin is systematically pumped and sprayed onto one of the four sediment drying ponds. Prior operations and storage capacities required that the ponds be cleaned annually. Present facilities will provide both Pine Valley and the new WTP with sufficient capacity to store their combined sediment production for a period of 15 to 20 years before removal is necessary.

12.1.4 Residuals Disposal

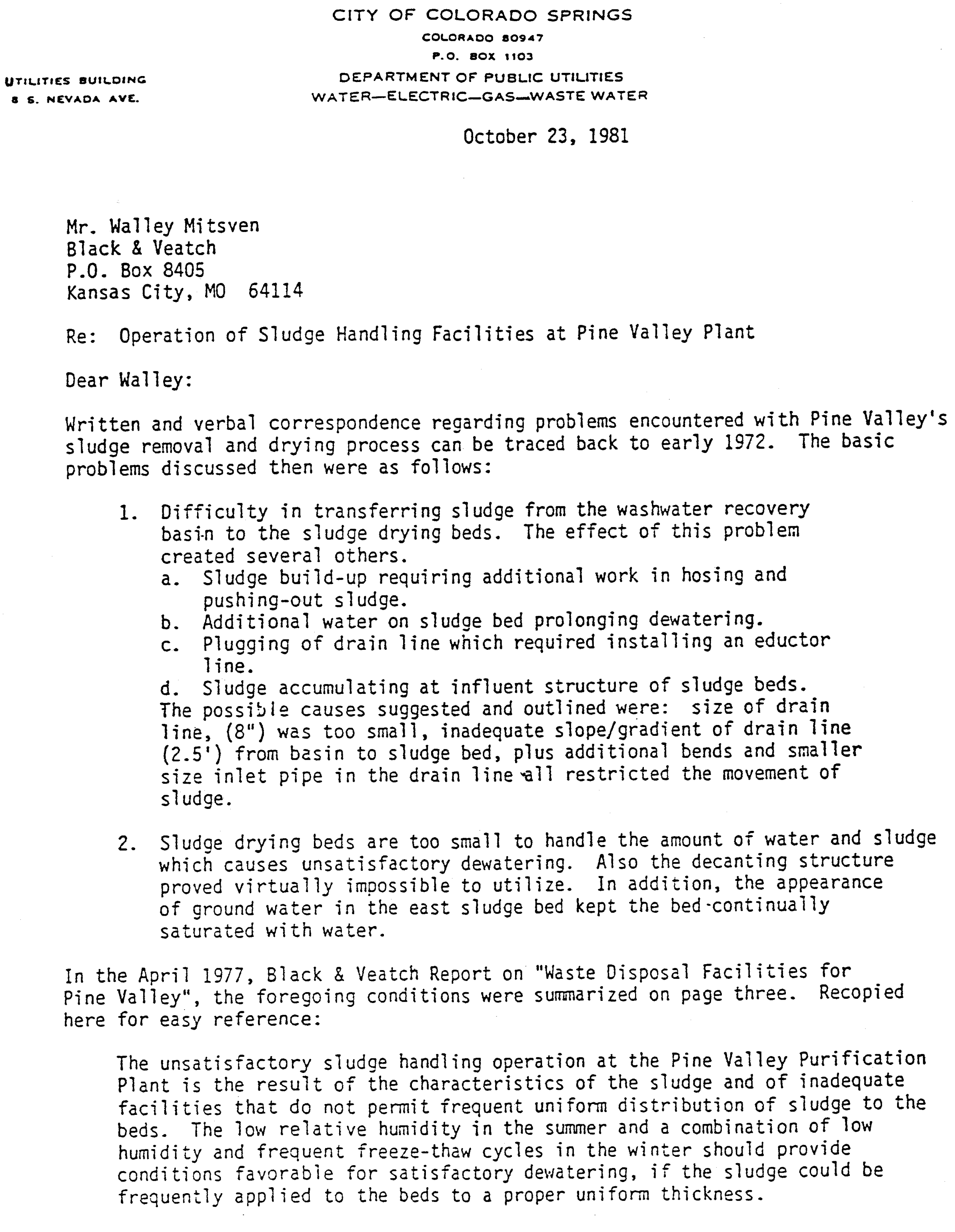
The first cleaning of the sediment drying ponds at Pine Valley occurred in 1971. Data such as yardage, cost, and disposal site for cleaning operations from 1971 through 1991 are presented in Table 12-2. (Note that in 1981, two more sediment drying ponds were added.) Chemical characteristics of the residuals generated from this process are represented in two water quality laboratory reports from 1982 and 1992 (Tables 12-3 and 12-4). The 1982 analysis employed the Aqua Regea total dissolving method, while the 1992 sample was analyzed using the EP toxicity test.

From 1971 to 1976, cleaning and sediment disposal operations were handled by the WTP's construction and equipment divisions. A dragline, trackloader, and loading dump trucks were used to haul the frozen sediment to an area on the U.S. Air Force Academy (USAFA) grounds, 1/4 mile away. In 1977, the sediment drying ponds were full well ahead of freezing and had to be cleaned. The water construction division did not have the equipment necessary to handle the soupy residue. As a result, that was the year the department began contracting sediment disposal operations. The contractor pumped the diluted sediment (2 to 4 percent solids) into a tanker and hauled it to a private landfill, with the exception of the first contractor who hauled it to the area on the USAFA.

Sediment, accumulating at the rate of 7,000 to 8,000 cubic yards per year, eventually became too expensive to contract. The Water Department purchased the needed equipment (sludge pump and tanker) and, in 1983, began hauling sediment (5 to 10 percent solids) itself. The sediment was hauled to Hanna Ranch (city-owned property), for which a certificate of designation was obtained. With the new sediment disposal facilities in operation and receiving most of the sediments, the sediment drying ponds at Pine Valley did not require cleaning in 1992.

12.1.5 Handling and Disposal Costs

Over the years, several thousands of dollars have been spent in removing tons of sediment from Pine Valley. Specifically, the costs associated with cleaning and hauling sediment from Pine Valley have totaled almost $363,000 since 1971 (see Table 12-2). The sludge pump and tank trailer were purchased for $57,000. The 1981 additions to the disposal facilities cost over $475,000, and the 1991 construction of the sediment disposal facilities cost almost 5 million dollars (excluding engineering fees). Additional expenses and time were spent obtaining certificates of designation for the Hanna Ranch and Pine Valley disposal sites. Annual engineering



CITY OF COLORADO SPRINGS
COLORADO 80947
P.O. BOX 1103
DEPARTMENT OF PUBLIC UTILITIES
WATER—ELECTRIC—GAS—WASTE WATER

UTILITIES BUILDING
8 S. NEVADA AVE.

October 23, 1981

Mr. Walley Mitsven
Black & Veatch
P.O. Box 8405
Kansas City, MO 64114

Re: Operation of Sludge Handling Facilities at Pine Valley Plant

Dear Walley:

Written and verbal correspondence regarding problems encountered with Pine Valley's sludge removal and drying process can be traced back to early 1972. The basic problems discussed then were as follows:

1. Difficulty in transferring sludge from the washwater recovery basin to the sludge drying beds. The effect of this problem created several others.
    a. Sludge build-up requiring additional work in hosing and pushing-out sludge.
    b. Additional water on sludge bed prolonging dewatering.
    c. Plugging of drain line which required installing an eductor line.
    d. Sludge accumulating at influent structure of sludge beds.

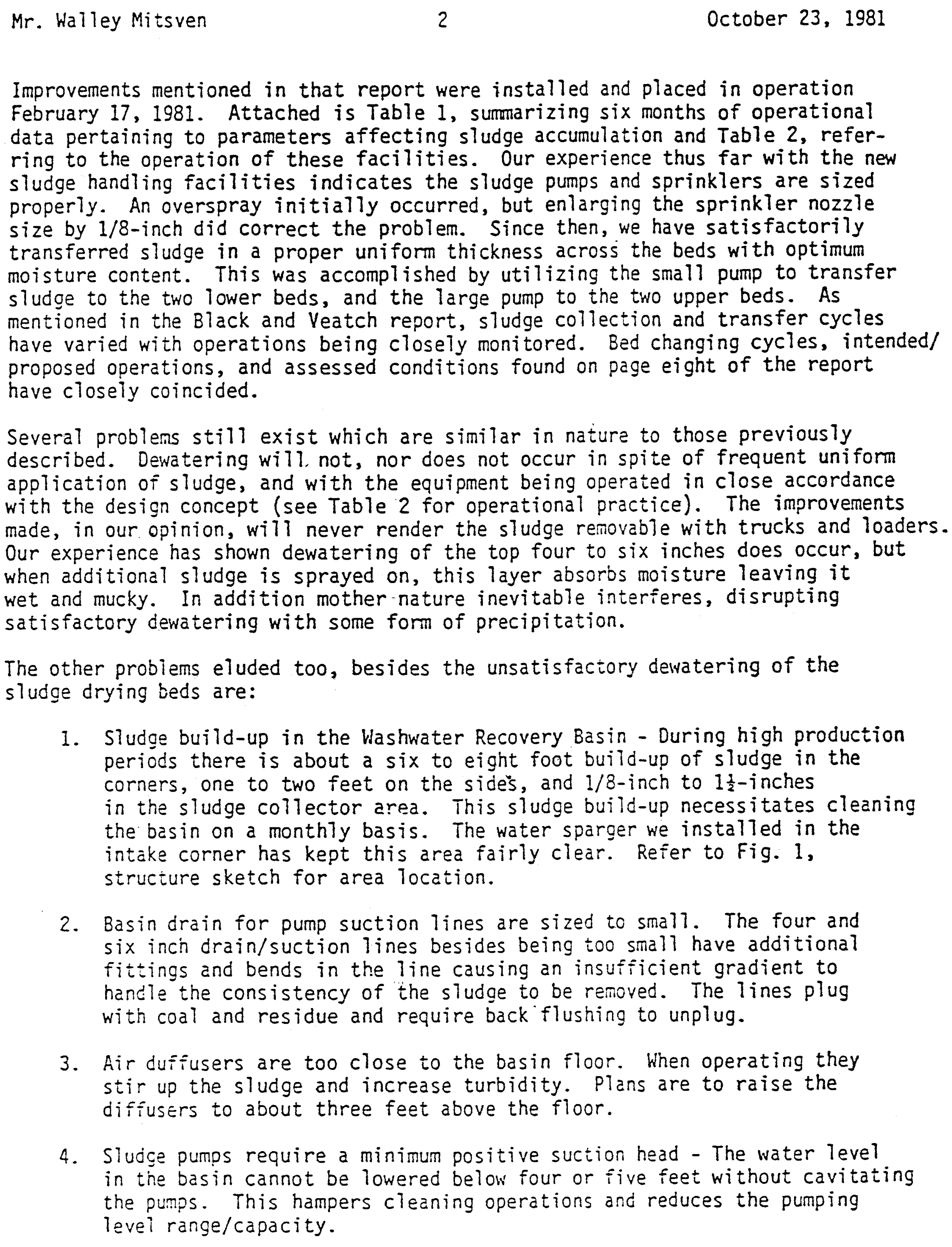
   The possible causes suggested and outlined were: size of drain line, (8") was too small, inadequate slope/gradient of drain line (2.5') from basin to sludge bed, plus additional bends and smaller size inlet pipe in the drain line all restricted the movement of sludge.

2. Sludge drying beds are too small to handle the amount of water and sludge which causes unsatisfactory dewatering. Also the decanting structure proved virtually impossible to utilize. In addition, the appearance of ground water in the east sludge bed kept the bed continually saturated with water.

In the April 1977, Black & Veatch Report on "Waste Disposal Facilities for Pine Valley", the foregoing conditions were summarized on page three. Recopied here for easy reference:

> The unsatisfactory sludge handling operation at the Pine Valley Purification Plant is the result of the characteristics of the sludge and of inadequate facilities that do not permit frequent uniform distribution of sludge to the beds. The low relative humidity in the summer and a combination of low humidity and frequent freeze-thaw cycles in the winter should provide conditions favorable for satisfactory dewatering, if the sludge could be frequently applied to the beds to a proper uniform thickness.

Figure 12-5. Letter regarding new sludge handling facility at Pine Valley WTP, Colorado Springs, CO (Colorado Springs Utilities, 1994).



Mr. Walley Mitsven 2 October 23, 1981

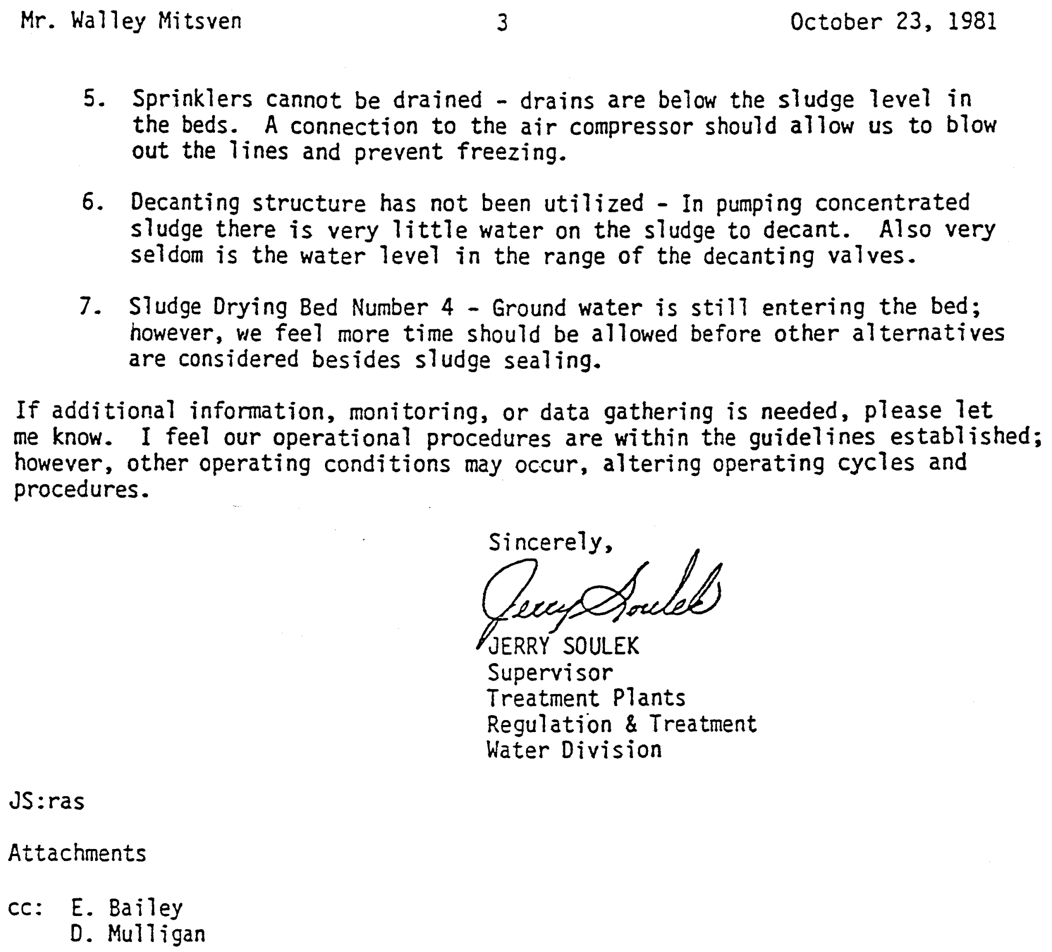
Improvements mentioned in that report were installed and placed in operation February 17, 1981. Attached is Table 1, summarizing six months of operational data pertaining to parameters affecting sludge accumulation and Table 2, referring to the operation of these facilities. Our experience thus far with the new sludge handling facilities indicates the sludge pumps and sprinklers are sized properly. An overspray initially occurred, but enlarging the sprinkler nozzle size by 1/8-inch did correct the problem. Since then, we have satisfactorily transferred sludge in a proper uniform thickness across the beds with optimum moisture content. This was accomplished by utilizing the small pump to transfer sludge to the two lower beds, and the large pump to the two upper beds. As mentioned in the Black and Veatch report, sludge collection and transfer cycles have varied with operations being closely monitored. Bed changing cycles, intended/proposed operations, and assessed conditions found on page eight of the report have closely coincided.

Several problems still exist which are similar in nature to those previously described. Dewatering will not, nor does not occur in spite of frequent uniform application of sludge, and with the equipment being operated in close accordance with the design concept (see Table 2 for operational practice). The improvements made, in our opinion, will never render the sludge removable with trucks and loaders. Our experience has shown dewatering of the top four to six inches does occur, but when additional sludge is sprayed on, this layer absorbs moisture leaving it wet and mucky. In addition mother nature inevitable interferes, disrupting satisfactory dewatering with some form of precipitation.

The other problems eluded too, besides the unsatisfactory dewatering of the sludge drying beds are:

1. Sludge build-up in the Washwater Recovery Basin - During high production periods there is about a six to eight foot build-up of sludge in the corners, one to two feet on the sides, and 1/8-inch to 1½-inches in the sludge collector area. This sludge build-up necessitates cleaning the basin on a monthly basis. The water sparger we installed in the intake corner has kept this area fairly clear. Refer to Fig. 1, structure sketch for area location.
2. Basin drain for pump suction lines are sized to small. The four and six inch drain/suction lines besides being too small have additional fittings and bends in the line causing an insufficient gradient to handle the consistency of the sludge to be removed. The lines plug with coal and residue and require back flushing to unplug.
3. Air duffusers are too close to the basin floor. When operating they stir up the sludge and increase turbidity. Plans are to raise the diffusers to about three feet above the floor.
4. Sludge pumps require a minimum positive suction head - The water level in the basin cannot be lowered below four or five feet without cavitating the pumps. This hampers cleaning operations and reduces the pumping level range/capacity.

Figure 12-5. Letter regarding new sludge handling facility at Pine Valley WTP, Colorado Springs, CO (Colorado Springs Utilities, 1994) (Continued).



5. Sprinklers cannot be drained - drains are below the sludge level in the beds. A connection to the air compressor should allow us to blow out the lines and prevent freezing.

6. Decanting structure has not been utilized - In pumping concentrated sludge there is very little water on the sludge to decant. Also very seldom is the water level in the range of the decanting valves.

7. Sludge Drying Bed Number 4 - Ground water is still entering the bed; however, we feel more time should be allowed before other alternatives are considered besides sludge sealing.

If additional information, monitoring, or data gathering is needed, please let me know. I feel our operational procedures are within the guidelines established; however, other operating conditions may occur, altering operating cycles and procedures.

Sincerely,

Jerry Soulek

JERRY SOULEK
Supervisor
Treatment Plants
Regulation & Treatment
Water Division

JS:ras

Attachments

cc: E. Bailey
D. Mulligan

Figure 12-5. Letter regarding new sludge handling facility at Pine Valley WTP, Colorado Springs, CO (Colorado Springs Utilities, 1994) (Continued).

Table 12-2. Sediment Disposal, Pine Valley WTP, Colorado Springs, CO (Pine Valley WTP, 1994)

Year	Dates	Yardage	Contractor	Cost	Disposal Site
1969		N/A			
1970		N/A			
1971	02/17	2,500–3,000	Water Const.	$3,600.00	USAFA
1972	01/17	2,500–3,000	Water Const.	$3,750.00	USAFA
1973	01/05	2,500–3,000	Water Const.	$3,900.00	USAFA
1974	02/26	2,500–3,000	Water Const.	$4,200.00	USAFA
1975	02/28	2,500–3,000	Water Const.	$4,350.00	USAFA
1976	04/20, 12/07	2,500–3,000	Water Const.	$4,500.00	USAFA
1977	Nov.	2,057	Waste Transport	$4,155.00	USAFA
1978	Aug.	3,491	JD Dye Landfill Costs	$17,105.90 $2,820.80	Landfill
1978	Sept.	3,609	JD Dye Landfill Costs	$13,630.28 $2,915.57	Landfill
1979	Apr.	2,997	JD Dye	$14,863.50	Mesa
1979	Sept.	3,961	Ready Mix	$19,370.17	Private
1980	May	3,900	Ready Mix	$17,511.00	Private
1980	Aug.	4,900	Ready Mix	$19,355.00	Private
1981		N/A	(added two new beds)		
1982	Mar.	7,100	JD Dye	$41,110.15	Mesa
1983	Apr.	7,200	Water Const.	$35,000.00	Hanna
1984	Sept.	7,500	Water Const.	$33,000.00	Hanna
1985	Oct.	7,100	Water Const.	$27,000.00	Hanna
1986	Sept.	7,100	Water Const.	$28,400.00	Hanna
1987	Apr.	7,100	Water Const.	$28,500.00	Hanna
1988	Mar.	2,600	Water Const.	$10,400.00	Hanna
1988	July	1,800	Water Const.	$7,200.00	Hanna
1989		N/A			
1990		N/A			
1991	Jan.	2,400	Water Const.	$10,800.00	Hanna
1991	May	1,200	Water Const.	$5,400.00	Mesa
1992		N/A			

costs for certification renewal since 1991 are unavailable. Additional operation and maintenance charges not accounted for include pest and weed control, landscaping, erosion control measures, and monitoring and collection data requirements.

12.2 Case Study 2: Disposal of Water Treatment Residuals From Mesa Treatment Plant, Colorado Springs, Colorado

The Mesa Water Treatment Plant, located near the Garden of the Gods in the western part of Colorado Springs, was built in 1942 with a design capacity of 4 mgd. It treats waters from the Pikes Peak North and South Slope watersheds. Several expansions and modifications since the plant's opening have increased its design and overload capacities to 42 and 64 mgd, respectively. Figure 12-6 shows the site plan and facilities.

12.2.1 Facility Information

12.2.1.1 Treatment Processes

The existing treatment plant is a conventional facility using chemical addition with rapid mixing, coagulation, flocculation, and sedimentation in three separate treatment trains. Filtration of the combined discharge from the three trains occurs in the filter building using eight dual-media filters (see Figure 12-6). Chemical additions

Table 12-3. Characteristics of Residuals Generated in 1982, Pine Valley WTP, Colorado Springs, CO (Pine Valley WTP, 1994)

Water Utility:	**Colorado Springs Water Dept.**
County:	**El Paso**
Source of Supply:	**Pine Valley Sludge**
Where Collected:	**Sludge Ponds**
Collected By:	**John Uhrich**
Date Sampled:	**9/16/82**
Date Analyzed:	**9/20/82**
Chemist:	**Monte Fryt**
MAIL TO:	
	% Dry Wt = 1.07

		Results[a]	
Parameters	**% Dry Weight**	**mg/L**	**μg/g**
Arsenic	0.0003	0.064	2.78
Barium	0.052	10.4	452.19
Cadmium	0.0007	0.14	6.1
Chromium	0.003	0.5	21.7
Fluoride	0.15	30.5	1,326
Lead	0.007	1.50	65.2
Mercury	0.00009	0.0175	0.76
Nitrate as N		0.09[b]	N/A
Total Hardness as $CaC0_3$		60[b]	N/A
Sulfate		22.3[b]	N/A
pH		6.62[b]	N/A
Selenium	0.00003	0.006	0.26
Silver	0.0000003	0.00054	0.023
Calcium as $CaC0_3$	0.65	130.0	5,652
Magnesium	0.075	15.0	652.2
Iron	2.01	400.0	17,391
Manganese	1.81	360.0	15,652
Zinc	0.66	132.0	5,739
Molybdenum	0.0007	0.158	6.87
PCB	<0.0025	<0.01	<0.5
Aluminum	0.078	15.5	674

[a] Factor converting mg/L to μg/g = 43.48.
[b] As supernatant.

at the Mesa Plant include chlorine, aluminum sulfate, sodium aluminate, coagulant aid (Nalco 8100), filter aid (Nalco 8170), and soda ash. Monthly average dosages for the chemicals used from 1987 through 1992 can be found in Appendix E.

12.2.1.2 Raw Water Supply

Presently, the plant treats a combination of waters from the Pikes Peak watershed, as well as from western slope waters diverted into the North Slope Reservoirs. Water is conveyed to the plant through three pipelines—two gravity flow pipelines from the Manitou Hydro Afterbay, and one pumped pipeline from the Fountain Creek intake pumping station. Water quality data for the plant influent raw water from 1987 through 1992 are shown in Appendix E. Results of influent laboratory analyses for April and August of 1992 are listed in Tables 12-5 and 12-6. Data from specific pipeline supplies are outlined in Table 12-7.

12.2.1.3 Plant Facility and Flow

Raw water enters the plant through three pipelines (see Figure 12-6). Modulating valves control the flow of water to one of three parallel treatment trains designated as numbers 24, 30A, and 30B. Water enters one of the rapid mix chambers, then travels through the flocculation and sedimentation basins before moving on to the filters. Treated water leaving the plant is stored in an adjacent 20-MG covered reservoir. Filter backwash water and other wastewater from the basins are discharged into an 18-MG earthen collection basin and are then recycled to the plant influent for retreatment. The volumes of water recycled from 1987 through 1992 are shown in Table 12-8.

12.2.2 Residuals Management

The management of water treatment plant residuals commenced with the Federal Water Pollution Control Act/Colorado Water Quality Control Act promulgated in 1973. This act prohibited the discharge of pollutants in excess of prescribed limits into streams. Direct discharges of residuals from the Mesa Plant did not meet the established standards; therefore, the practice was discontinued. Residuals management and disposal was then a reality and had to be dealt with.

Over the years, several contingency plans were studied to determine the most effective and efficient method of residuals management. This meant considering ways to minimize operating expenses, reduce handling costs, and maximize water reuse. Sediment lagoons (sediment drying beds) were determined to be the most effective way to handle residuals. Until this option could be implemented, certificates of designation were obtained or amended, and alternate disposal sites were sought. Other proposals that were occasionally considered included various types of mechanical dewatering equipment, chemical conditioning of the sediment, and discharging wastes to a sanitary sewer. None of these proposals ever materialized.

12.2.3 Residuals Handling Facilities and Operations

12.2.3.1 Facilities

Current residuals disposal facilities at Mesa Plant consist of a large collection and holding basin (18 MG-washwater recovery basin), a sediment drying pond (4 MG),

Table 12-4. Characteristics of Residuals Generated in 1992, Pine Valley WTP, Colorado Springs, CO (Pine Valley WTP, 1994)

CITY OF COLORADO SPRINGS WATER QUALITY LABORATORY
SAMPLE REPORT

Sample Site: **PVSD Pine Valley Sediment (PVSD)**
Sample Number: **92120331**
Comments:

Date: **3/18/93** **09:56 AM**
Water Type: **M**
Sample Date: **12/30/92**

Analysis Description/Code		**Result**	**Result Code**	**Minimum Reported Level**	**Test Units**	**Method Code**
pH	PH	5.8			S.U.	E
Percent solid	PS	14.8			mg/L	GRAV
Magnesium (dissolved)	MGD	0.16		0.10	mg/L	FAASA
Silver (dissolved)	AGD	0	ND	0.5	µg/L	GFAAS
Aluminum (dissolved)	ALD	3,200		400	µg/L	FAASN
Arsenic (dissolved)	ASD	0	ND	4	µg/L	GFAAS
Barium (dissolved)	BAD	150		100	µg/L	FAASN
Cadmium (dissolved)	CDD	2.2		0.10	µg/L	GFAAS
Chromium (dissolved)	CRD	0	ND	50	µg/L	FAASN
Copper (dissolved)	CUD	19		12	µg/L	FAASA
Mercury (dissolved)	HGD	0	ND	0.2	µg/L	CVAAS
Lead (dissolved)	PBD	2		1.0	µg/L	GFAAS
Selenium (dissolved)	SED	0	ND	1.0	µg/L	VGAAS

Key
ND = Not detected.

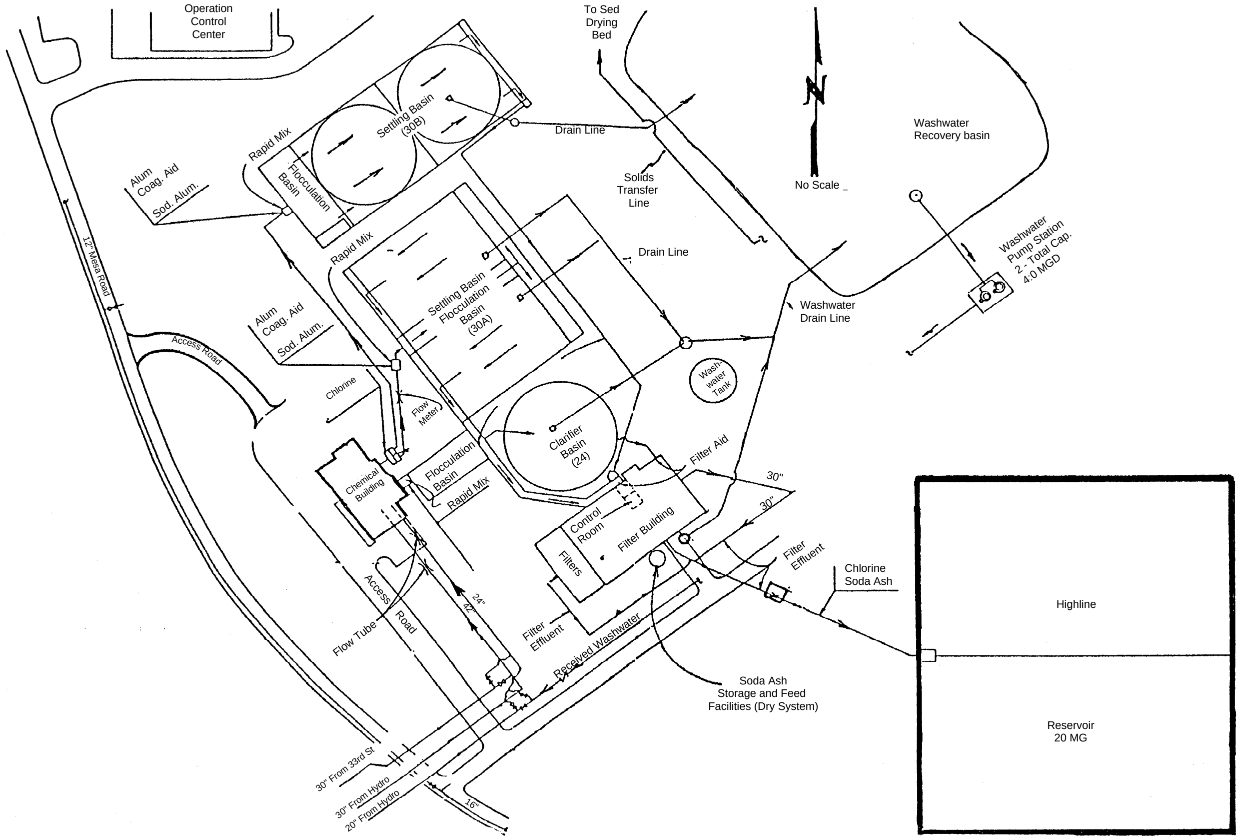


Figure 12-6. Mesa WTP, Colorado Springs, CO (Colorado Springs Utilities, 1994).

Table 12-5. Laboratory Samples: April 1992, Fountain Creek, Colorado Springs, CO (Mesa WTP, 1994)

CITY OF COLORADO SPRINGS WATER QUALITY LABORATORY
SAMPLE REPORT

Sample Site:	**FCR Fountain Creek at 33rd Street (FCR)**	**Date:**	**31-Mar-1993 03:59 PM**
Sample Number:	**92040281**	**Water Type:**	**R**
Comments:		**Sample Date:**	**28-APR-92**

Analysis Description/Code		Result	Result Code	Minimum Reported Level	Test Units	Method Code
pH	PH	8.07			S.U.	E
Conductivity	CON	245			µmhos	E
Turbidity	TUR	23			NTU	N
HCO_3 alkalinity	ALK	81			mg/L	T
CO_3 alkalinity	CO3	0			mg/L	T
Hardness as $CaCO_3$	HAR	94			mg/L	T
Fluoride	F	2.56		0.10	mg/L	E
Total dissolved solids	TDS	161			mg/L	GRAV
Ammonia (total)	NH3	0.06		0.05	mg/L	E
Chloride	CL	13		0.4	mg/L	IC
Nitrate	NO3	0.66		0.01	mg/L	IC
Sulfate	SO4	13.2		1.0	mg/L	IC
Barium (dissolved)	BAD	67		5	µg/L	GFAAS
Cadmium (dissolved)	CDD	0.26		0.10	µg/L	GFAAS
Copper (total)	CU	1		1	µg/L	GFAAS
Iron (total)	FE	980		40	µg/L	FAASA
Iron (dissolved)	FED	120		40	µg/L	FAASA
Manganese (total)	MN	120		20	µg/L	FAASA
Manganese (dissolved)	MND	30		20	µg/L	FAASA
Selenium (dissolved)	SED	0	ND	1.0	µg/L	VGAAS
Chlorine residual	CL2	0			mg/L	DPD
Temperature centigrade	TEC	8			C	THER
Threshold odor number	TON	3				NO
Total coliform	TC	24,000			cfu/100mL	MTF
Fecal coliform	FC	5,000			cfu/100mL	MTF
Fecal *Streptococus*	FS	70			cfu/100mL	MF

Key
ND = Not detected.

and an onsite designated landfill area. Equipment used to transfer sediment from the holding basin to the landfill area includes a Mudcat/dredge, trackloader, dump trucks, and bulldozer.

12.2.3.2 Operations

Spent filter backwash water and water latent with sediment from the flocculation and sedimentation basins are discharged into the recovery basin. Decant water is periodically pumped back to the head of the plant and retreated. The sediment that is retained in the wastewater recovery basin accumulates at the rate of approximately 7,000 to 8,000 cubic yards per year and remains there until removed. Mesa has sufficient capacity to store approximately 40,000 to 50,000 cubic yards of sediment in its recovery basin before storage becomes a problem. This storage capacity is equivalent to roughly 4 to 5 years of residuals production. Analyses of the chemical characteristics of the residuals generated in 1978 and 1992 are presented in Tables 12-9A, 12-9B, and 12-10, respectively. Note that there are two different

Table 12-6. Laboratory Samples: August 1992, Mesa WTP, Colorado Springs, CO (Mesa WTP, 1994)

CITY OF COLORADO SPRINGS WATER QUALITY LABORATORY
SAMPLE REPORT

Sample Site:	**MPI Mesa Plant Influent (MPI)**	**Date:**	**31-Mar-1993 04:01 PM**
Sample Number:	**92080222**	**Water Type:**	**R**
Comments:	**recov. pumps on; plant TUR=3.0**	**Sample Date:**	**25-AUG-92**

Analysis Description/Code		Result	Result Code	Minimum Reported Level	Test Units	Method Code
pH	PH	7.45			S.U.	E
Conductivity	CON	50			µmhos	E
Color	COL	25				COMP
Turbidity	TUR	3.8			NTU	N
HCO_3 alkalinity	ALK	16			mg/L	T
CO_3 alkalinity	CO3	0			mg/L	T
Hardness as $CaCO_3$	HAR	24			mg/L	T
Fluoride	F	2.28		0.10	mg/L	E
Chloride	CL	0.4		0.4	mg/L	IC
Nitrate	NO3	0.13		0.01	mg/L	IC
Sulfate	SO4	3.3		1.0	mg/L	IC
Sodium (dissolved)	NAD	2.1		0.5	mg/L	IC
Potassium (dissolved)	KD	1.3		0.5	mg/L	IC
Magnesium (dissolved)	MGD	0.7		0.5	mg/L	IC
Calcium (dissolved)	CAD	5.8		3	mg/L	IC
Silver (dissolved)	AGD	0	ND	0.5	µg/L	GFAAS
Arsenic (dissolved)	ASD	0	ND	4	µg/L	GFAAS
Chromium (dissolved)	CRD	0	ND	0.4	µg/L	GFAAS
Iron (total)	FE	240		40	µg/L	FAASA
Iron (dissolved)	FED	120		40	µg/L	FAASA
Manganese (total)	MN	20		20	µg/L	FAASA
Manganese (dissolved)	MND	0	ND	20	µg/L	FAASA
Chlorine residual	CL2	0			mg/L	DPD
Temperature centigrade	TEC	12			C	THER
Threshold odor number	TON	1				NO
Total coliform	TC	170			Cfu/100mL	MF
Fecal coliform	FC	4			Cfu/100mL	MF
Fecal *Streptococus*	FS	20			Cfu/100mL	MF

Key
ND = Not detected.

test methods employed in these analyses—the Aqua Regea total dissolving method used in 1978, and the EP toxicity test used in 1992.

12.2.4 Residuals Disposal

Since the first cleaning of the recovery basin in 1974, the basin has been cleaned every 5 years. Information pertaining to these cleaning operations can be found in Table 12-11. In late 1973, the plant was taken out of service, recycling most of the water from the recovery basin before shutting down. Over the winter, the recovery basin was allowed to dry and freeze; in early spring, a track loader, dragline, and dump trucks removed and transferred the sediment to a city-owned landfill area.

In 1979, a different method of sediment disposal was employed. Higher levels of water demand prevented the

Table 12-7. Raw Water Source for Mesa WTP, Colorado Springs, CO (Mesa WTP, 1994)

	Fountain Creek Sample		Manitou Afterbay Sample	
	Mean	Range	Mean	Range
pH	7.58	8.15–7.19	7.56	8.05–7.17
Alkalinity (mg/L)	27.30	76.0–11.0	24.5	62.0–12.0
Hardness (mg/L)	29.8	86.0–13.0	26.4	64.0–14.0
Temperature (°F)	47.2	62.7–35.1	47.0	62.7–35.6
Turbidity (NTU)	4.70	133.6–0.6	3.70	36.0–0.6

plant and recovery basin from being taken out of service. A floating dredge (Mudcat) was purchased to dredge and pump the sediment to a small berm area isolated from one end of the recovery basin. From here, a contractor pumped the diluted sediment, which was mostly water, into a tanker and hauled it to a private landfill.

Cleaning and sediment disposal operations in 1984 and 1989 were handled in the same manner—the Mudcat was used to dredge and pump sediment from the recovery basin into a large sediment drying pond built in 1984. A floating outlet discharge line was used to decant clear water from the drying pond back into the recovery basin. Cleaning operations were discontinued when the recovery basin was considered cleaned or when the drying pond was full of sediment. The Mudcat was removed from the recovery basin, and the sediment drying pond was allowed to freeze, thaw, and dry over the winter and into the next year.

In the spring of 1985, the underlying sediment was still too soupy to load and haul. In autumn of 1985 and 1990, after 1 year of the freeze-thaw drying cycle, the sediment had dewatered to a level conducive to hauling (estimated between 10 to 15 percent solids). The depth of sediment had been reduced from roughly 8 feet to 3 feet. A track loader, belly dump, and bulldozer were used to transfer and spread the sediment onto the adjacent landfill in a layer approximately 1 foot thick.

When the top layer of sediment at the landfill dried, it would turn into a fine granular powder, which created quite a dust problem when the wind blew. A dust ban polymer was applied to minimize the dust problem during the dewatering process. In addition, newly placed sediment at the landfill was seeded with native grasses to control erosion and dust pollution. Ground-water monitoring wells are not required at Mesa; however, they are in place at Hanna Ranch where sediment from Mesa has been taken. Results of a leachate sample collected in April 1990 are shown in Figure 12-7.

12.3 Case Study 3: Land Application of Water Treatment Plant Residuals at Cobb County-Marietta Water Authority, Marietta, Georgia

The Cobb County-Marietta Water Authority (CCMWA) operates two WTPs in Cobb County, Georgia, which is in the metropolitan Atlanta area. The James E. Quarles Plant is permitted to treat 58 mgd of raw water from the Chattahoochee River. The Chattahoochee River is the primary source of drinking water for metropolitan Atlanta. The Hugh A. Wyckoff Plant is permitted to treat 72 mgd of raw water from Lake Allatoona, a U.S. Army Corps of Engineers impoundment on the Etowah River in northwest Georgia.

Both plants use the same method of residuals handling. Sludge is removed daily from the settling basins and sent to gravity sludge thickeners. Backwash water from the filters is sent to a backwash recycle tank where the sludge settles out and the clarified backwash water is returned to the head of the plant. The settled sludge is scraped to one end of the recycle tank and pumped to the gravity sludge thickeners. The gravity thickened sludge is then pumped to a conditioning tank where lime slurry is mixed with the sludge on a 10 to 15 percent dry weight basis. The conditioned sludge is then dewatered in a pressure filter to obtain a sludge cake of approximately 35 percent total solids. The cake is then transported to a sludge storage yard where it is stockpiled for several months. Two to three times each year, the sludge is loaded into trucks and hauled to agricultural land in or near Cobb County. The dewatered sludge is applied to the fields with a manure spreader. The

Table 12-8. Water Treated and Used, Mesa WTP, Colorado Springs, CO, in Million Gallons (Mesa WTP, 1994)

	Water Treated	Washwater	Percent Washwater	Maintenance Water	Percent W/W & M/W	Recycled Water
1987	6,089.827	149.977	2.46	2.729	2.51	319.408
1988	6,695.425	196.539	2.94	6.830	3.04	431.176
1989	6,554.006	269.520	4.11	8.359	4.24	388.040
1990	6,662.514	215.598	3.24	11.069	3.40	344.745
1991	6,929.688	201.402	2.91	19.709	3.19	389.142
1992	6,530.272	132.187	2.02	7.587	2.14	366.373
Averages	6,576.955	194.202		9.381		373.147

Table 12-9A. Chemical Characteristics of Residuals (Sludge) From Mesa WTP, Colorado Springs, CO, 1978 (Mesa WTP, 1994)

Water Analysis[a]

Location: Mesa Plant Sludge (6.571% dry wt) **Date: 20 March, 1978**

Analysis (Dry Wt)	Limits	Results	Analysis (Dry Wt)	Limits	Results
Total coliform (per 100 mL)	4	—	Iron (Fe) 1.30	0.30	1,600
Carbon dioxide (CO_2)	NL	—	Manganese (Mn) 0.22	0.05	270.0
Residual Cl_2	NLT 0.1	—	Nickel (Ni) 0.001	NL	1.60
Temperature (°F)	78°	—	Copper (Cu) 0.012	1.0	14.6
Color intensity	15	—	Lead (Pb) 0.000002	0.05	0.003
Taste and odor (Threshold Odor Units)	3	—	Zinc (Zn) 0.014	5	17.0
Turbidity (NTU)	1.0	—	Calcium (Ca) 0.23	NL	280.0
Specific conductance (Micromhos –25°C)	NL	—	Magnesium (Mg) 0.12	125	145
Total dissolved solids	NL	—	Cadmium (Cd) 0.0002	0.01	0.20
Hardness as $CaCO_3$	NL	—	Barium (Ba) 0.003	1.0	3.50
Noncarbonate hardness as $CaCO_3$	NL	—	Sodium (Na) 1.82	NL	2,250
Bicarbonate (HCO_3)	NL	—	Potassium (K) 1.30	NL	1,600
Carbonate (CO_3)	NL	—	pH	9.5 5.6	—
Chloride (Cl)	250	—	Acidity	NL	—
Silica (SiO_2) 0.085	NL	104.7	Mercury (Hg) 0.002 (μg/100 mL)	0.2	3.0
Sulfate (SO_4) 0.28	250	350	Molybdenum (Mo)		—
Nitrate (NO_3)	45	—	Arsenic (As) 0.0001	0.05	0.14
Total phosphate (PO_4) 0.17	NL	205.4	Cyanide (CN) 0.0001	0.2	0.14
Fluoride (F) 0.22	62° 2.0	272	Fecal coliform (per 100 ml)	4	—
Aluminum (Al) 3.08	NL	3,800	Fecal *Streptococcus* (per 100 mL)		—
Total chromium (Cr) 0.006	0.05	7.8	Plate count (org/mL)[b]		Backwash H_2O 700 Sludge pond 39
Cobalt (Co) 0.0004		0.50			

[a] All chemical analyses in milligrams per liter except where otherwise indicated.
[b] Sample collected 3/22/78.

application rate depends on the lime requirements of the soil and the lime content of the sludge.

As of 1995, there are no state or local regulations that deal with the land application of WTP residuals. The Georgia Environmental Protection Division has classified the residuals as “recovered materials” under the Rules for Solid Waste Management, Chapter 391.3 to 4.04(7). Under this classification the residuals do not require a solids waste handling permit.

12.3.1 Facility Information

The James E. Quarles Plant is presently permitted for a total flow of 58 mgd. It consists of an older plant, Plant No. 1, that is permitted to handle 36 mgd, and a newer plant, Plant No. 2, that is permitted to handle 22 mgd. Figure 12-8 is a schematic outline of the plant.

The raw water for both plants is drawn from the Chattahoochee River and pumped 8,500 feet to a small raw water reservoir with a capacity of 20 MG. From the reservoir, the water flows through separate lines to Plants No. 1 and 2.

Plant No. 1 adds alum, chlorine, and chlorine dioxide at the rapid mix facility. The water then flows through eight hydraulic flocculators with a detention time of 17 minutes at the rated flow of 36 mgd. There are eight rectangular sedimentation basins with a theoretical detention

Table 12-9B. Chemical Characteristics of Residuals (Supernatant) From Mesa WTP, Colorado Springs, CO, 1978 (Mesa WTP, 1994)

Water Analysis[a]

Location: Mesa Plant Sludge (Supernatant) (93.429%) **Date: 20 March, 1978**

Analysis (Dry Wt)	Limits	Results	Analysis (Dry Wt)	Limits	Results
Total coliform (per 100 mL)	4	Backwash H_2O Sludge pond 20,000	Iron (Fe)	0.30	1.5
Carbon dioxide (CO_2)	NL	2.7	Manganese (Mn)	0.05	20.0
Residual Cl_2	NLT 0.1	0	Nickel (Ni)	NL	0.014
Temperature (°F)	78°	—	Copper (Cu)	1.0	0.009
Color intensity	15	—	Lead (Pb)	0.05	0.001
Taste and odor (Threshold Odor Units)	3	—	Zinc (Zn)	5	0.025
Turbidity (NTU)	1.0	—	Calcium (Ca)	NL	41.6
Specific conductance (Micromhos –25°C)	NL	260	Magnesium (Mg)	125	3.5
Total dissolved solids	NL	174	Cadmium (Cd)	0.01	0.001
Hardness as $CaCO_3$	NL	109	Barium (Ba)	1.0	0.002
Noncarbonate hardness as $CaCO_3$	NL	40	Sodium (Na)	NL	11.2
Bicarbonate (HCO_3)	NL	69	Potassium (K)	NL	4.0
Carbonate (CO_3)	NL	0	pH	9.5 5.6	7.70
Chloride (Cl)	250	8.5	Acidity	NL	3.5
Silica (SiO_2) 0.085	NL	1.55	Mercury (Hg) (µg/100 mL)	0.2	0.062
Sulfate (SO_4) 0.28	250	30	Molybdenum (Mo)		—
Nitrate (NO_3)	45	0.25	Arsenic (As)	0.05	0.004
Total phosphate (PO_4)	NL	0.55	Cyanide (CN)	0.2	0
Fluoride (F)	62° 2.0	2.40	Fecal coliform (per 100 mL)	4	—
Aluminum (Al)	NL	0.17	Fecal *Streptococcus* (per 100 mL)		—
Total chromium (Cr)	0.05	0.01	Plate count (org/mL)[b]		Backwash H_2O 700 Sludge pond 39
Cobalt (Co)		0.0025			

[a] All chemical analyses in milligrams per liter except where otherwise indicated.
[b] Sample collected 3/22/78

time of 160 minutes. The sludge is removed with retrofitted siphon-type sludge collectors located in the upper one-third of each basin. Once or twice each year the basins are drained to remove any buildup of sludge that occurs.

The water is then filtered through eight double-bay gravity dual media filters. The present permitted filter rate is 4.5 gallons per minute per square foot (gpm/ft^2). The filter backwash system consists of surface sweeps and underdrain backwash.

The filtered water is then treated with lime, fluoride, and additional chlorine before storage in the plant clearwells. The finished water is then pumped to customers.

Plant No. 2, which was built in 1980, has the same process train as Plant No. 1. Alum, chlorine, and chlorine dioxide are added at the rapid mix facility. The water then flows through four hydraulic flocculators with a detention time of 19 minutes at the rated flow of 22 mgd. There are four rectangular sedimentation basins with a theoretical detention time of 155

Table 12-10. Chemical Characteristics of Residuals Generated in 1992, Mesa WTP, Colorado Springs, CO (Mesa WTP, 1994)

CITY OF COLORADO SPRINGS WATER QUALITY LABORATORY
SAMPLE REPORT

Sample Site:	**MPSD Mesa Plant Sediment (MPSD)**	**Date:**	**03/18/93 09:56 AM**
Sample Number:	**92120330**	**Water Type:**	**M**
Comments:		**Sample Date:**	**12/30/92**

Analysis Description/Code		Result	Result Code	Minimum Reported Level	Test Units	Method Code
pH	PH	7.03			SU	E
Percent solid	PS	39.8			mg/L	GRAV
Magnesium (dissolved)	MGD	3.6		0.10	mg/L	FAASA
Silver (dissolved)	AGD	0	ND	0.5	µg/L	GFAAS
Aluminum (dissolved)	ALD	7,300		400	µg/L	FAASN
Arsenic (dissolved)	ASD	0	ND	4	µg/L	GFAAS
Barium (dissolved)	BAD	370		100	µg/L	FAASN
Cadmium (dissolved)	CDD	0.25		0.10	µg/L	GFAAS
Chromium (dissolved)	CRD	0	ND	50	µg/L	FAASN
Copper (dissolved)	CUD	0	ND	12	µg/L	FAASA
Mercury (dissolved)	HGD	0	ND	0.2	µg/L	CVAAS
Lead (dissolved)	PBD	0	ND	1.0	µg/L	GFAAS
Selenium (dissolved)	SED	0	ND	1.0	µg/L	VGAAS

Key
ND = Not detected.

Table 12-11. Sediment Disposal, Mesa WTP, Colorado Springs, CO (Mesa WTP, 1994)

Year	Date	Cubic Yards	Contractor	Cost	Disposal Site	Method
1974	Mar.	40,000	Water construction	$17,000	City landfill	Loader and dump trucks
1979	May	50,000	Dye construction	$150,000	Private landfill	Dredge and tanker
1984	Oct.	25,000	Water construction Water treatment	$40,000	On site	Dredge loader, dump trucks, bulldozer
1989	Oct.	25,000	Water construction Water treatment	$45,000	On site	Dredge loader, dump trucks, bulldozer
Totals		140,000		$252,000		

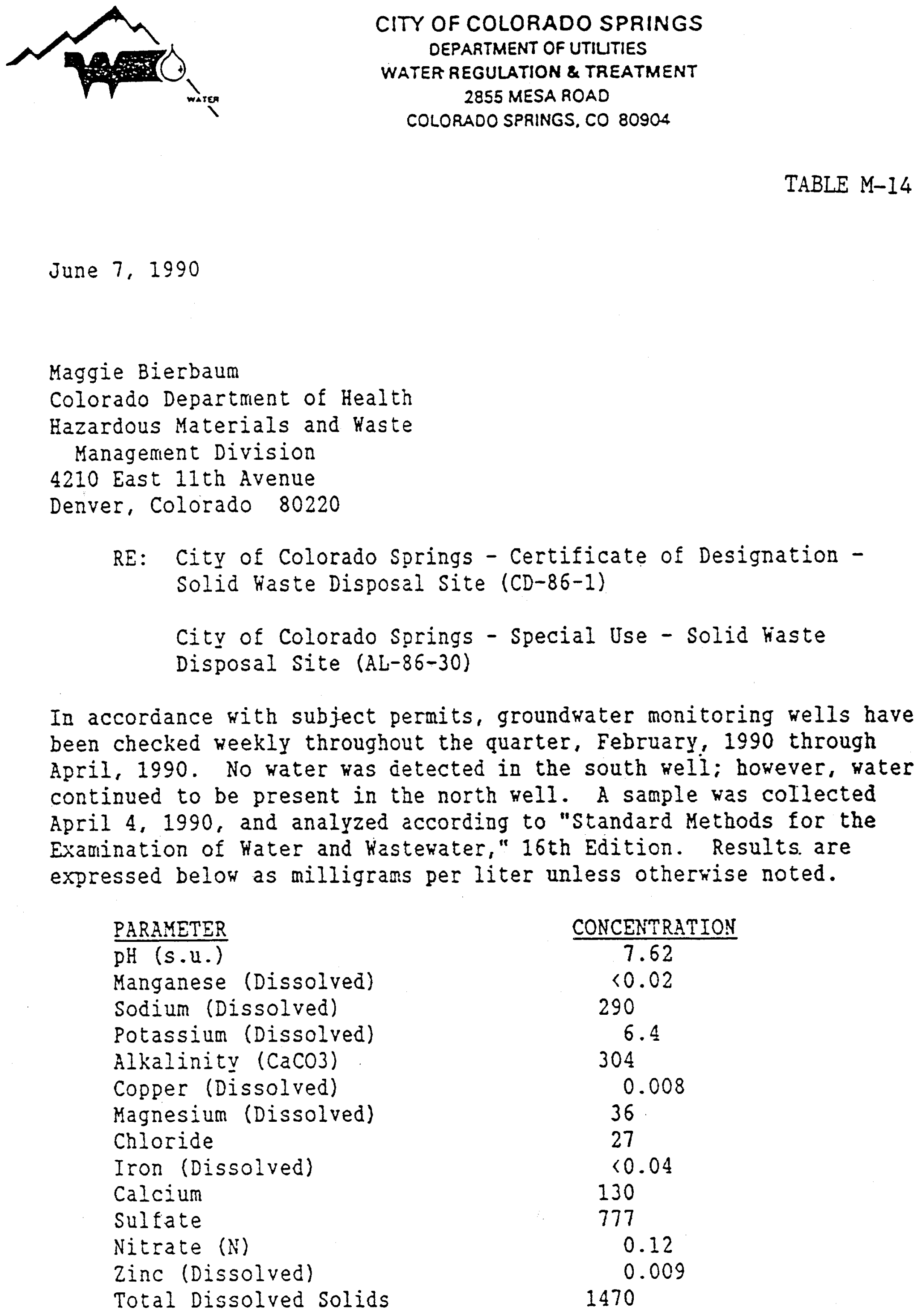
minutes. The sludge is removed with siphon-type sludge collectors that traverse the entire length of each basin. The sedimentation basins are drained once or twice each year to remove any sludge buildup.

The water is then filtered through four double-bay gravity dual media filters. The present permitted filter rate is 5.5 gpm/ft^2. The filter backwash system consists of surface sweeps and underdrain backwash. One filter is fitted with an air backwash system.

The filtered water is then treated with lime, fluoride, and additional chlorine before storage in the plant clearwells. The finished water is then pumped to customers.

The Hugh A. Wyckoff Plant is presently permitted for a total flow of 72 mgd. The raw water for this plant is drawn from Lake Allatoona and pumped 4.5 miles to the plant. Figure 12-9 is a schematic outline of the plant.

Alum, chlorine, and chlorine dioxide are added before an in-line mechanical mixer. The water then goes through six flocculator basins. Four basins have mechanical mixers with a detention time of 23 minutes. Two basins have hydraulic mixing with a detention time of 32 minutes. Six rectangular sedimentation basins have a detention time of 136 minutes. The sludge is removed from four of the basins with circular mechanical sludge collectors, located in the first two-thirds of each basin. The two newer basins are equipped with floating siphon-type sludge collectors for the entire length of the basins. The sedimentation basins are drained each year to remove sludge buildup.



CITY OF COLORADO SPRINGS
DEPARTMENT OF UTILITIES
WATER REGULATION & TREATMENT
2855 MESA ROAD
COLORADO SPRINGS, CO 80904

TABLE M-14

June 7, 1990

Maggie Bierbaum
Colorado Department of Health
Hazardous Materials and Waste
Management Division
4210 East 11th Avenue
Denver, Colorado 80220

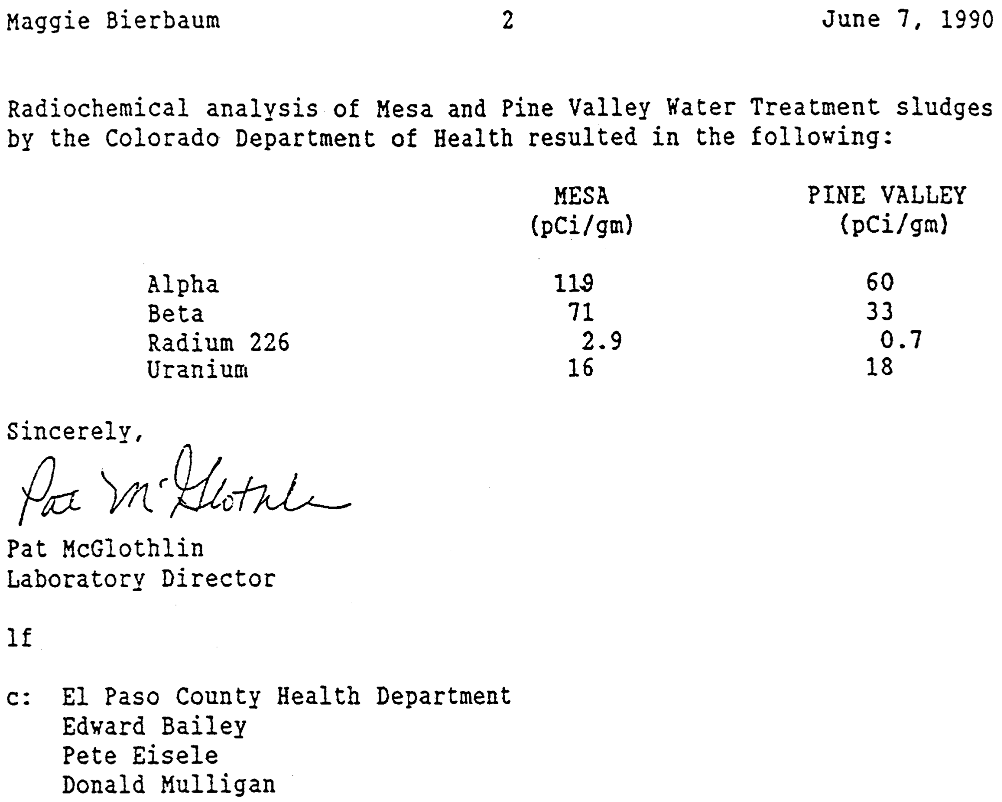
RE: City of Colorado Springs - Certificate of Designation - Solid Waste Disposal Site (CD-86-1)

City of Colorado Springs - Special Use - Solid Waste Disposal Site (AL-86-30)

In accordance with subject permits, groundwater monitoring wells have been checked weekly throughout the quarter, February, 1990 through April, 1990. No water was detected in the south well; however, water continued to be present in the north well. A sample was collected April 4, 1990, and analyzed according to "Standard Methods for the Examination of Water and Wastewater," 16th Edition. Results are expressed below as milligrams per liter unless otherwise noted.

PARAMETER	CONCENTRATION
pH (s.u.)	7.62
Manganese (Dissolved)	<0.02
Sodium (Dissolved)	290
Potassium (Dissolved)	6.4
Alkalinity (CaCO3)	304
Copper (Dissolved)	0.008
Magnesium (Dissolved)	36
Chloride	27
Iron (Dissolved)	<0.04
Calcium	130
Sulfate	777
Nitrate (N)	0.12
Zinc (Dissolved)	0.009
Total Dissolved Solids	1470

Figure 12-7. Letter from Colorado Springs Department of Utilities, CO (Colorado Springs Utilities, 1994).



Radiochemical analysis of Mesa and Pine Valley Water Treatment sludges by the Colorado Department of Health resulted in the following:

	MESA (pCi/gm)	PINE VALLEY (pCi/gm)
Alpha	119	60
Beta	71	33
Radium 226	2.9	0.7
Uranium	16	18

Sincerely,

Pat McGlothlin

Pat McGlothlin
Laboratory Director

lf

c: El Paso County Health Department
Edward Bailey
Pete Eisele
Donald Mulligan

Figure 12-7. Letter from Colorado Springs Department of Utilities, CO (Colorado Springs Utilities, 1994) (Continued).

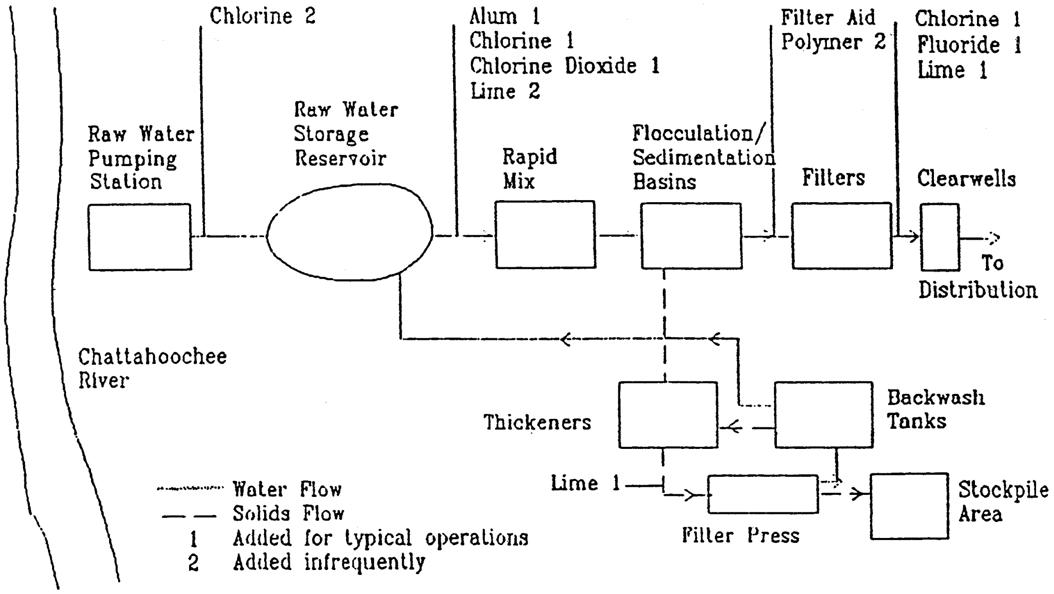


Figure 12-8. Schematic process diagram of Quarles WTP, Marietta, GA (Cobb County, 1994).

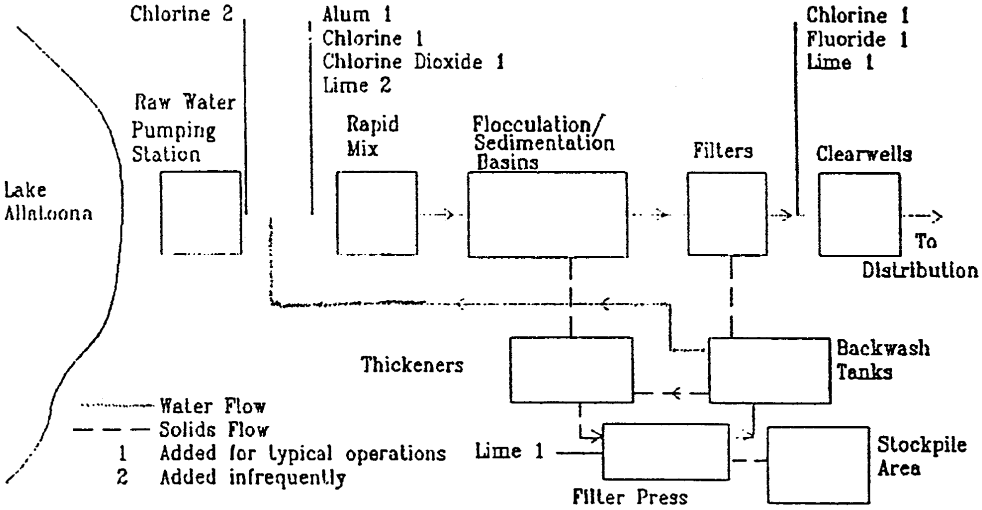


Figure 12-9. Schematic process diagram of Wyckoff WTP, Marietta, GA (Cobb County, 1994).

The water is then filtered through eight double-bay gravity dual media filters. The present permitted filter rate is 6.0 gpm/ft^2. The filters can be operated in the constant rate or declining rate modes. The filter backwash system consists of surface sweeps and underdrain backwash.

The filtered water is then treated with lime, fluoride, and additional chlorine before storage in the plant clearwells. The finished water is then pumped to customers.

12.3.2 Residuals Handling Facilities

The physical and chemical characteristics of the residuals generated from the two plants differ slightly because of the different raw water sources and the amount of lime used by the plants in the dewatering process. The residuals handling facilities for both treatment plants are the same.

The residuals generated by the two plants each have a solids content of approximately 35 percent following pressure filtration. The lime content on a dry weight basis is approximately 10 to 15 percent. The calcium carbonate equivalency (CCE) for the material ranges from 20 to 40 percent. The material is highly plastic until it is dried further. It has the appearance of moist dirt and has a slight but noticeable odor.

Extensive testing of the sludge has been conducted. The Toxicity Characteristic Leaching Procedure (TCLP) has been used to analyze the residuals from both plants on several occasions. TCLP analyses for metals,

Table 12-12. TCLP Data, Cobb County-Marietta Water Authority, Marietta, GA (Cobb County, 1994)

	Regulatory Level	Laboratory Data	
Compound	**EPA (mg/L)**	**Quarles (mg/L)**	**Wyckoff (mg/L)**
Volatile Organics			
Chloroform	6.0	0.013	<0.005
Carbon tetrachloride	0.5	<0.005	<0.005
Tetrachloroethene	0.7	<0.005	<0.005
Clorobenzene	100.0	<0.005	<0.005
1,2-Dichloroethane	0.5	<0.005	<0.005
Benzene	0.5	<0.005	<0.005
Vinyl chloride	0.2	<0.005	<0.005
1,1-Dichloroethene	0.7	<0.005	<0.005
Trichloroethene	0.5	0.016	<0.005
1,4-Dichlorobenzene	7.5	<0.005	<0.005
2-Butanone (MEK)	200.0	<0.10	<0.10
Metals			
Arsenic	5.0	<0.005	<0.005
Barium	100.0	1.03	1.22
Cadmium	1.0	<0.005	<0.005
Chromium	5.0	0.12	<0.05
Lead	5.0	<0.05	<0.05
Mercury	0.2	<0.0002	<0.0002
Selenium	1.0	<0.005	<0.005
Silver	5.0	<0.01	<0.01
Pesticides and Herbicides			
Chlordane	0.03	<0.01	<0.01
Endrin	0.02	<0.001	<0.001
Heptachlor	0.008	<0.001	<0.001
Heptachlor epoxide	0.008	<0.001	<0.001
Lindane	0.44	<0.001	<0.001
Methoxychlor	10.00	<0.002	<0.002
Toxaphene	0.50	<0.01	<0.01
2,4-D	10.00	<0.005	<0.005
2,4,5-TP (Silvex)	1.00	<0.005	<0.005
Semivolatile Organics			
Cresol (total)	200.0	<0.01	<0.01
1,4-Dichlorobenzene	7.5	<0.01	<0.01
2,4-Ninitrotoluene	0.13	<0.01	<0.01
Hexachlorobenzene	0.13	<0.01	<0.01
Hexachloro-1,3-butadiene	0.5	<0.01	<0.01
Hexachloroethane	3.0	<0.01	<0.01
Nitrobenzene	2.0	<0.01	<0.01
Pentachlorophenol	100.0	<0.05	<0.05
Pyridine	5.0	<0.01	<0.01
2,4,5-Trichlorophenol	400.0	<0.01	<0.01
2,4,6-Trichlorophenol	2.0	<0.01	<0.01

Table 12-13. Total Metals Data, Cobb County-Marietta Water Authority, Marietta, GA (Cobb County, 1994)

	Laboratory Data	
Compound	**Quarles (mg/kg)**	**Wyckoff (mg/kg)**
Arsenic	3.37	3.26
Cadmium	3.37	3.26
Chromium	25	20
Copper	29	35
Lead	<5	<5
Mercury	<0.01	<0.01
Molybdenum	5	9
Nickel	15	15
Selenium	0.75	0.44
Zinc	53	58

pesticides, herbicides, volatile organics, and semivolatile organics have been conducted. Table 12-12 presents the most recent TCLP data for the Quarles and Wyckoff plant residuals. The toxics levels have been extremely low for all samples.

The levels of heavy metals in the sludge have also been determined and are shown in Table 12-13.

Additional analyses of nutrients and other metals, pesticides and PCBs, triazine herbicides, and reactivity for cyanide and sulfide were conducted. The data from these analyses show that this material has a very low nutrient content. Neither pesticides, PCBs, nor herbicides were detected. The reactivity for cyanide and sulfide was below the EPA criteria limits stated in 40 CFR Part 261.23. The data for these analyses are reported in Tables 12-14, 12-15, 12-16 and 12-17.

The Quarles Plant has two gravity thickeners, both 40 feet in diameter with a sidewater depth of 14 feet. The sludge conditioning takes place in two tanks with 10,000-gallon capacities each. Pebble lime is slaked and fed to the conditioning tank. Approximately 10 to 15 percent of the dry weight of the dewatered sludge is lime. The plant controls conditioning by maintaining a pH of 11.7 to 12.0 in the conditioned sludge. The plant has two filter presses containing 50 chambers each. The total volume per press is 60 cubic feet. The plates are spray washed approximately every load and acid washed every month.

The dewatered sludge is discharged into dump trucks and hauled to a sludge storage site at the Wyckoff Plant. The sludge storage site is an approximately 15-acre fenced site with gravel roads and a washdown station.

The Wyckoff Plant has three gravity thickeners. Two units are 40 feet in diameter with a 14-foot sidewater depth. The third unit is 65 feet in diameter with a 14-foot sidewater depth. The sludge conditioning tank capacity is 7,000 gallons. Hydrated lime is mixed with water and fed to the conditioning tank. Approximately 10 to 15

Table 12-14. Other Analyses, Cobb County-Marietta Water Authority, Marietta, GA (Cobb County, 1994)

	Laboratory Data	
Compound	Quarles (mg/L)	Wyckoff (mg/L)
TKN nitrogen	2,000	5,600
Nitrate nitrogen	300	750
Ammonia nitrogen	100	300
Phosphorus	900	2,000
Potassium	700	500
Sulfur	3,400	2,300
Calcium	161,000	228,000
Magnesium	4,800	6,000
Sodium	100	200
Iron	14,600	18,600
Aluminum	46,000	67,300
Manganese	2,100	2,260
Calcium carbonate equivalency	250,600	338,500
Chloride	700	12,500
Boron	5	15
TOC	66,000	146,000

Table 12-15. Pesticides and PCBs (Solids), Cobb County-Marietta Water Authority, Marietta, GA (Cobb County, 1994)

	Laboratory Data	
Compound	Quarles (μg/kg)	Wyckoff (μg/kg)
Aldrin	<5.0	<5.0
alpha-BHC	<5.0	<5.0
beta-BHC	<5.0	<5.0
delta-BHC	<5.0	<5.0
gamma-BHC (Lindane)	<5.0	<5.0
Chlordane (Technical)	<5.0	<5.0
4,4′-DDD	<5.0	<5.0
4,4′-DDE	<5.0	<5.0
4,4′-DDT	<5.0	<5.0
Dieldrin	<5.0	<5.0
Endosulfan I	<5.0	<5.0
Endosulfan II	<5.0	<5.0
Endosulfan sulfate	<5.0	<5.0
Endrin	<5.0	<5.0
Endrin aldehyde	<5.0	<5.0
Heptachlor	<5.0	<5.0
Heptachlor epoxide	<5.0	<5.0
Mothoxychlor	<5.0	<5.0
Mirex	<5.0	<5.0
Parathion	<5.0	<5.0
Toxaphene	<50.0	<50.0
PCB 1016	<20.0	<20.0
PCB 1221	<20.0	<20.0
PCB 1232	<20.0	<20.0
PCB 1242	<20.0	<20.0
PCB 1248	<20.0	<20.0
PCB 1254	<20.0	<20.0
PCB 1260	<20.0	<20.0

percent of the dry weight of the dewatered sludge is lime. The plant controls conditioning by maintaining a pH of 11.5 to 12.0 in the conditioned sludge. The plant has two filter presses containing 61 chambers, with a total volume per press of 74 cubic feet. The plates are spray washed when necessary and acid washed once per month.

The dewatered sludge is also discharged into dump trucks and taken to the sludge storage site adjacent to the Wyckoff Plant.

12.3.3 Ultimate Disposal—Land Application

The two plants together produce approximately 8,000 cubic yards of dewatered sludge per year. The solids content varies from 25 to 40 percent solids but is usually around 35 percent solids. Because of the size of the storage site it is necessary to haul sludge to the application sites two or three times per year.

The ultimate disposal is handled by a contractor for the Water Authority. The contractor uses trailers with a dumping capability and that haul 30 cubic yards per load. Usually, approximately 3,000 to 5,000 cubic yards of sludge are hauled during an application period, involving up to 170 truck loads.

The Authority has also contracted with a firm to obtain the land necessary for the application program and to coordinate the monitoring program. The firm has acquired under contract approximately 1,560 acres to receive the WTP residuals, which are called lime byproduct. As of this writing, approximately nine sites totaling 1,107 acres have received lime byproduct. The firm that handles the land application program meets with farmers who may be interested in receiving lime byproduct and discusses the program with them. The farmers' fields are sampled to determine their lime requirements. Fields that are included in the program must be of a certain size and slope to be acceptable. The farmers must also practice good soil management techniques in the operation of their farms.

Table 12-16. Triazine Herbicides, Cobb County-Marietta Water Authority, Marietta, GA (Cobb County, 1994)

	Laboratory Data	
Compound	Quarles (μg/kg)	Wyckoff (μg/kg)
Ametryn	<1.0	<1.0
Atraton	<1.0	<1.0
Atrazine	<1.0	<1.0
Cyanazine	<1.0	<1.0
Dipropetryn	<1.0	<1.0
Prometon	<1.0	<1.0
Prometryn	<1.0	<1.0
Propazine	<1.0	<1.0
Simazine	<1.0	<1.0
Simetryne	<1.0	<1.0
Terbuthylazine	<1.0	<1.0
Tertbutryn	<1.0	<1.0
Trietazine	<1.0	<1.0

Table 12-17. Reactivity, Solids, Cobb County-Marietta Water Authority, Marietta, GA (Cobb County, 1994)

	Laboratory Data	
Parameter	Quarles (μg/kg)	Wyckoff (μg/kg)
Reactivity, cyanide	<0.25	<0.25
Reactivity, sulfide	<5.0	<5.0

Actual land application involves transporting the dewatered sludge to a farmer's field and discharging it in a location mutually agreeable to the farmer and the contractor. The sludge is then placed into a manure spreader using a front end loader. The manure spreader is a side-discharging unit that evenly spreads the material across the field.

The scheduling of the land application is determined by the farmer involved. The farmer's planting and harvesting periods, condition of his fields (particularly the soil dryness), and the availability of the lime byproduct are considerations. Because of the demand for the material, some farmers have had to wait a year before they receive any material.

Lime byproduct is applied to both pasture and cropland. Approximately 88 percent of the land under contract is pasture, with the remainder in crops. No restrictions exist on the types of crops or pasture where the material is applied, except for the agronomic pH requirements of the plants involved. Because the land used in the program is not a dedicated site, and because the metal and nutrient content of the material is very low, there are no concerns about heavy metal or nitrogen loadings.

The application rate for the material depends on the lime requirements of the field and the lime content of the plant residuals. The wet tons per acre value ranges from approximately 34 to 100 metric tons per hectare (mt/ha) (15 to 45 tons/acre) and dry tons per acre ranges from 11 to 34 mt/ha (5 to 15 tons/acre). The CCE application rate ranges from approximately 2 to 5.6 mt/ha (1 to 2.5 tons/acre). As of this writing, it has not been determined how frequently lime byproduct can be applied on the same field, but it will probably be no sooner than every three years. One farmer in the program had material applied in the fall of 1990 and still had good pH levels in his fields in the spring of 1993. The material is normally surface applied with no soil incorporation. On some farms a drag has been run over the field after application to further break up the material and speed up its incorporation into the soil. Normally, the material is washed by rain into the soil within a few weeks.

The monitoring program that the Authority uses for its land application program includes sampling and analysis of the residuals and the affected soil and vegetation. Ground-water monitoring is not conducted because the program does not use a dedicated site. Stream monitoring is conducted on the stream in the drainage area where the sludge storage yard is located.

The sludge is sampled quarterly for percent solids, percent CCE, pH, and TCLP for certain metals. Semiannual sampling is conducted for TCLP for a wider list of metals, total metals, and nutrients. Annual sampling is conducted for chlorotriazine herbicides. TCLP for organics is presently sampled every other year pending the results of the analyses. PCBs, total organic carbon (TOC), total Kjeldahl nitrogen (TKN), phosphorus, ammonia nitrogen, and nitrate nitrogen have all been sampled once.

Soil monitoring is conducted on ten fields per year including one field on each farm in the program for that year. Each field is sampled before and after application of residuals. Among the analyses conducted are total metals and standard soil fertility tests. The soil analysis parameters for the metals tests are: aluminum, arsenic, boron, cadmium, calcium, chromium, chloride, copper, iron, lead, magnesium, manganese, mercury, molybdenum, nickel, potassium, selenium, sodium, sulfur, and zinc. The standard soil fertility test includes tests for: percent organic matter, nitrate, phosphorus, potassium, calcium, magnesium, cation exchange capacity (CEC), pH, and percent base saturation. Soil monitoring was conducted in a 1991 pilot study of three soil plots. One plot had commercial lime applied, one had lime byproduct applied, and one had no lime applied. Except for the pH and calcium content of each plot, no significant differences were noted between them. Table 12-18 presents the soil data from that study.

Table 12-18. Pilot Study, Soil Data, Cobb County-Marietta Water Authority, Marietta, GA (Cobb County, 1994)

Component (ppm)	Soil Without Sludge	Soil With Agricultural Lime[a]	Soil With Sludge
pH	5.7	6.6	6.7
Nitrogen	22,900	18,400	23,150
Phosphorous	3,100	3,200	3,750
Potassium	24,900	13,300	20,280
Aluminum	80	30	90
Cadmium	<0.5	<0.5	<0.5
Chromium	<5	<5	<5
Copper	9	8	9
Nickel	<5	<5	<5
Lead	<5	<5	<5
Calcium	2,900	2,875	3,500
Magnesium	1,800	1,800	1,800
Boron	8	7	9
Iron	106	96	119
Manganese	68	80	60
Molybdenum	0.4	0.9	0.6
Sodium	100	100	100
Zinc	22	25	22

[a] Only four samples were taken on these plots compared with six samples on the other plots. This sampling difference accounts for lower levels for nitrogen, potassium, aluminum, and calcium.

Plant tissue analyses will be conducted on several fields during the first year of the monitoring program. At least one field on each farm in the program during that year will be sampled. Row crops are sampled before harvest, and pastures, or forages, are sampled prior to grass dormancy. Standard plant analyses will be conducted on the samples. The parameters for these analyses are as follows: aluminum, boron, cadmium, calcium, chloride, copper, iron, lead, magnesium, manganese, molybdenum, nickel, nitrogen, phosphorus, potassium, sodium, sulfur, and zinc.

Plant tissue analyses were conducted on corn grown on the same three soil plots from the aforementioned 1991 pilot study. No significant differences were found in the tissue analyses conducted or in the crop yields on the plots. Table 12-19 presents the plant tissue data from that study. The average yields for the plots ranged from 122.9 bushels/acre to 130.9 bushels/acre with the lime byproduct-treated plot having the highest yield.

Table 12-19. Pilot Study, Plant Tissue Data, Cobb County-Marietta Water Authority, Marietta, GA (Cobb County, 1994)

Component (ppm)	Plant Tissue Without Sludge	Plant Tissue With Agricultural Lime[a]	Plant Tissue With Sludge
Nitrogen	22,900	18,400	23,200
Phosphorous	3,100	3,200	3,800
Potassium	20,000	13,250	20,300
Aluminum	80	30	90
Cadmium	<0.5	<0.5	<0.5
Chromium	<5	<5	<5
Copper	9	8	9
Nickel	<5	<5	<5
Lead	<5	<5	<5
Calcium	2,900	2,900	3,500
Magnesium	1,800	1,800	1,800
Boron	8	7	9
Iron	106	96	119
Manganese	68	80	60
Molybdenum	0.4	0.9	0.6
Sodium	100	100	100
Zinc	22	25	22

[a] Only four samples were taken on these plots compared with six samples on the other plots. This sampling difference accounts for lower levels for nitrogen, potassium, aluminum, and calcium.

Stream monitoring is conducted quarterly on sampling sites above and below the lime byproduct storage area at the Wyckoff Plant. The analyses evaluate pH, calcium, aluminum, and turbidity levels. The data at present show little variation between the upstream and downstream sites.

12.3.4 Disposal Costs

The costs for dewatering residuals include personnel, equipment, chemical, and maintenance costs. The total 1992 costs for Quarles and Wyckoff plants were approximately $360,000. These costs do not include employee benefits or the costs for capital improvement projects.

The costs for the land application program are on a cost-per-ton basis, excluding laboratory costs. The elements of the program include land acquisition, transportation to the application site, spreading, monitoring support, and annual reporting. During 1992, this program cost both plants a total of approximately $170,000.

The laboratory monitoring costs for the first year of the CCMWA's program will be about $10,000. In subsequent years the Authority will do many of the analyses themselves and eliminate others that have not been detected in the testing. Based on this reduced program, external laboratory costs should be reduced to about $2,000 per year.

The economic benefits gained from the land application program are based on the avoided costs of tipping fees that must be paid to the local landfill. These costs are expected to increase in the years to come with the increase in regulations and the difficulty in obtaining landfill sites. Based on landfill tipping fees of $32.50/ton, the annual cost would be about $220,000 per year. Because the dewatering costs are incurred whether the material is land applied or placed in a landfill, the savings per year are approximately $50,000. During the first year of the present monitoring program the savings will be less due to the higher monitoring costs.

12.4 Case Study 4: Treatment of Residuals at the Lake Gaillard Water Treatment Plant, North Branford, Connecticut

12.4.1 Facility Information

12.4.1.1 Major Plant Modifications

- Added four new rapid sand dual media filters to the plant, thereby expanding the number of filters in the plant to 16, and increasing plant capacity from 60 mgd to 80 mgd.
- Expanded flocculation basins by 20 percent by adding a new flocculation basin system.
- New control valves and magmeters were added to outflow of equalization basins to optimize the clarifier loading rate.
- Doubled the number of clarifiers.
- Installed new sludge pumps-rotary lobe (variable frequency drive).
- Installed new recycle pumps and controls (variable frequency drive).
- Installed new sludge lagoon/freeze-thaw/sand drying beds.

12.4.1.2 Sludge Lagoon/Freeze-Thaw/Sand Drying Beds

Lagoon Construction

- Site constraints influenced lagoon construction, i.e., wetlands required lagoons to be constructed with gabions (1 on 1 slopes).
- Site includes approximately 20,000 square feet of lagoon bottom in each of the four lagoons.
- Lagoons were initially constructed with natural soil bottoms and gravity decant chambers.
- Following unsatisfactory performance of the first lagoon once placed into service, all lagoons were retrofitted with a sand/filter fabric/gravel PVC piping underdrain system.

12.4.1.3 Dewatering Method as Designed

Backwash Water From Equalization Basin to Gravity Clarifiers

- Clarifiers Parkson (lamella) and EIMCO (stainless “Flex-Klear”) parallel units.
- Clean water recycled to head of the plant.
- Sludge thickened to approximately 2 percent solids.

Sludge Pumped by Rotary Lobe Pumps to Natural Silt Bottom/Freeze-Thaw Lagoons

Evaporation

Decant Through Gravity Decant Chambers Back to Equalization Basins

12.4.1.4 Retrofitted Dewatering Method

- Gravity clarifiers (same).
- Sludge pumped by hose pumps to sand drying bed/freeze-thaw lagoons with piped underdrain systems. Rotary lobe pumps could not achieve 2 percent solids based on our piping arrangement, and anthracite particles caused the pumps to fail.
- Evaporation.
- Decant through gravity chambers (limited).
- Gravity drainage through underdrain system.
- Some initial short circuiting was noted at the interior lagoon perimeter. Modifications were made to the filter fabric/sand/gabion interface that solved this problem. This type of short circuiting (at any vertical interior lagoon boundary condition) is something that needs to be considered very carefully during lagoon design and construction.
- Installed vertical filter drains to enhance dewatering from sludge column to sand underdrain system.
- Conducted summer furrowing of thin sludge layers to increase evaporation by breaking up the top skim layer, which greatly increased the effective surface area of the sludge.
- Conducted mechanical (in-lagoon) winter mixing of the sludge to enhance the freeze-thaw process. This method allows freezing of three to inch-inch layers each night.
- Following the freeze/thaw process, the sludge at approximately 40 percent solids is removed from the lagoon by front end loader and trucked to a monofill

owned and operated by the Lake Gaillard Water Treatment Plant.

12.4.2 Lessons Learned

- Sludge will not dewater by evaporation and decanting alone.
  - Some means of drainage at the bottoms of lagoons is required. At this plant a sand/gravel underdrain system with PVC piping was installed.
  - A skim layer forms on the surface of sludge causing a rapid decrease in the rate of evaporation.
  - The skim layer should be broken up during the summer months to enhance evaporation and simultaneously to increase the surface area. Surface cracking down to the sand layer (while beneficial to enhance dewatering and channel away rain) is not in itself sufficient to dewater the sludge. In our case, furrowing with a small tractor over thin (8 to 10 inches) layers in summer produced good results, i.e., 25 to 30 percent solids.
  - Decanting a lagoon while it is being filled with new sludge may result in short circuiting. This happened at this plant and is unacceptable because decant water is directed back to the equalization basins/recycle system, causing undesireable turbidity to the head waters of the WTP.
- Sludge blinds the sand layer at the surface boundary of the sand but not completely. Blinding does not occur throughout the layer of sand depth.
  - The rate of underdrain flow will decrease over time given a relatively constant head of sludge in the lagoon. The headloss is due to a blinding layer immediately at the sand layer surface boundary.
  - What appears to be an anaerobic (dark) layer of sludge becomes evident at the sand layer surface boundary. At this plant, the dark layer was approximately one to several inches thick after several (summer) months of initial application.
  - In the case of this plant, what appeared to be an (orange) iron precipitate formed at the filter fabric layer between the bottom of the sand layer and the top of the gravel underdrain pipe layer. Over time this may blind the filter fabric.
  - Vertical filter drains, designed by the plant operator, were found to enhance the dewatering process by automatically decanting surface water (and some sludge column), and directing it to the sand underdrain system.
- Sand layers should be approximately 12 to 18 inches thick, to protect the filter fabric below the sand layer.
  - Heavy equipment entering the lagoon digs into the sand and tears the fabric if it is too close to the sand surface. At this plant, the initial sand depth was 6 inches and we experienced substantial tearing of the filter fabric. After the initial cleaning of the lagoons the sand layer was increased to 12 inches. Approximately two inches of sand was removed during the lagoon cleaning process.
  - Track equipment is superior to rubber tire equipment for working on sand drying beds.
  - Careful attention to the sand/filter fabric/gabion connection at the lagoon interior perimeter needs to be made during construction to avoid short circuiting of sludge into the underdrain system.
- Lagoons divided into several “compartments” or “sub lagoons” are beneficial because sludge drying and cracking do not proceed effectively until loading of the lagoon is complete. This has not been implemented at Lake Gaillard WTP at this time.
  - Loading should be completed in thin layers (three to four inches) to enhance summer drying or winter freeze-thawing. Lagoon compartments will accommodate this.
  - Sufficient area is the key to the successful operation of sand drying/freeze-thaw beds with underdrains. This plant is seeking ways to expand its workable lagoon area.
- Only the top three or four inches of sludge will freeze. This frozen layer then insulates the sludge below and stops the freezing process.
  - Winter mixing by use of a track-mounted backhoe or other similar equipment can expose fresh sludge for freezing every night. A layer of approximately three inches can be frozen every night at temperatures of 20°F or less.
  - A thin layer of snow will insulate the sludge lagoon and prevent the freezing process.
- The sludge lagoons produce a “musty,” “low tide” odor during the summer months.
- Elevated levels of chloroform were noted in the underdrain water during the spring melt period (1992). This may be due to prechlorination at the plant.

12.5 Case Study 5: Disposal of Water Treatment Plant Residuals From the San Benito Water Plant, Brownsville, Texas

The San Benito Water Plant is located in the Rio Grande Valley near Brownsville, Texas. The water plant is a mixed media, rapid filtration, surface water plant. The source of water for the San Benito plant is the Rio Grande River. Water from this river is pumped into a system of canal-type lakes called Resacas. The design capacity of the plant is 6 mgd. The maximum daily demand is approximately 2.7 mgd in the winter, and 4 mgd in the summer. WTP residuals are removed from the San Benito plant as follows:

1. Sludge is removed from two upflow clarifier units to storage lagoons through a drain system at the bottom of the clarifiers. Backwash water (205,000 gallons per day) containing residuals is also directed to the storage lagoon. The lagoon is equipped with a pump to return excess water to the headworks, to be treated again.
2. Settled sludge is retained in the storage lagoons for approximately 3 months.
3. At the end of the 3-month period, the plant uses a tractor loader and dump truck to remove the thickened sludge to a local landfill.

12.5.1 Residuals Handling Facilities

The storage lagoons are lined with concrete below ground level. Sludge is removed from the treatment plant to the lagoons by gravity flow, thus saving pumping costs.

The plant does not use any sludge thickening processes. The supernatant is removed after the sludge settles, and the sludge is exposed to the sun. Approximately 2 weeks before removing the sludge from the lagoons, a divider is used to separate the basin into two halves. The portion not receiving flow dries in the hot Texas sun. The sludge is then dry enough for a tractor loader to remove it to a landfill.

12.5.2 Final Disposal

The sludge is disposed of in a local landfill that is classified to receive this type of waste. At the time of this writing, the operation of the landfill had recently been privatized. Instead of being run by the City of San Benito, it is to be operated by a private company. The San Benito WTP intends to continue disposing of its residuals at this landfill.

12.5.3 Disposal Costs

Labor costs form the greatest portion of the San Benito plant's total disposal costs. Normally, sludge removal to the landfill requires a week of two people working full-time. In 1992, labor costs totaled an estimated $5,000. Equipment costs were estimated at $1,000, including fuel and maintenance costs. The plant's estimated total annual disposal costs for 1992 were $6,000.

12.6 Case Study 6: Management of Water Treatment Plant Residuals in the Chicago Area, Chicago, Illinois

The major source of drinking water in the metropolitan Chicago area is Lake Michigan. Seven water treatment plants draw water from Lake Michigan to supply drinking water to Chicago and various neighboring communities. The largest water system is the City of Chicago system, which serves a population of more than 4.6 million people in Chicago and 118 suburban municipalities in both Cook and DuPage counties. The Chicago water system includes two of the largest water treatment plants in the world (Jardine Water Purification Plant (WPP) and South Water Filtration Plant (WFP)), three water intake cribs located 2 to 2.5 miles offshore in Lake Michigan, 11 pumping stations, and thousands of miles of water mains. The finished water from the Chicago water system is supplied for drinking and industrial use.

Six other municipalities along the lakeshore (Evanston, Glencoe, Kennilworth, Northbrook, Wilmette, and Winnetka) obtain water directly from Lake Michigan and maintain their own treatment plants, which serve a combined population of approximately 0.44 million people.

Twenty-five years ago, the residuals generated from all of the abovementioned WTPs were disposed of by direct discharge into Lake Michigan. In keeping with a mandate to prevent pollution of Lake Michigan, the Metropolitan Water Reclamation District of Greater Chicago (the District) adopted an ordinance that contained a provision prohibiting the discharge of waste of any kind to Lake Michigan. In the early 1970s, the District took action requiring WTPs to eliminate the practice of discharging their residuals into Lake Michigan. By 1974, all of these discharges were diverted into the sewerage system of the District.

The District covers over 872 square miles within Cook County, and serves 125 municipalities including the City of Chicago. The population served by the District is 5.2 million people. In addition, a nondomestic equivalent of 4.5 million people is also served by the District, resulting in a total sewage treatment load equivalent to almost 10 million. The wastewater collection system operated by the District consists of more than 500 miles of intercepting sewers and 23 pumps stations. The District owns and operates seven water reclamation plants (WRPs) with a combined maximum capacity of about 2,000 million gallons. They include the world's largest activated sludge plant, the Stickney WRP, which has a maximum capacity of 1,440 mgd. All of the other District WRPs are also activated sludge treatment plants. About 500 dry tons of sludge solids are generated per day from these plants.

This case study focuses on the Jardine WPP, which discharges its residuals into the District's Stickney WRP.

12.6.1 Description of the Jardine Water Purification Plant

The Jardine WPP is located in Chicago on Lake Michigan, just north of the Chicago River. It has a design

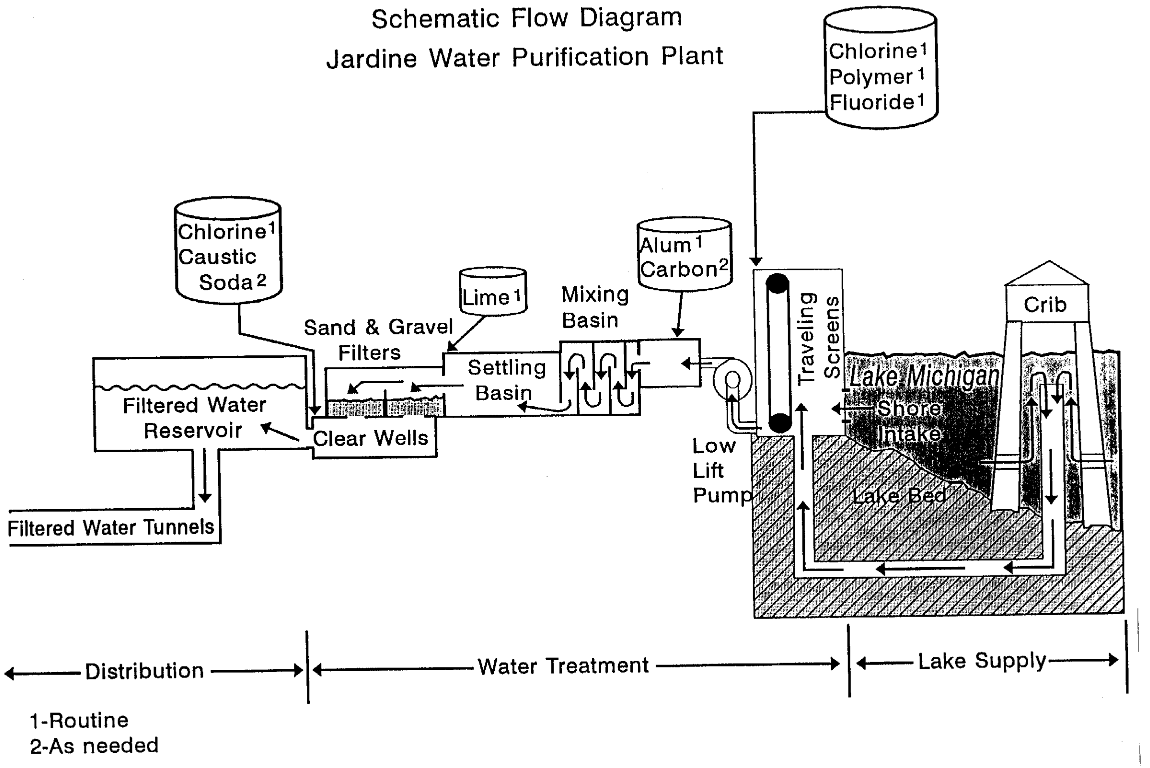


Figure 12-10. Schematic flow diagram of Jardine Water Purification Plant, Chicago, IL (MWRDGC, 1994).

capacity of 960 mgd based on a sand filter design rate of 2 gpm/ft^2. A schematic flow diagram is shown in Figure 12-10. The plant has 192 sand filters, each with a nominal filter rate of 5 mgd. Water is drawn from Lake Michigan either through a 20-foot diameter tunnel from a crib located 2.5 miles out in the lake, or directly through a shore intake at the plant.

The unit processes at the Chicago WTPs include traveling screens to remove fish, aquatic weeds, and trash. The water from Lake Michigan is then pumped upward 21 feet to permit gravity flow through the WFP. Chlorine, fluoride, and polyelectrolyte are added after the screen in the intake basin.

Alum is added ahead of flocculation basins where slow-speed mixers enhance the flocculation process. The flocculation water then passes into settling basins with a detention time of 4 hours. The settled water is then conveyed to sand filters. Hydrated lime is added ahead of the filters to minimize corrosion in the water distribution system. After filtration, the water flows to filtered water clearwells where additional chlorine is added, along with caustic soda. From the clearwells the water is taken to a 10-acre filtered water reservoir prior to being conveyed to the distribution system.

Sludge from the settling basins of the WFP is removed and collected in a holding tank. The filter backwash is returned to the settling basins. Organic material removed from the intake screens is macerated and pumped into the sludge holding tank. The collected sludge in the holding tank is pumped at a controlled rate into an interceptor sewer of the District. A schematic of the sludge management system at the Jardine WPP is presented in Figure 12-11.

12.6.2 Residuals Handling and Disposal

Table 12-20 shows the volume and strength of the water treatment residuals discharged by Chicago's Jardine and South WFP into the District's WRPs from 1984 to 1992. The Jardine WPP's residuals volume ranged from 192 to 378 million gallons per year, with suspended solids ranging from 7,605 to 11,148 tons per year (20.1 to 30.6 dt/day). These quantities are roughly 5 to 6 percent of the residuals produced by the District's WRPs. The biochemical oxygen demand (BOD) is relatively low, less than 200,000 pounds per year, because the majority of the sludge solids is inorganic. Similar data for the other six WFPs, which treat Lake Michigan water and discharge their sludges into the District system, are summarized in Table 12-21.

The District has, under its Sewage and Industrial Waste Ordinance, imposed local limits for several heavy metals (see Table 12-22). A summary of the metals concentrations observed in the WTP residuals is shown in Table 12-23. A comparison of the residual metals concentrations with the limits imposed by the Ordinance indicates

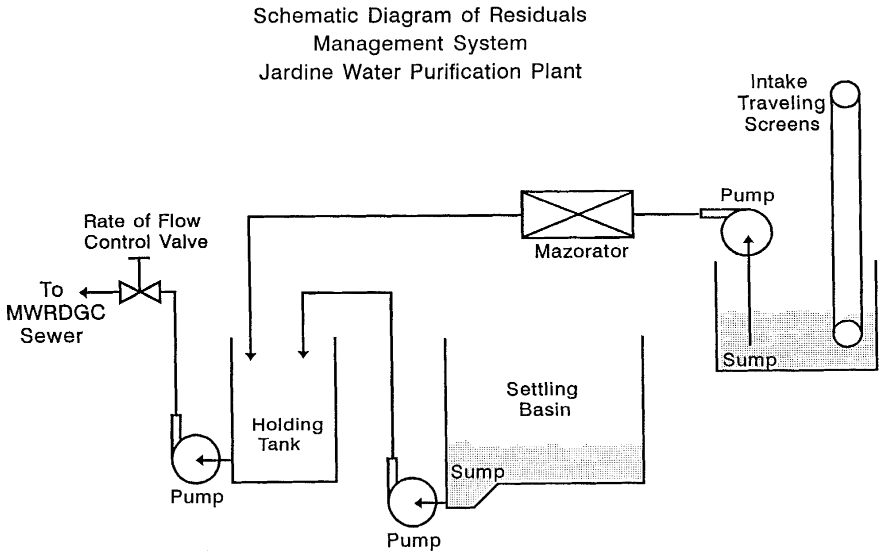


Figure 12-11. Schematic diagram of residuals management system, Jardine Water Purification Plant, Chicago, IL (MWRDGC, 1994).

Table 12-20. Water Treatment Sludges Discharged by the City of Chicago WTPs to the District, Chicago, IL, 1984–1992 (MWRDGC, 1994)

	Jardine WPP Plant			South WFP		
Year	Volume (MG)	BOD (lb)	TSS (lb x 10^3)	Volume (MG)	BOD (lb)	TSS (lb x 10^3)
1984	205.66	20,441	22,296	355.9	117,955	10,956
1985	215.53	226,447	21,118	336.5	179,637	12,036
1986	191.59	182,156	18,405	396.3	185,074	8,920
1987	200.82	166,822	17,869	471.3	146,008	13,069
1988	176.75	271,268	15,209	339.6	185,567	11,222
1989	244.65	272,126	17,588	315.1	131,116	11,570
1990	378.26	276,049	16,417	323.9	165,435	12,913
1991	362.87	238,360	18,439	326.5	119,815	11,486
1992	239.1	305,342	19,937	382.3	95,174	12,522

that the residuals metal concentrations are within the Ordinance limits, except for lead at the Evanston, Winnetka, and Wilmette plants.

12.6.3 Disposal Costs

The costs for discharging wastes into the District's treatment system are based on volume, BOD, and suspended solids. In 1992, the user charge rates for tax-exempt entities such as the WFPs were:

Volume	$131.60 per million gallons
BOD	$148.34 per thousand pounds
Suspended solids	$132.74 per thousand pounds

A summary of the annual charges paid by the City of Chicago's two WTPs for the period of 1984 through 1992 is presented in Table 12-24. In 1992, the Jardine WPP and South WFP paid $2,723,317 and $1,726,593 in user charges, respectively. In 1990, the Jardine WPP paid $1,837,940, or an average of $5,035 per day, to annually dispose of 378 million gallons of residuals containing 16.4 million pounds of suspended solids (approximately 22.5 tons per day). During 1990, the WTP supplied 214 billion gallons of filtered water to its consumers. From 1984 through 1992, the average cost to the City of Chicago for disposal of the WTP residuals was $0.00858 per thousand gallons of finished water, or $224 per ton of dry solids.

For the years 1984 through 1992 the residuals disposal costs paid by the six water treatment plants on Lake Michigan not operated by the City of Chicago are summarized in Table 12-25. The total costs are based upon volume and BOD as well as solids; thus, the variation in

Table 12-21. Water Treatment Sludges Discharged by Various Suburban WTPs to the District, Chicago, IL, 1984–1992 (MWRDGC, 1994)

	Evanston			Wilmette			Northbrook		
Year	Volume (MG)	BOD (lb)	TSS (lb x 10^3)	Volume (MG)	BOD (lb)	TSS (lb x 10^3)	Volume (MG)	BOD (lb)	TSS (lb x 10^3)
1984	2.056	7,928	528	18.06	4,497	679	3.77	5,320	353
1985	2.032	14,436	456	13.661	4,494	705	3.77	3,716	340
1986	1.286	3,652	364	15.406	11,716	755	2.407	3,212	363
1987	1.601	5,480	851	11.055	3,436	788	1.414	2,521	334
1988	0.898	3,383	466	12.629	5,918	685	0.777	1,962	176
1989	7.335	28,996	2,447	12.740	6,418	597	0.976	1,703	183
1990	6.345	26,547	2,238	11.326	5,671	565	1.709	4,396	481
1991	3.801	17,109	2,350	13.034	6,957	820	1.889	3,386	512
1992	8.521	31,711	2,641	11.244	3,881	644			

	Winnetka			Kennilworth			Glencoe		
Year	Volume (MG)	BOD (lb)	TSS (lb x 10^3)	Volume (MG)	BOD (lb)	TSS (lb x 10^3)	Volume (MG)	BOD (lb)	TSS (lb x 10^3)
1984	0.683	1,563	330	0.44	60	33	0.324	354	102
1985	0.631	1,761	220	0.44	171	22	0.342	661	52
1986	1.694	2,057	321	0.44	408	22	0.388	618	58
1987	1.361	1,879	198	0.44	200	24	0.398	915	129
1988	1.096	3,678	271	0.44	270	86	0.359	777	47
1989	1.111	1,712	114	0.44	220	34	0.337	744	63
1990	1.423	4,161	246	0.44	387	45	0.238	835	97
1991	1.075	5,444	308	0.44	288	55	0.292	613	108
1992									

Table 12-22. Local Limits on Dischargers Into District Sewarage Systems, Chicago, IL[a] (MWRDGC, 1994)

Pollutant	Maximum Concentration (mg/L)
Cadmium	2.0
Chromium (total)	25.0
Chromium (hexavalent)	10.0
Copper	3.0
Cyanide (total)	5.0
Fats, oils, and greases (FOG)	250.0
Iron[b]	250.0
Lead	0.5
Nickel	10.0
Zinc	15.0
pH range: not lower than 5.0 or greater than 10.0	

[a] Sewage and Waste Control Ordinance as amended September 5, 1991.

[b] Discharges from domestic WTPs that supply potable water to the general public are exempt from this limitation for iron.

Table 12-23. Concentrations of Metals Found in WTP Sludges Discharged Into the District, Chicago, IL (MWRDGC, 1994)

	Concentration in mg/L							
	Total							
Water Treatment Plant	Cd	Cu	Cr	Fe	Ni	Ph	Hg[a]	Zn
Jardine	0.01	0.02	0.04	7.1	0.0	0.03	0.3	0.1
South (Chicago)	0.02	0.08	0.06	11.1	0.1	0.07	0.9	0.2
Evanston	0.29	1.40	1.44	866	1.3	4.89	1.5	5.2
Wilmette	0.18	0.58	0.83	225	0.7	2.37	0.7	2.6
Winnetka	0.23	1.48	0.90	464	0.9	4.35	2.6	2.8
Northbrook	0.03	0.06	0.05	8.5	0.1	0.09	0.1	0.1

[a] Hg concentration in µg/L.

the costs per dry ton is due to differences in the amount of water discharged with the solids. The average annual cost of discharging WTP residuals from the Chicago facilities ranged from $159/dt in 1984 to $275/dt in 1992 (see Table 12-24). Similarly, the average annual cost for discharging sludges from the other treatment plants (see Table 12-25) ranged from $164/dt in 1984 to $246/dt in 1991.

Table 12-24. User Charge Costs for Disposal of Water Treatment Residuals to the Metropolitan Water Reclamation District, Chicago, IL, 1984–1992 (MWRDGC, 1994)

Year	Jardine WRP	Cost/dt	South WFP	Cost/dt	Average Cost/dt
1984	$1,744,204	$156	$887,063	$162	$159
1985	$1,617,996	$153	$944,780	$157	$155
1986	$1,317,075	$143	$669,625	$150	$147
1987	$1,511,031	$169	$1,135,339	$174	$172
1988	$1,188,896	$156	$893,417	$159	$158
1989	$1,762,091	$200	$1,167,141	$202	$201
1990	$1,837,940	$224	$1,442,383	$223	$224
1991	$2,335,947	$253	$1,464,025	$255	$254
1992	$2,723,317	$273	$1,726,593	$276	$275

Key
dt = dry ton.

Table 12-25. Water Treatment Sludges Discharged From Various Water Treatment Plants to the District From 1984 Through 1992 (MWRDGC, 1994)

	Evanston			Wilmette			Northbrook			Winnetka			Kennilworth			Glencoe		
Year	Volume (MG)	BOD (lb)	TSS (lb x 10^3)	Volume (MG)	BOD (lb)	TSS (lb x 10^3)	Volume (MG)	BOD (lb)	TSS (lb x 10^3)	Volume (MG)	BOD (lb)	TSS (lb x 10^3)	Volume (MG)	BOD (lb)	TSS (lb x 10^3)	Volume (MG)	BOD (lb)	TSS (lb x 10^3)
1984	2.056	7,928	528	18.06	4,497	679	3.77	5,120	353	0.683	1,563	330	0.44	60	33	0.324	354	102
1985	2.032	14,436	456	13.661	4,494	705	3.77	3,716	340	0.631	1,761	220	0.44	171	22	0.342	661	52
1986	1.286	3,652	364	15.406	11,716	755	2.407	3,212	363	1.694	2,057	321	0.44	408	22	0.388	618	58
1987	1.601	5,480	851	11.055	3,436	788	1.414	2,521	334	1.361	1,879	198	0.44	200	24	0.398	915	129
1988	0.898	3,383	466	12.629	5,918	685	0.777	1,962	176	1.096	3,678	271	0.44	270	86	0.359	777	47
1989	7.335	28,996	2,447	12.740	6,418	597	0.976	1,703	183	1.111	1,712	114	0.44	220	34	0.337	744	63
1990	6.345	26,547	2,238	11.326	5,671	565	1.709	4,396	481	1.423	4,161	246	0.44	387	45	0.238	835	97
1991	3.801	17,109	2,350	13.034	6,957	820	1.889	3,386	512	1.075	5,444	308	0.44	288	55	0.292	613	108
1992	8.521	31,711	2,641	11.244	3,881	644												

Chapter 13
Waste Minimization and Reuse

In the process of treating a raw water supply for potable water use, contaminants are removed from the raw water typically in the form of residual solids, brines, and concentrates. The treatment and disposal of these waste streams have been discussed extensively in prior chapters. Historically, these waste streams have been stored, treated, and then disposed of.

Many years ago, the typical method of disposal was direct discharge into a nearby receiving stream, or lagoon storage. Often, solids would accumulate in an onsite lagoon for years. Once capacity was reached, the waste stream would simply flow directly into an overflow structure, onto adjacent properties, or into receiving streams, with little or no settling. This would typically leave the water utility with an immediate problem, requiring removal of solids and, often, onsite disposal with few or no control measures. The potential for polluting the environment, both aquatic and terrestrial, was high.

Over the years, there has been an increased emphasis on minimizing negative impacts on the environment through pollution prevention and reuse of materials. This is most evident in the adoption of the National Pollutant Discharge Elimination System (NPDES) program. The implementation of the NPDES program has eliminated direct discharge to a receiving stream as a viable option for many water supply systems. Uncontrolled onsite disposal is generally no longer permissible, requiring the utility to construct a monofill, discharge to a sanitary sewer system, or transport the material to a nearby sanitary landfill.

While all of these are viable options, implementation is becoming more problematic. Construction of an onsite monofill requires available land and is costly both to construct and to monitor. Discharge to a sanitary sewer is often limited by a wastewater treatment plant's capacity to accept more flow and inert solids, both of which might affect the wastewater treatment process.

The regulatory framework that controls the wastewater industry has also become increasingly stringent, requiring higher levels of wastewater treatment and increasing costs. In addition, the impact of 40 CFR Part 503 on the wastewater industry has caused increased concerns over accepting new material into the waste stream that may be perceived to limit disposal or reuse options.

Transport and disposal to a nearby landfill is also increasingly difficult. Many landfills have reached capacity or are nearing capacity, requiring utilities to look beyond the immediate area for disposal options and significantly increasing costs. Often, the associated costs of disposal drive the utility to consider means of reducing quantities of residuals generated and to give more consideration to recovery or reuse options.

The net result of these changes is that utilities need to develop long-term, flexible management plans. They need to consider innovative techniques for reducing the quantities of waste, and they need to implement reuse options. Ultimately, this will result in more cost-effective solutions for residuals handling and will improve public acceptance. The dilemma, however, is that some of the treatment techniques for reducing residual waste stream volume or recovering chemical aids (i.e., coagulant or lime recovery) have had limited application over the years and, at present, may be costly. The markets for beneficial reuse of residuals at this time are limited.

Some of the available waste minimization, recovery, and reuse options that apply primarily to residuals generated from the coagulation/filtration or lime softening processes are discussed below.

13.1 Waste Minimization

13.1.1 Process Modifications

Perhaps the best means of reducing waste is to minimize or optimize the types and quantities of coagulants and chemical aids used and to optimize associated mixing conditions. Control at the source is the most effective first step and should be evaluated in all facilities. Process optimization allows facilities to achieve a balance between water quality, chemical addition, and physical parameters. Ultimately, the quality of the raw water supply will dictate the type of treatment provided. For example, EPA's proposed Disinfection/Disinfectant Byproduct Rule may require utilities to increase coagulant dosages to maximize water quality benefits and reduce the potential for disinfectant byproduct formation.

13.1.2 Dewatering

Dewatering—both mechanical and nonmechanical—is a potential step in the process of minimizing waste. Details of this process are provided in Chapter 4.

Several mechanical dewatering techniques are available and employed throughout the country. These typically include filter presses and centrifuges. Mechanical dewatering significantly reduces the residuals volume by removing some of the water, thereby increasing the percent solids content. Several other mechanical dewatering processes have the potential for increased use in the future. As discussed by Malmrose and Wolfe (1994) and presented in *Slib, Schlamm, Sludge* (Cornwell and Koppers, 1990), these include:

- Mechanical freeze-thaw machine
- Centrifuge
- Diaphragm filter press
- Continuous high-pressure (CHP) filter press
- Tubular filter press
- Cross-flow microfiltration process
- Hi-Compact press
- Compactor

The applicability of these processes depends on raw water quality, coagulant practices, and desired dewatered cake characteristics. The need for chemical conditioning or pretreatment is also site specific. Most of these techniques were developed for application in the wastewater industry. As a result, little information is available relative to performance in the drinking water industry. A brief description of these alternatives is provided below.

The mechanical freeze-thaw machine employs freezing as the method of separating the solid and liquid phases. This method is employed primarily in Europe; no operating facilities exist in the United States. Energy-intensive and generally costly, the process is initiated by freezing through refrigeration, followed by a thawing process. This conditioning process, when followed by other dewatering processes, can achieve a solids content of 30 to 50 percent (Malmrose and Wolfe, 1994).

Scroll and spinning centrifuges have been used in the United States for many years; however, they typically have been associated with wastewater operations. Solid-liquid separation occurs through application of centrifugal force. Application in the drinking water industry has typically yielded a dewatered cake with a solids content on the order of 15 percent. Recent improvements in the equipment, as well as the availability of more operational data, indicate that the dewatered cake can achieve a solids content in excess of 20 percent, depending on the solids characteristics, velocity gradient (g value), and feed concentrations. Thickening to 2 to 4 percent prior to centrifugation yields improved results. Centrifuges are usually lower in cost, smaller in size, and easier to operate than many other types of mechanical dewatering equipment.

The diaphragm filter press is similar to a basic filter press, which comprises filter plates that are pressed together to form a chamber. The plates are lined with a porous cloth. Residuals are pumped into the chamber; the solids are retained by the cloth liner, and the liquid fraction is pushed through as the driving pressure increases with continued pumping. In the diaphragm press, the liner is replaced with a membrane which, when pressurized, aids the dewatering process by squeezing the solids retained in the chamber. This batch process can achieve a dewatered cake with a solids content in excess of 30 percent (Cornwell and Koppers, 1990).

The CHP filter press combines elements of both the belt and pressure filters. An initial gravity thickening zone is followed by wedging the residuals between two belts at a low pressure (35 psi). High pressure, on the order of 300 psi, is then applied through cylinders that exert pressure to a mobile armored belt (Malmrose and Wolfe, 1994).

The tubular filter press and cross-flow microfiltration process feed residuals, under pressure, to a tubular matrix. The tubes are composed of porous material, and the solids are retained on the tubes. In the tubular filter press, the solids are removed by rollers; the cross-flow microfiltration unit employs a vortex cleaning system (Malmrose and Wolfe, 1994). Conditioning prior to application on the presses may not be required. Limited experience indicates that a solids content in the 20 to 40 percent range can be achieved (Malmrose and Wolfe, 1994; Cornwell and Koppers, 1990).

The remaining processes, specifically the Hi-Compact press and the Compactor system, have limited full-scale applications, even in the wastewater industry. The Hi-Compact press requires prior thickening and dewatering to greater than 20 percent solids content.

13.1.3 Drying

As discussed in Chapter 4, a drying step would serve to further reduce the quantities of solids generated for reuse or disposal. Two processes for drying are the Carver-Greenfield and electroacoustical dewatering (EAD) processes. They are difficult to operate and potentially very costly because of elaborate equipment and high energy requirements. The Carver-Greenfield process, however, may be able to achieve a dewatered cake with a greater than 90 percent solids content, thereby reducing the volume of residuals significantly. Applicability to the drinking water industry is still somewhat unclear, however. Continued investigations are warranted.

13.2 Chemical Recovery

13.2.1 Coagulant Recovery

Coagulant recovery provides a method for recovering a resource (i.e., coagulant) and for minimizing waste by extracting aluminum or iron coagulants from the waste stream. Extraction is achieved by acidification, which puts the metals back into solution. Critical design and operations factors include extraction pH and acid contact time. Extraction pH is typically in the range of 1.8 to 3.0. Acid contact time of 10 to 20 minutes appears to be reasonable based on full-scale operations data and laboratory testing (Saunders and Roeder, 1991). Another critical component is the amount of metal coming from the raw water compared with the coagulant. For example, aluminum associated with raw water solids is more difficult to dissolve than that associated with an alum coagulant.

The quality of the recovered coagulant is an important factor in determining the feasibility of coagulant recovery. The impact of coagulant reuse on treatment plant operation and resulting finished water quality must be given serious consideration. While the acidification process is beneficial for achieving dissolution of the coagulant, concentrations of coagulant impurities and raw water contaminants may also become dissolved and get recycled to the head of the plant. While these contaminants may again be removed from the water supply by the coagulant, settling, and filtration processes, the net result is a concentration of contaminants. This impact must be carefully considered as the regulatory climate becomes increasingly stringent.

Concentration and recycling of contaminants may present a problem in achieving compliance. Of particular interest is the recycling of organic material, which may increase disinfection byproduct formation potential. This may be due in part to the demonstrated need for increased chlorination when organic matter is recycled (Saunders and Roeder, 1991). Full-scale operations experience at the Vernon S. Wade plant in Athens, Georgia, also indicates an increased alkalinity demand as a result of using the recovered coagulant.

Coagulant recovery has been investigated in the United States over the last 30 to 40 years; however, few full-scale facilities are currently active (Saunders and Roeder, 1991). Saunders and Roeder (1991) have reviewed coagulant recovery extensively and present their findings in the American Water Works Association Research Foundation research report *Coagulant Review: A Critical Assessment*. Pilot- or laboratory-scale operations should be performed when evaluating the feasibility of implementing this option.

13.2.2 Lime Recovery

Lime recovery is accomplished through the recalcination process whereby the lime residual is dewatered and then burned to produce quicklime (Cornwell et al., 1987). As with other recovery processes, strong consideration must be given to the expected quality of the recovered product and to incorporating processes to remove impurities. For example, as presented in *Handbook of Practice: Water Treatment Plant Waste Management* (Cornwell et al., 1987), the first step is generally a purification process such as centrifugation.

After purification and dewatering of the calcium carbonate, a drying process is generally employed. A variety of furnaces are available, as well as other means of drying. Certainly, power and fuel costs are a key factor in evaluating the applicability of this process for a particular utility. As mentioned previously, the required purity of the quicklime must also be considered in the decision-making process.

13.3 Innovative Use and Disposal Options

Several alternative uses and disposal methods for residual solids are presented in *Slib, Schlamm, Sludge* (Cornwell and Koppers, 1990) and in recent articles by Copeland et al. (1994, 1995). These alternatives include options such as land application and reclamation, supplementation of a soil matrix, blending with compost material, and brick and cement production. Implementation of these alternative methods may prove to be more cost-effective than traditional disposal methods and will likely achieve a high degree of public acceptance.

The regulatory framework for acceptance of these options varies among states. It appears that few states have a well-defined program for approving these uses as they apply to water treatment plant (WTP) residuals. Disposal of wastewater treatment plant (WWTP) biosolids is more clearly regulated than disposal of WTP residuals, primarily because WWTP biosolids are covered under Part 503 regulations. While Part 503 specifically excludes WTP residuals, many states are applying similar approaches and standards to the drinking water industry. As the characteristics of water and wastewater residuals are very different from one another, more definitive guidance and regulations are needed to address WTP residuals specifically. Key factors and considerations associated with these innovative use and disposal alternatives are discussed below.

13.3.1 Beneficial Use

Beneficial use options for WTP residuals include land application and reclamation, supplementation of a soil matrix, and blending with compost material. Potential concerns have been raised about aluminum levels in alum coagulant residuals and their tendency to bind up available phosphorus in the soil. Phosphorus is needed as a nutrient for vegetation. To overcome this potential deficiency, supplemental fertilization may be necessary. This may be of particular relevance when using residuals for land application, soil matrix supplementation, or blending with compost material. Some potential benefits associated with using coagulant solids include improvements in aeration and moisture retention (Copeland et al., 1994, 1995).

Soil conditioning with lime softening residuals is also a beneficial use. Farmers often add calcium carbonate to soils to counteract the chemical reduction that results from nitrogen application and to maintain an appropriate soil pH. Any difficulty associated with this use is due more to farmers' lack of familiarity with handling the residual solids than to the residuals themselves.

Blending with compost requires further definition of mix ratios and maintenance of temperature requirements. As discussed in Copeland et al. (1994), different mix ratios must be evaluated and the optimum defined, both in terms of the existing regulatory framework as well as the end-use requirements. Maintenance of appropriate temperature during composting is essential to a successful project.

13.3.2 Disposal Options

Some research and actual implementation has been conducted in the development of co-products such as bricks and cement. Again, limited application has occurred in the United States. Willingness of all parties to conduct preliminary research as well as bench- and pilot-scale studies is essential.

One successful project involves the Santa Clara Valley Water District, where bricks are constructed using coagulant residuals. The project took a great deal of effort and time, however. Careful consideration of associated costs, benefits, and other economic factors must also be considered in the development of a co-product.

Chapter 14
References

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

National Technical Information Service
5285 Port Royal Road
Springfield, VA 22161
703-487-4650
1-800-553-NTIS

Albrecht, A.E. 1972. Disposal of alum sludges. JAWWA 64(1):46-52.

Alfa-Laval Sharples. 1994. Manufacturer's product literature. Alfa-Laval Sharples, 955 Mearns Rd., Warminster, PA 18974.

Ambrose, R.B., Jr., and T.O. Barnwell, Jr. 1989. Environmental software at the U.S. Environmental Protection Agency's Center for Exposure Assessment Modeling. Environmental Software 4(2):76-93.

American Society of Civil Engineers (ASCE)/American Water Works Association (AWWA). 1990. Water treatment plant design, 2nd ed. New York, NY: McGraw-Hill.

ASTM. 1982. Standard definitions of terms relating to water. D1129. Philadelphia, PA: American Society for Testing and Materials.

AWWA. 1986. 1984 water utility operating data. Denver, CO: American Water Works Association.

AWWA/AWWARF. 1992. Water industry database: Utility profiles. Report no. 20293. Denver, CO: American Water Works Association/American Water Works Association Research Foundation. November.

American Water Works Association Research Foundation (AWWARF). 1969. Disposal of wastes from water treatment plants. JAWWA 61:541, 619.

AWWA Water Treatment Residuals Management Committee, American Society of Civil Engineers. 1991. State regulatory approaches for water plant wastes. Proceedings of the 1991 Annual AWWA Conference and Exposition. Presented at Preconference Seminar, Current Perspectives on Water Treatment Plant Sludge Disposal: Practices and Regulations, Philadelphia, PA. Denver, CO: American Water Works Association. June 23.

Andelman, J.B. 1973. Incidence, variability, and controlling factors for trace elements in natural, fresh waters. In: Singer, P.C., ed. Trace metals and metal-organic interactions in natural waters. Ann Arbor, MI: Ann Arbor Science Publishers. pp. 57-88.

Anderson, D.T., and M.J. Hammer. 1973. Effects of alum addition on activated sludge biota. Water and Sewage Works. January.

Andritz Ruthner, Inc. 1994. Manufacturer's product literature. Andritz Ruthner, Inc., 1010 Commercial Blvd. S., Arlington, TX 76017.

Assadian, N.W., and L.B. Fenn. 1995. Beneficial use of combined biosolids and water plant residuals on agricultural land. In: Proceedings from the AWWA/WEF Joint Residuals Management Conference, Kansas City, MO. July 23-26.

Baker, J.P. 1982. Effects on fish of metals associated with acidification. In: Proceedings of the International Symposium on Acidic Precipitation Fishing Impacts in Northeastern N. America. Ithaca, NY: Cornell University. pp. 165-176.

Baker, J.P., and C.L. Schofield. 1982. Aluminum toxicity to fish in acidic waters. Water, Air, and Soil Pollution 18:289-309.

Barth, E.F., M.B. Ettinger, B.V. Salotto, and G.N. McDermott. 1965. Summary report on the effects of heavy metals on the biological treatment processes. J. Water Pollut. Control Fed. 37(1):86-96.

Bassett, R.L., and D.C. Melchoir. 1990. Chemical modeling of aqueous systems. In: Melchoir, D.C., and R.L. Bassett, eds. Chemical modeling in aqueous systems II. American Chemical Society Symposium Series 416. Washington, DC. pp. 1-14.

Bennett, D.L. 1978. The efficiency of water treatment processes in radium removal. JAWWA 70(12):698-701.

Birchall, J.D., C. Exley, J.S. Chappel, and M.J. Phillips. 1989. Acute toxicity of aluminum to fish eliminated in silicon-rich acid waters. Nature 338(6211):146-148.

Bowen, P.T., M.K. Jackson, and R.A. Corbitt. 1991. Sludge treatment, usage, and disposal. J. Water Pollut. Control Fed. 63(4):406-414.

Bratby, J., C. Baugh, and W. Ambrose. 1993. Water plant sludge dewatering by recessed chamber filter press. In: Proceedings of the AWWA/WEF Joint Residuals Management Conference, Phoenix, AZ. December 5-8.

Brink, W.L., R.J. Schliekelman, D.L. Bennett, C.R. Bell, and I.M. Markwood. 1978. Radium removal efficiencies in water treatment processes. JAWWA 70(1):31-35.

Brinkhurst, R.O., A.L. Hamilton, and H.B. Herrington. 1968. Components of the bottom fauna of the St. Lawrence, Great Lakes. No. PR 33. Great Lakes Institute, University of Toronto, Toronto, Ontario, Canada.

Brobst, R. 1994. Survey of 12 land-applied WTP residuals. Unpublished data. U.S. Environmental Protection Agency Region 8. Denver, CO.

Brown, D.J.A. 1983. Effect of calcium and aluminum concentrations on the survival of Brown Trout *(Salmo trutta)* at low pH. Bull. Env. Contam. Toxicol. 30:582-587.

Brown and Caldwell, Inc. 1990. City of Phoenix water residuals management study. Study produced for the City of Phoenix, AZ.

Brusewitz, S. 1984. Aluminum. Stockholm, Sweden: University of Stockholm, Institute of Theoretical Physics. 203:138.

Buckler, D.R., P.M. Mehrle, L. Cleveland, and F.J. Dwyer. 1987. Influence of pH on the toxicity of aluminum and other inorganic contaminants to the East Coast striped bass. Water, Air, and Soil Pollution. 35:97-106.

Bugbee, G.J., and C.R. Frink. 1985. Alum sludge as a soil amendment: Effects on soil properties and plant growth. Connecticut Agricultural Experiment Station Bulletin 827.

Burrows, W.D. 1977. Aquatic aluminum: Chemistry, toxicology, and environmental prevalence. CRC Critical Reviews in Environmental Control. Boca Raton, FL: CRC Press. pp. 167-216.

Cairns, J., Jr. 1977. Quantification of biological integrity. In: Ballentine, R.K., and L.J. Guarraia, eds. The integrity of water. U.S. Environmental Protection Agency, Office of Water and Hazardous Materials, Washington, DC. U.S. Government Printing Office Stop No. 055-001-01068-1.

Cairns, J., Jr., J.S. Crossman, K.L. Dickson, and E.E. Herricks. 1971. The recovery of damaged streams. ASB Bulletin 18(3):79-106.

Calkins, R.J., and J.T. Novak. 1973. Characteristics of chemical sludges. JAWWA 65(6):523.

Campbell, P.G.C., M. Bisson, R. Bougie, A. Tessier, and J. Villeneuve. 1983. Speciation of aluminum in acidic freshwaters. Anal. Chem. 55:2246-2252.

CDM. 1989. City of Norfolk, 37th Street Water Treatment Plant, sludge disposal study. Raleigh, NC: Camp Dresser & McKee. January.

Chang, A.C., A.L. Page, and J.E. Warneke. 1983. Soil condition effects of municipal sludge compost. J. Environ. Eng. 109:574.

Che, M.D., T.J. Logan, S.J. Traina, and J.M. Bigham. 1988. Properties of water treatment sludges and their effectiveness as agricultural limestone substitutes. Columbus, OH: Ohio State Agricultural Research and Development Center. 60:674-680.

Chisholm, J.L., and S.C. Downs. 1978. Stress and recovery of aquatic organisms as related to highway construction along Turtle Creek, Boone County, West Virginia. U.S. Geological Survey Water Supply Paper 2055. Washington, DC.

Clifford, D.A. 1990. Removal of radium from drinking water. In: Cothern, C.R., and P. Rebers, eds. Radon, radium, and uranium in drinking water. Chelsea, MI: Lewis Publishers. pp. 225-247.

Clifford, D.A., W. Vijjesworapu, and S. Subramonian. 1988. Evaluating various adsorbents and membranes for removing radium from groundwater. JAWWA 80(7):94.

Cobb County-Marietta Water Authority. 1994. Data from the Cobb County-Marietta Water Authority, 1660 Barnes Mill Road, Marietta, GA 30062.

Colorado Department of Health. 1986. Domestic sewage sludge regulations, 5CCR 1003-7.

Colorado Department of Health. 1990. Colorado regulation pertaining to solid waste disposal sites and facilities promulgated under the Solid Waste Disposal Sites and Facilities Act. Title 30, Part 1, CRS as amended.

Colorado Springs Utilities. 1994. Data from the Colorado Springs Utilities, Water Resources Department, P.O. Box 1103, Colorado Springs, CO 80947.

Copeland, A.M., C. Vandermeyden, and D.A. Cornwell. 1994. Beneficial use alternatives for water treatment plant residuals. National Conference on Environmental Engineering, Boulder, CO.

Copeland, A.M., C. Vandermeyden, and D.A. Cornwell. 1995. Residuals benefits. Civil Engineering Journal. January: 70-72.

Cornwell, D., M.M. Bishop, R.G. Gould, and C. Vandermeyden. 1987. Handbook of practice: Water treatment plant waste management. Denver, CO: American Water Works Association Research Foundation.

Cornwell, D.A., and H.M.M. Koppers, eds. 1990. Slib, schlamm, sludge. Denver, CO: American Water Works Association Research Foundation and KIWA Ltd.

Cornwell, D.A., and R. Lee. 1993. Recycle stream effects on water treatment. Denver, CO: American Water Works Association Research Foundation.

Cornwell, D.A., and R.G. Lee. 1994. Waste stream recycling: Its effect on water quality. JAWWA 86(11):50-63.

Cornwell, D.A., C. Vandermeyden, G. Dillow, and M. Wang. 1992. Landfilling of water treatment plant coagulant sludges. Denver, CO: American Water Works Association Research Foundation/American Water Works Association.

Dentel, S., M. Abu-Orf, and N. Griskowitz. 1993. Guidance manual for polymer selection in wastewater treatment plants. Water Environment Research Federation.

Dempsey, B.A., and H.A. Elliott. 1990. Land application of water treatment sludges. In: Proceedings of the 44th Purdue Industrial Waste Conference, Purdue University. W. Lafayette, IN: Lewis Publishers.

Dixon, K.L. 1993. Hazardous waste classification. In: Proceedings of the AWWA/WEF 1993 Joint Residuals Management Conference. Phoenix, AZ. December 5-8.

Dixon, K.L., R.G. Lee, and R.H. Moser. 1988. Water treatment plant residuals: A management strategy for the Pennsylvania region. Voorhees, NJ: American Water Works Service Company. June 23.

Donovan, J.F., and W.E. Toffey. 1993. Impact of industrial pretreatment programs on biosolids quality in Boston and Philadelphia. In: Proceedings of the AWWA/WEF 1993 Joint Residuals Management Conference, Phoenix, AZ. December 5-8.

DPRA, Inc. 1992. Telephone survey of municipal sewer rates, September 15-17. DPRA, Inc., 332 Minnesota St., St. Paul, MN 55101.

Dragun, J. 1988. The soil chemistry of hazardous materials. p. 77.

Driscoll, C.T., Jr. 1985. Aluminum in acidic surface waters: Chemistry, transport, and effects. Environ. Health Perspectives 63:93-104.

Driscoll, C.T., J.P. Baker, J.J. Bisogni, and C.L. Schofield. 1980. Effects of aluminum speciation on fish in dilute acidified waters. Nature 284:161-164.

Driscoll, C.T., Jr., and W.D. Schecher. 1988. Aluminum in the environment. In: Sigel, H., ed. Metal ions in biological systems: Aluminum and its role in biology, Vol. 24. New York, NY: Decker Publishing. pp. 1-41.

Driscoll, C.T., Jr., B.J. Wyskowski, and P. Destaffan. 1987. Chemistry and transfer of aluminum in a forested watershed in the Adirondack region of New York, USA. Presented at the 194th National Meeting of the American Chemical Society, New Orleans, LA.

Dulin, D.E., and W.R. Knocke. 1989. The impact of incorporated organic matter on the dewatering characteristics of aluminum hydroxide sludges. JAWWA 81(5):74.

EET, Inc. 1992. Residuals Management Study for West Virginia American Water Company. Submitted to the American Water Works Service Company, Inc. Newport News, VA: Environmental Engineering & Technology, Inc. November.

EIFAC. 1969. Water quality criteria for European freshwater fish. European Inland Fisheries Committee.

Elliot, H.A., B.A. Dempsey, and J. DeWolfe. 1988. Land disposal of drinking water sludge. Presented at 1988 International Winter Meeting of the American Society of Agricultural Engineers, December 13-16.

Elliott, H.A., B.A. Dempsey, and P.J. Maille. 1990a. Content and fractation of heavy metals in water treatment sludges. J. Environ. Qual. 19:330-334.

Elliott, H.A., and B.A. Dempsey. 1990b. Land application of water treatment sludges: Impact and management. Final report. Denver, CO: American Water Works Association Research Foundation.

Elliott, H.A., and L.M. Singer. 1988. Effect of water treatment sludge on growth and element composition of tomato *(Lycopersicon esculentum)* shoots. Communications in Soil Science and Plant Analysis 11:969-980.

Emig, D.K. 1979. Excessive alum causes indigestion. Water and Waste Engineering. April.

Esernick, E.L., J.L. Maas-Diepeveen, and C.J. Leeuwen. 1991. Combined effects of metals—an ecotoxicological evaluation. Water Res. 25(6):679-688.

Evans, R.L., D.H. Schnepper, and T.H. Hill. 1979. Impact of wastes from a water treatment plant: Evaluation procedures and results. Illinois State Water Survey Circular 135. p. 39.

Evans, R.L., D.H. Schnepper, T.E. Hill, and D.L. Hullinger. 1982. Waste from the water treatment plant at Alton and its impact on the Mississippi River. Illinois State Water Survey Circular 156. p. 62.

Evans, R.E., S.B. Brown, and T.J. Hara. 1988. The effects of aluminum and acid on the gill morphology in rainbow trout, *Salmo gardneri*. Environ. Biol. Fishes 22(4):299-311.

Foley, J.F., K. Folks, and K.J. Yakich. 1989. Treating water filter plant sludge at a wastewater treatment plant. Presented at the 1989 AWWA/WPCF Joint Residuals Management Conference, San Diego, CA. August 13-16.

Foster, W.S. 1975. Get the water out of alum sludge. American City and County 90(9).

Foth, H.D., and B.G. Ellis. 1988. Soil fertility. New York, NY: Wiley & Sons.

Freeman, R.A., and W.H. Everhart. 1971. Toxicity of aluminum hydroxide complexes in neutral and basic media to rainbow trout. Trans. Am. Fish. Soc. 100:644.

Freeman, R.A., and W.H. Everhart. 1973. Recovery of rainbow trout from aluminum poisoning. Trans. Am. Fish Soc. 100:152.

Funk, W.H., H.R. Gibbons, and S.K. Bhagat. 1977. Nutrient interaction by large scale aluminum sulfate treatment. Presented at Conference on Mechanics of Lake Restoration, University of Wisconsin, Madison, WI, April 25-28.

Gammon, J.R. 1970. The effect of inorganic sediment on stream biota. Water Poll. Cont. Res. Series. 18050 DWC 12/70. Washington, DC.

Gandhi, R.L. 1992. Slurry and sludge piping. In: Piping handbook, 6th ed. New York, NY: McGraw-Hill.

Geertsema, W.S., W.R. Knocke, J.T. Novak, and D. Dove. 1994. Long-term effects of sludge application to land. JAWWA 86(11):64-74.

George, D.B., S.G. Berk, V.D. Adams, E.L. Morgan, R.O. Roberts, C.A. Holloway, R. Lott, L.K. Holt, R.S. Ting, and A.W. Welch. 1991. Alum sludge in the aquatic environment. Denver, CO: American Water Works Association Research Foundation.

George, D.B., S.G. Berk, V.D. Adams, R.S. Ting, R.O. Roberts, L.H. Parks, and R.C. Lott. 1995. Toxicity of alum sludge extracts to a freshwater alga, protozoan, fish, and marine bacterium. In: Archives of Environmental Contamination and Toxicology 29(2): 149-158.

Goodnight, C.J. 1973. The use of aquatic macroinvertebrates as indicators of aquatic pollution. Transactions of the American Microscopical Society 92(1):1-13.

Goss, G.G., and C.M. Wood. 1988. The effects of acid and acid-aluminum exposure on circulating plasma cortisol levels and other blood parameters in rainbow trout, *Salmo gairdneri*. J. Fish Biol. 32(1):63-76.

Grabarek, R.J., and E.C. Krug. 1987. Silviculture application of alum sludge. JAWWA 79(6):84-88.

Grabarek, R.J., and E.C. Krug. 1989. Silviculture application of alum sludge. In: Sludge: Handling and disposal. Denver, CO: American Water Works Association. pp. 97-101.

Great Lakes Upper Mississippi River Board of State Public Health and Environmental Managers. 1992. Recommended standards for water works. New York, NY: Health Education Services.

Hahn, N.A., Jr. 1989. Disposal of radium removal from dirty water. In: Sludge: Handling and disposal. Denver, CO: American Water Works Association.

Hall, R.J., C.T. Driscoll, G.E. Likens, and J.M. Pratt. 1985. Physical, chemical and biological consequences of episodic aluminum additions to a stream. Limnology and Oceanography 30:212-218.

Hall, W.S., and L.W. Hall, Jr. 1989. Toxicity of alum sludge to *Ceriodaphnia dubia* and *Pimephales promplas.* Bull. Env. Contam. Toxicol. 42(5):791-798.

Harberg, B., and P. Heppler. 1993. Co-application of water treatment plant residuals and wastewater plant biosolids to dry land winter wheat in Colorado. In: Proceedings of the AWWA/WEF Joint Residuals Management Conference, Phoenix, AZ. December 5-8.

Havas, M. 1985. Aluminum bioaccumulation and toxicity to *Daphnia magna* in soft water at low pH. Can. J. Fish. Aquat. Sci. 42:1741-1748.

Havas, M., and T.C. Hutchinson. 1983. Effects of low pH on the chemical composition of aquatic invertebrates from tundra ponds at Smoking Hills, N.W.T., Canada. Can. J. Zool. 61:241-249.

HDR Engineering, Inc. 1995. Study to develop BCT for NPDES permit limits for water treatment plants. Draft report to the City of Phoenix, Water Services Department, Phoenix, AZ.

Heil, D.M. 1988. The influence of water treatment sludges on the growth of sorghum-sudangrass. M.S. thesis. Colorado State University, Fort Collins, CO.

Heil, D.M., and K.A. Barbarick. 1989. Water treatment sludge influence on the growth of sorghum-sudangrass. J. Environ. Qual. 18:292-298.

Holmgren, G.G.S., M.W. Myer, R.H. Chaney, and R.B. Daniels. 1993. Cadmium, lead, zinc, copper, and nickel in agricultural soils of the United States of America. J. Environ. Qual. 22:335-348.

HPD, Inc. 1994. Product literature. Naperville, IL.

Hsu, C.Y., and W.O. Pipes. 1973. Aluminum hydroxide effects on wastewater treatment processes. JWPCF 45(4):681-697.

Hundt, T.R. 1986. Aluminum-fulvic acid interactions: Mechanisms and applications. In: Proceedings of the Annual AWWA Conference, Denver, CO. June 22-26. pp. 1199-1201.

Hunn, J.B., L. Cleveland, and E.E. Little. 1987. Influence of pH and aluminum on developing brook trout in low calcium water. Environmental Pollution 43:63-73.

Illinois Department of Energy and Natural Resources. 1987. Wastes from water treatment plants: Literature review, results of an Illinois survey, and effects of alum sludge application to cropland. Peoria, IL.

Illinois Department of Nuclear Safety. 1984. Management of water treatment sludge containing elevated levels of radium.

Infilco Degremont. 1994. Manufacturer's product literature. Infilco Degremont, 2924 Emergy Wood Parkway, Richmond, VA 23294.

Ingram, W.T. 1969. Glossary: Water and wastewater control engineering. APHA, American Society of Civil Engineers, American Water Works Association, Water Pollution Control Federation. p. 40.

Jelinek, R.T., and T.J. Sorg. 1988. Operating a small full-scale ion exchange system for uranium removal. JAWWA 80(8):79-83.

Public Works. 1992. Job postings. July.

John Carollo Engineers, Inc. 1994. Unpublished material. John Carollo Engineers, Inc., 3877 N. Seventh St., Suite 400, Phoenix, AZ 85014.

Karlson-Norrgren, L., I. Borklund, O. Lujungberg, and P. Runn. 1986. Acid waters and aluminum exposure: Experimentally induced gill lesions in brown trout, *Salmo trutta.* J. Fish Diseases 9:11-25.

Kaverkamp, J.F., W.A. MacDonald, W.R. Lillie, and A. Lutz. 1983. Joint toxicity of mercury and selenium in *Salmonid* eggs. Arch. Environ. Contam. Toxicol. 12(4):415-419.

Kaviraj, A., and S.K. Konar. 1983. Lethal effects of mixtures of mercury, chromium, and cadmium on fish. Environ. Ecol. 1(3): 153-158.

Kawczyinski, E., and V. Achterrman. 1991. A water industry data base report. In: Proceedings of the AWWA/WEF Joint Residuals Management Conference, Research Triangle Park, NC.

Kay, J. 1986. Cadmium accumulation and protein binding patterns in tissues of the rainbow trout, *Salmo gairdneri*. Conference on High-Affinity Metal-Binding Proteins in Non-mammalian Species. Environ. Health Perspect. 65:133-139.

King, C.H., C.P. Houck, and L.L. Deadmond. 1988. Land application of Kodak Colorado Division Wastewater Treatment Plant sludge. Presented at Residuals Management Specialty Conf. Water Pollution Control Federation, Athens, GA. April 19-20.

Knocke, W.R., J.R. Harmon, and B.E. Dulin. 1987. Effects of coagulation on sludge thickening and dewatering. JAWWA 79:89.

Knocke, W.R., M.M. Ghosh, and J.T. Novak. 1980. Vacuum filtration of metal hydroxide sludges. J. Environ. Eng. Division, ASCE. 106(2):363.

Knocke, W.R., C.M. Dishman, and G.F. Miller. 1993. Measurement of chemical sludge floc density and implications related to sludge dewatering. Water Environ. Res. 65:735.

Koorse, S.J. 1993a. The role of residuals disposal laws in treatment plant design. JAWWA 85(10):57.

Koorse, S.J. 1993b. Toxic regulations take hold. Water Environment and Technology. January.

Koplish, D.E., and M.E. Watson. 1991. Demonstration and feasibility of combined dewatering of alum and waste sludges and the economic benefits of a single site facility. Presented at the 1991 AWWA/WPCF Joint Residuals Management Conference, Durham, NC. August 11-14.

Koplish, D.E., J. McMahon, and J. Valek. 1995. Efficiencies of centralized residuals dewatering and co-disposal. In: Proceedings of the AWWA/WEF Joint Residuals Management Conference, Kansas City, MO. July 23-26.

Lagvankar, A.L., and R.S. Gemmell. 1968. A size-density relationship for flocs. JAWWA 60(9):1040.

Lamb, D.S., and G. Bailey. 1981. Acute and chronic effects of alum to Midge larva. Bulletin Env. Contam. Toxicol. 27:59.

Lapedes, D.N. 1978. Dictionary of scientific and technical terms, 2nd ed. New York, NY: McGraw-Hill.

Lassovszky, P., and S. Hathaway. 1983. Treatment technologies to remove radionuclides from drinking water. Preconference report for the National Workshop on Radioactivity in Drinking Water. Washington, DC: U.S. Environmental Protection Agency.

League of Minnesota Cities. 1991. A research memo for city officials: Minnesota water and sewer rates. League of Minnesota Cities, 183 University Ave., E., St. Paul, MN 55101.

Lee, S.Y., and E.A. Bondietti. 1983. Removing uranium from drinking water by metal hydroides and anion-exchange resin. JAWWA 75(10):536-540.

Lee, R.G., H. Eckhardt, N. Wortel, and H.M.M. Koppers. 1990. Toxic concerns regarding water plant wastes. In: Cornwell, D., and H.M.M. Koppers, eds. Slib, schlamm, sludge. Denver, CO: American Water Works Association Research Foundation.

Lin, S.M. 1988. Effects of alum sludge on corn and soybeans. In: Proceedings of the Joint CSCE-ASCE National Conference on Environmental Engineering, Specialty Conference. July 13-15.

Lin, S.D. 1989. Impacts of alum sludge discharge from a mid-sized water treatment plant on a small stream. Presented at the 1989 AWWA/WPCF Joint Residuals Management Conference, San Diego, CA. August 13-16.

Lin, S., and C. Green. 1987. Wastes from water treatment plants: Literature review, results of an Illinois survey, and effects of alum sludge application to cropland. Illinois State Water Survey. November.

Logsdon, G.L., ed. 1991. Slow sand filtration. Task Committee Report. New York, NY: American Society of Civil Engineers.

Lowry, J.D., and J.E. Brandow. 1985. Removal of radon from water supplies. J. Environ. Eng. 3(4):511-527.

Mackenthun, K.M. 1973. Toward a cleaner aquatic environment. Washington, DC: U.S. Government Printing Office.

Malate, H. 1986. Effects of aluminum in hard, acid water on metabolic rate, blood gas tensions and sonic status in rainbow trout. J. Fish Biol. 29:187-198.

Malmrose, P.E., and T.A. Wolfe. 1994. Recent advances in technologies for dewatering coagulant residuals. AWWA Conference on Development of Programs for Engineering Tomorrow's Water Systems, Cincinnati, OH.

Mance, G. 1987. Pollution threat of heavy metals in aquatic environments. New York, NY: Elsevier Applied Science.

Martell, A.E., and R.J. Motekaitus. 1989. Coordination chemistry and speciation of Al(III) in aqueous solution. In: Lewis, T.E., ed. Environmental Chemistry and Toxicology of Alum. Chelsea, MI: Lewis Publishers.

McDonald, D.G., R.L. Walker, and P.R.H. Wilkes. 1983. The interaction of environmental calcium and low pH on the physiology of the rainbow trout, *Salmo gairdneri*. J. Eper. Biol. 102:141-155.

McNatt, R.M. 1977. Zooplankton species composition, abundance, and distribution in a desert, pumped-storage reservoir. Ph.D. Dissertation. Arizona State University, Tempe, AZ.

McTigue, N., and D.A. Cornwell. 1994. The hazardous potential of activated carbons used in water treatment. Denver, CO: American Water Works Association Research Foundation.

McTigue, N.E., D.A. Cornwell, and A.T. Rolan. 1989. Impact of water plant waste discharge on wastewater plants. Presented at the 1989 AWWA/WPCF Joint Residuals Management Conference, San Diego, CA. August 13-16.

Mesa WTP. 1994. Mesa Water Treatment Plant, Colorado Springs Utilities, Water Resources Department. P.O. Box 1103, Colorado Springs, CO 80947.

Metcalf & Eddy. 1991. Wastewater engineering: Treatment, disposal, and reuse, 3rd ed. New York, NY: McGraw-Hill.

Meyers, A.G., V.L. Snoeyink, and D.W. Snyder. 1985. Removing barium and radium through calcium cation exchange. JAWWA 77(5):60-66.

Mickley, M., R. Hamilton, L. Gallegos, and J. Truesdall. 1993. Membrane concentrate disposal. Denver, CO: American Water Works Association/American Water Works Association Research Foundation.

Missouri Department of Natural Resources. 1985. Agricultural use of municipal wastewater sludge—a planning guide. Division of Environmental Quality.

Mulbarger, M.C., S.R. Copas, J.R. Kordic, and F.M. Cosh. 1981. Pipeline friction losses for wastewater sludges. J. Water Pollut. Control Fed., Vol. 53. (August).

Mukhopadhayay, M.K., and S.K. Konar. 1985. Effects of copper, zinc, and iron mixture on fish and aquatic ecosystem. Environ. Ecol. 3(1):58-64.

Muniz, I.P., and H. Leivestad. 1980. Toxic effects of aluminum on the brown trout, *Salmo trutta.* In: Drablos, D., and A. Tollam, eds. Ecological impact on acid precipitation. Mysen, Norway: Grefslie Press.

MWRDGC. 1994. Metropolitan Water Reclamation District of Greater Chicago, 100 East Erie St., Chicago, IL 60611.

New Jersey DEPE. 1993. New Jersey statewide sludge management plan. New Jersey Department of Environmental Protection and Energy. November.

Novak, J.T. 1986. Historical and technical perspective of sludge treatment and disposal. Presented at Recent Advances in Sludge Treatment and Disposal, American Water Works Association. Preconference Seminar Proceedings, Washington, DC. June.

Novak, J.T. 1989. Historical and technical perspective at sludge treatment and disposal. In: Sludge: Handling and disposal. Denver, CO: American Water Works Association.

Novak, J.T., and D.C. Calkins. 1973. Sludge dewatering and its physical properties. JAWWA. 67(18).

O'Connor, J.T., and J.T. Novak. 1978. Management of water treatment plant residues. In: Proceedings of the AWWA Water Treatment Waste Disposal Seminar, Atlantic City, NJ. AWWA.

Olsen, R.D. 1975. The phytoplankton ecology of a desert lake. Ph.D. Dissertation. Arizona State University, Tempe, AZ.

Page, A., R.H. Miller, and D.R. Keeney, eds. 1982. Methods of soil sampling and analysis. Madison, WI: American Society of Agronomy/Soil Science Society of America.

Parrotta, M.J. 1991. Radioactivity in water treatment wastes: A U.S. Environmental Protection Agency perspective. JAWWA 83(4):134-140.

Parsons, J.M., and W.L. Waldrip. 1995. A land application study utilizing both lime treated alum sludge from a water treatment plant and aerobically digested biosolids. In: Proceedings of the AWWA/WEF Joint Residuals Management Conference, Kansas City, MO. July 23-26.

Peck, B., J. Russell, R. Ardizzone, and J. Hayes. 1993. A case history at the Val Vista Water Treatment Plant: Implementing a used water recovery system. Proceedings of the 1993 AWWA/WEF Joint Residuals Conference.

Pennak, R.W. 1989. Freshwater invertebrates of the United States: Protozoa to mollusca, 3rd ed. New York, NY: John Wiley & Sons.

Pine Valley WTP. 1994. Data from Pine Valley Water Treatment Plant, Colorado Springs Utilities, Water Resources Department. P.O. Box 1103, Colorado Springs, CO 80947.

Pontius, F.W., ed. 1990. Water quality and treatment: A handbook of community water supplies, 4th ed. New York, NY: McGraw-Hill.

Price, S.D., D.M. Owen, G.C. Cline, and K.E. Spiker. 1989. Water treatment plant residuals management for San Diego: The feasibility of co-disposal. In: Proceedings of the 1989 AWWA/WPCF Joint Residuals Management Conference, San Diego, CA. August 13-16.

Raghu, D., H.N. Hsieh, T. Neilman, and C.T. Yih. 1987. Water treatment plant sludge as landfill liner. Geotechnical Practice for Waste Disposal 1987, Geotechnical Special Publication 13. New York, NY.

Ramamoorthy, S. 1988. Effects of pH on speciation and toxicity of aluminum to rainbow trout *(Salmo gardneri).* Can. J. Fishery Aquatic Sci. 45:634-642.

Reed, R.C. 1975. The effects of alum sludge on anaerobic digestion. Master's thesis. Auburn University, Auburn University, AL.

Reh, C.W. 1978. Proceedings of the Twentieth Annual Public Water Supply Engineer's Conference—New Directions for Water Supply Design and Operation. Department of Civil Engineering, University of Illinois of Champaign-Urbana. American Water Works Association, Illinois Environmental Protection Agency, and Illinois Section of AWWA. Champaign, IL. April 3-5.

Reid, G.W., R.Y. Nelson, C. Hall, U. Bonilla, and B. Reid. 1968. Effects of metallic ions on biological waste treatment processes. Water and Sewage Works. July.

Rindt, B.A. 1973. The effects of alum sludge on primary sedimentation and anaerobic digestion. M.S. thesis. Auburn University, Auburn University, AL.

Roberts, M.H., and R.J. Diaz. 1985. Assessing the effects of alum sludge discharges into coastal water. In: Proceedings of the AWWA Seminar on Recent Advances in Sludge Treatment and Disposal, Washington, DC.

Robertson, R.F., and Y.T. Lin. 1981. Proceedings of the Twenty-Third Annual Public Water Supply Engineer's Conference—New Directions for Water Supply Design and Operation. Department of Civil Engineering, University of Illinois of Champaign-Urbana. American Water Works Association, Illinois Environmental Protection Agency, and Illinois Section of AWWA. Champaign, IL. April 21-23.

Robinson, M.P., and J.B. Witko. 1991. Overview of issues and current state-of-the-art water treatment plant waste management programs. 1991 Annual Conference Proceedings. American Water Works Association Water Quality for the New Decade, Philadelphia, PA. June 23-27.

Rolan, A.T., and J.C. Brown. 1973. Effect of water treatment plant sludge on the treatment of municipal wastewater. In: Proceedings of the 53rd Annual Meeting of the North Carolina Section of the American Waste Water Association and WPCA.

R.S. Means Co., Inc. 1991. 1992 Means building construction cost data, 50th ed. Kingston, MA: Construction Consultants and Publishers.

Rueter, J.G., Jr., K.T. O'Reilly, and R.R. Petersen. 1987. Indirect aluminum toxicity to the green alga *Scenedesmus* through increased cupricion activity. Environ. Sci. Technol. 21(5):435-438.

Saltonstall, C.W., Jr., and R.W. Lawrence. 1982. Calculation of the expected performance of reverse osmosis plants. Desalination 42:247-253.

Sanville, W.D., A.R. Gahler, J.A. Searcy, and C.F. Powers. 1976. Studies on lake restoration by phosphorus inactivation. EPA/600/3-76/041. U.S. Environmental Protection Agency, Corvallis Environmental Research Laboratory, Corvallis, OR.

Saunders, M., and M. Roeder. 1991. Coagulant review: A critical assessment. Denver, CO: American Water Works Association Research Foundation.

Schofield, C., and J.R. Trojnar. 1980. Aluminum toxicity to brook trout *(Salvelinus fontinalis)* in acidified waters. In: Toribara, T.Y., M.W. Miller, and P.E. Morrow, eds. Polluted Rain. New York, NY: Plenum Press. pp. 341-363.

Shacklette, H.T., and J.G. Boerngen. 1984. Element concentrations in soils and other surficial materials of the conterminous United States. U.S. Geological Survey Professional Paper 1270.

Smith, R.W., and J.D. Hem. 1972. Effect of aging on aluminum hydroxide complexes in dilute aqueous solutions. Chemistry of aluminum in natural water. U.S. Geological Survey Water Supply Paper 1827-D. Washington, DC: U.S. Government Printing Office.

Snoeyink, V.L., C.K. Jongeward, A.G. Myers, and S.K. Ritcher. 1989. Characteristics and handling of wastes from groundwater treatment systems. In: Sludge: Handling and disposal. Denver, CO: American Water Works Association.

Sorensen, E.M. 1991. Metal poisoning in fish. Boca Raton, FL: CRC Press.

Sorg, T.J., R.W. Forbes, and D.S. Chambers. 1980. Removal of radium-226 from Sarasota County, FL, drinking water by reverse osmosis. JAWWA 72(4):230-237.

Sorg, T.J. 1988. Methods for removing uranium from drinking water. JAWWA 80(7):105-111.

Sorg, T.J., and G.S. Logsdon. 1980. Treatment technology to meet the interim primary drinking water regulations for inorganics, Part 5. JAWWA 72(7):411.

Sorg, T.J., D.A. Lytle, J. Egan, and Z. Zhang. 1991. Characterization of wastes (brine) from uranium removal plant. Presented at AWWA/WEF Conference on Future of Residuals Management, Research Triangle Park, NC. August 11-19.

Spehar, R.L., and J.T. Fiandt. 1986. Acute and chronic effects of water-quality criteria-based metal mixtures on three aquatic species. Environ. Toxicol. Chem. 5(10):917-931.

Suffern, J.S., C.M. Fitzergerald, and A.T. Szluha. 1981. Trace metal concentrations in oxidation ponds. J. Water Pollut. Control Fed. 53(11):1599-1608.

Tanner, S.A., and J.E. Ongerth. 1990. Evaluating the performance of slow sand filters in northern Idaho. JAWWA 82(12):51.

Tebo, L.B. 1965. Effects of siltation, resulting from improper logging, on the bottom fauna of a small trout stream in the southern Appalachians. Progressive Fish Culturist 17:64.

Thomann, R.V., and J.A. Mueller. 1987. Principles of surface water quality modeling and control. New York, NY: Harper and Row Publishers. p. 644.

Tsang, K.R., and R.L. Hurdle. 1991. The co-disposal of water and wastewater treatment residuals—feasibility study. Presented at the 1991 AWWA/WPCF Joint Residuals Management Conference, Durham, NC. August 11-14.

U.S. Department of Health and Human Services (DHHS). 1990. Toxicological profile for aluminum. Washington, DC.

U.S. EPA. 1976a. Determination of radium removal efficiencies in Illinois water supply treatment processes. Technical Note ORP/TAD-76-2. NTIS PB296153. Washington, DC.

U.S. EPA. 1976b. Determination of radium removal efficiencies in Iowa water supply treatment processes. Technical Note ORP/TAD-76-1. NTIS PB259509. Washington, DC.

U.S. EPA. 1977. Manual of treatment techniques for meeting the interim primary drinking water regulations. EPA/600/8-77/995. Cincinnati, OH.

U.S. EPA. 1978a. Beneficial disposal of water purification plant sludges in wastewater treatment. EPA/600/2-78/089. Cincinnati, OH.

U.S. EPA. 1978b. Process design manual: Municipal sludge landfills. EPA/625/1-78/010. p. 5-9.

U.S. EPA. 1979a. Estimating water treatment plant costs. EPA/600/2-79/162a. Cincinnati, OH.

U.S. EPA. 1979b. Process design manual for sludge treatment and disposal. EPA/625/1-79/011. Cincinnati, OH.

U.S. EPA. 1983. Methods for chemical analysis of water and wastes. Revised ed. EPA/600/4-79/020. Cincinnati, OH.

U.S. EPA. 1984a. Barium and radium in water treatment plant wastes. EPA/600/2-85/006. Cincinnati, OH.

U.S. EPA. 1984b. Environmental regulations and technology: Use and disposal of municipal wastewater sludge. EPA/625/10-84/003. As printed in Metcalf and Eddy, Inc., Wastewater Engineering, McGraw-Hill. p. 772.

U.S. EPA. 1984c. MINTEQ—a computer program for calculating aqueous geochemical equilibria. EPA/600/3-84/032. Washington, DC.

U.S. EPA. 1985a. Slow sand filter maintenance costs and effects of water quality. EPA/600/2-85/056. NTIS PB85199669. Cincinnati, OH.

U.S. EPA. 1985b. Water quality assessment: A screening procedure for toxic and conventional pollutants in surface and groundwater. EPA/600/6-85/002b. NTIS PB86122504.

U.S. EPA. 1986a. National water quality inventory. 1986 Report to Congress. EPA/440/4-87/008. Washington, DC.

U.S. EPA. 1986b. Test methods for evaluating solid waste: Physical/Chemical methods, 3rd ed. SW-846. NTIS PB88239223. Washington, DC.

U.S. EPA. 1986c. Water quality criteria 1986. EPA/440/5-86/001. NTIS PB87226759. Washington, DC.

U.S. EPA. 1987. Removal of uranium from drinking water by ion exchange and chemical clarification. EPA/600/52-87/076. Cincinnati, OH.

U.S. EPA. 1988a. Ambient water quality criteria for aluminum. EPA/440/5-86/008. Washington, DC.

U.S. EPA. 1988b. Radium removal for a small community water supply system. EPA/600/52-88/039. Cincinnati, OH.

U.S. EPA. 1988c. Reverse osmosis treatment to remove inorganic contaminants from drinking water. EPA/600/52-87/109. Cincinnati, OH.

U.S. EPA. 1989. POTW sludge sampling and analysis guidance document. U.S. Environmental Protection Agency Permits Division. NTIS PB93227957. August.

U.S. EPA. 1990a. National sewage sludge survey: Availability of information and data, and anticipated impacts on proposed regulations. Proposed rule. Fed. Reg. 55:47210. November 9.

U.S. EPA. 1990b. Radon removal techniques for small community public water supplies. EPA/600/52-90/036. Cincinnati, OH.

U.S. EPA. 1990c. Suggested guidelines for the disposal of drinking water treatment wastes containing naturally occurring radionuclides. EPA/570/F-90/018. Washington, DC.

U.S. EPA. 1991a. MINTEQA2/PRODEFA2, a geochemical assessment model for environmental systems: Version 3.0 user's manual. EPA/600/3-91/021. Washington, DC.

U.S. EPA. 1991b. National primary drinking water regulations; radionuclide; proposed rule. Fed. Reg. 56(138):33,050. July 18.

U.S. EPA. 1992a. Organic air emissions from waste management facilities. Seminar Publication. EPA/625/R-92/003. Washington, DC.

U.S. EPA. 1992b. The inorganic chemical characterization of water treatment plant residuals. EPA/600/SR-92/117. Cincinnati, OH.

U.S. EPA. 1992c. Toxicity characteristic leaching procedure, method 1311. In: Test methods for evaluating solid waste, Vol. 1C. Laboratory manual--physical/chemical methods. SW-846, Revision O. NTIS PB88239223. July.

U.S. EPA. 1993a. Quality assurance and quality control for waste containment facilities. Technical guidance document. EPA/600/R-93/182. Washington, DC.

U.S. EPA. 1993b. Sewage sludge sampling techniques (video). Washington, DC: Office of Water. March.

U.S. EPA. 1993c. Large water system byproducts treatment and disposal cost document. Draft final report. EPA/811/D-93/002. Washington, DC.

U.S. EPA. 1994a. Design, operation, and closure of municipal solid waste landfills. Seminar Publication. EPA/625/R-94/008. Washington, DC.

U.S. EPA. 1994b. Suggested guidelines for disposal of drinking water treatment waste containing radioactivity (draft). Washington, DC: Office of Groundwater and Drinking Water. June.

U.S. EPA. 1995a. Biosolids management handbook for small municipal wastewater treatment plants, U.S. Environmental Protection Agency Regions 7, 8, and 10. Denver, CO.

U.S. EPA. 1995b. Process design manual: Land application of sewage sludge and domestic septage. EPA/625/R-95/001. Cincinnati, OH.

USGS. 1982. Probabilistic estimates of maximum acceleration of velocity in rock in the contiguous United States. U.S. Geological Survey Open-file Report 82-1033.

van Nieuwenhuyze, R.F., N.E. McTigue, and R.G. Lee. 1990. Beneficial applications and innovative sludge disposal methods. In: Cornwell, D.A., and H.M.M. Koppers, eds. Slib, schlamm, sludge. Denver, CO: American Water Works Association Research Foundation. pp. 267-304.

Vandermeyden, C., and E.K. Potter. 1991. Joint composting of water and wastewater sludge. Presented at Joint AWWA/WPCF Residuals Management Conference, Research Triangle Park, NC.

Voorhees, J.R. 1974. Model studies on the effects of alum sludge on the activated sludge waste treatment process. Master's thesis. Auburn University, Auburn University, AL.

Wang, M.C., J.Q. Hull, and M. Jao. 1992a. Stabilization of water treatment plant sludge for possible use as embankment material. Transportation Research Record 1345. Washington, DC: Transportation Research Board.

Wang, M.C., J.Q. Hull, M. Jao, B.A. Dempsey, and D.A. Cornwell. 1992b. Engineering behavior of water treatment sludge. J. Environ. Engin. ASCE 118(6).

Washington, H.G. 1984. Diversity, biotic and similarity indices: A review with special relevance to aquatic ecosystems. Water Resources Res. 18(6):653-694.

Watson, J.E., Jr., and D.J. Crawford-Brown. 1991. Use of activated carbon to remove radon from drinking water. UNC-WRRTI-91-260. Chapel Hill, NC: Water Resources Research Institute of the University of North Carolina.

W.E. Gates and Associates, Inc. 1981. A study of wastewater discharges from water treatment plants. Technical Report to the Ohio River Valley Water Sanitation Commission.

Weaver, G. 1985. Sewering alum sludge: A look down the pipe. Public Works.

Werle, C.P., J.T. Novak, W.R. Knocke, and J.H. Sherrard. 1984. Mixing intensity and polymer sludge conditioning. J. Environ. Eng. Division of American Society of Civil Engineers. 110(5):191.

Westerman, A.G., and W.J. Birge. 1978. Accelerated rate of albinism in channel catfish exposed to metals. Progressive Fish Culturist 40(4):143-146.

White, R.E. 1981. Retention and release of phosphate by soil and soil constituents. Soil and Agriculture. Critical Report on Applied Chemicals 2:71.

White, S.K., and E.A. Bondietti. 1983. Removing uranium by current municipal water treatment processes. JAWWA 75(7):374-380.

Wilson, T.E., R.E. Bizzarri, T. Burke, P.E. Langdon, Jr., and C.M. Courson. 1975. Upgrading primary treatment with chemicals and water treatment sludge. J. Water Pollut. Control Fed. 47(12):2,820-2,833.

Wisconsin Department of Natural Resources. 1984. Radium in Wisconsin groundwater and removal methods for community water systems.

Wisconsin Department of Natural Resources. 1985. Interim guidelines for the disposal of liquid and solid wastes containing radium for Wisconsin water treatment plants.

Witters, H., J.H.D. Vangenechten, S. Van Puymbroeck, and O.L.J. Vanderborght. 1984. Interference of aluminum and pH on the Na-influx in an aquatic insect *Corixa punctata* (Illig.) Bull. Environ. Contam. Toxicol. 32:575-579.

Wood, C.M., and D.G. McDonald. 1987. The physiology of acid/aluminum stress in trout. In: Witters, H., and O. Vanderborght, eds. Ecophysiology of acid stress in aquatic organisms. Soc. R. Zool. Belg. 117:399-410.

Young, R.C., W.D. Burrows, and F.G. Ziegler. 1975. Biological survey of Five Mile Creek, Jefferson County, Alabama. Associated Water and Air Resources Engineers, Nashville, TN.

Zarini, S., D. Annoni, and P. Ravera. 1983. Effects produced by aluminum in freshwater communities studied by “enclosure” method. Environmental Technol. Lett. 4:247-256.

Appendix A
Survey of State Regulatory Requirements: Summary of Results

The American Water Works Association's (AWWA's) Water Treatment Residuals Management Committee and the American Society of Civil Engineers (ASCE) conducted a survey of state regulations governing the disposal of water treatment plant residuals. The results of this survey were presented at the 1991 Annual AWWA Conference and Exposition Preconference Seminar, "Current Perspectives on Water Treatment Plant Sludge Disposal: Practices and Regulations" on June 23, 1991, in Philadelphia, Pennsylvania (AWWA/ASCE, 1991). Those results are summarized below.

ALABAMA

Department of Environmental Management
Municipal Waste Department
1751 Dickinson Drive
Montgomery, AL 36130
(205) 271-7825

The state regulates water treatment plant (WTP) wastes under the Division of Municipal Wastes in the Department of Environmental Management. All discharges are regulated and require a permit. The permits have limits of waste constituents (liquid wastes) that must be monitored and cannot be exceeded. Most liquid wastes, such as backwash decant, are sent to settling basins, then discharged to surface water bodies after settling. Monofilling on site is the most common disposal practice for sludges.

ALASKA

Department of Environmental Conservation
Treatment Section
P.O. Box O
Juneau, AK 99811-1800
(907) 465-2656

No regulations exist specifically covering WTP waste disposal. A state statute requires a waste disposal permit to discharge solid or liquid waste to lands or waters of the state. The statute also provides that a state-certified U.S. Environmental Protection Agency (EPA) National Pollutant Discharge Elimination System (NPDES) permit also serves as a state waste disposal permit.

ARIZONA

Department of Environmental Quality (ADEQ)
P.O. Box 600
Phoenix, AZ 85001-0600
(602) 392-4002

ADEQ is responsible for the regulation of water plant wastes within the state. ADEQ is in the process of implementing the NPDES program and obtaining primacy from EPA. Currently, Arizona has specific regulations governing land application of wastewater biosolids; these regulations are expected to apply to WTP residuals as well. Landfilling, monofilling, and co-disposal have no additional state requirements beyond those given in the existing federal regulations. The Department's current policy allows the disposal of WTP residuals provided that 1) sampling and analysis of the residuals reveal that they are nonhazardous; and 2) the residuals are dewatered prior to disposal. No permits are required by ADEQ for disposal of nonhazardous WTP residuals.

ARKANSAS

Department of Pollution Control and Ecology
P.O. Box 8913
Little Rock, AR 72219-8913
(501) 570-2164

Arkansas has been granted the NPDES permit process by EPA. They have a general permit that must be met by all surface water treatment plants. Parameters regulated in the permit include aluminum, manganese, iron, pH, and total suspended solids (TSS). If land application is practiced, a permit is required. Co-disposal is commonly practiced in standard municipal landfills.

CALIFORNIA

Regional Water Quality Control Board
Los Angeles Region
101 Center Plaza Drive
Monterey Park, CA 91754
(213) 266-7512

California has regulations that classify wastes, including water plant wastes, as hazardous, inert, and other categories. The classification system is used to identify the appropriate management plan. The Regional Board can regulate waste streams that are discharged to surface water. A broad range of management practices is used, including monofilling, co-disposal, direct discharge, and land application. For WTP residuals to be landfilled, the landfill must be equipped with a leachate collection and removal system. The solids content must be at least 15 percent and a minimum solids-to-liquid ratio of 5 to 1 by weight must be maintained. The state also maintains criteria regarding the burial or the release of radioactive material into air, water, or sanitary sewer systems.

COLORADO

Department of Public Health and Environment (CDPHE)
Water Quality Control Division
4300 Cherry Creek Drive South
Denver, CO 80222-1530
(303) 692-3500

CDPHE classifies WTP residuals and wastewater treatment plant (WWTP) biosolids according to heavy metal and PCB concentrations. Classes 1 and 2 can be applied to agricultural land. Class 3 can be applied to non-food chain land only. Class 4 is regulated by solid waste rules. Monitoring requirements increase with the higher classifications. Common disposal practices include monofilling, land application on site, lagooning, and recycle/reuse. CDPHE permits the co-application of WTP residuals and WWTP biosolids to land at a rate at which the phosphorus fixing capability of the mixture does not exceed the available phosphorus content of the mixture.

CONNECTICUT

Not contacted.

DELAWARE

Delaware Pollution Control Branch
P.O. Box 1401
Dover, DE 19903
(302) 739-5731

Only two surface water treatment plants exist in the state. Each discharge is reviewed and permitted separately according to NPDES guidelines.

FLORIDA

Department of Environmental Regulation (DER)
Drinking Water Section
2600 Blair Stone Road
Tallahassee, FL 32399-2400
(904) 487-1762

DER regulates water treatment plant wastes through the Drinking Water Section. Florida is not a NPDES-delegated state and therefore is unable to issue NPDES permits. The majority of wastes are lime softening or reverse osmosis (RO) concentrates. RO concentrates can be disposed of through ocean outfalls. Recycling other wastewaters to the head of the plant is the most common practice. Most residuals are recycled or land-applied, with some lime softening residuals used in road bed mixtures. Monofills are also used. Radium content in residuals is a major concern.

GEORGIA

Drinking Water Program
205 Butler Street Southeast
Floyd Tower East
Suite 1066
Atlanta, GA 30334
(404) 656-2750

Georgia regulates WTP wastes under the Drinking Water Program with aid from the Wastewater Division of NPDES. Georgia law requires all WTPs to have waste disposal facilities and plans, and the Water Program enforces all permits. No specific regulations address WTP residuals. Most liquid wastes such as backwash decant are sent to settling basins, then discharged to surface water bodies after settling. Residuals are either disposed of in municipal landfills (co-disposal) or by land application.

HAWAII

Department of Health
Safe Drinking Water Branch
5 Waterfront Plaza, Suite 250-C
500 Ala Moana Boulevard
Honolulu, HI 96813
(808) 543-8258

Hawaii is aware of the concern about WTP residuals, but because of staffing and budget constraints, it is unable to actively pursue a waste treatment program. Land application is the primary method of disposal. The state is awaiting EPA guidance.

IDAHO

Division of Environmental Quality (DEQ)
Water Quality Bureau
1410 North Hilton
Boise, ID 83706
(208) 334-5865

DEQ does not administer the NPDES program, and there are no plans for developing regulations for WTP residuals at this time. The most common disposal practice is probably landfilling.

ILLINOIS

Environmental Protection Agency
2200 Churchill Road
Springfield, IL 62794-9276
(217) 785-8653

Illinois EPA regulates WTPs that discharge any treatment waste stream to waters of the state. The pollutants regulated depend on the plant, but suspended solids, pH, and iron are the primary concerns. The most common method of disposal is discharge to sanitary sewers. Lagooning is also used, with effluent discharged to streams (NPDES permit required) and solids disposed of in a landfill. Another concern is the discharge of salt backwash from zeolite softeners to sanitary sewers.

INDIANA

Department of Environmental Management
105 South Meridian Street
P.O. Box 6015
Indianapolis, IN 46206
(317) 233-4222

WTP residuals have not been a major problem for the state. Filter backwash is usually discharged to a sanitary sewer or allowed to percolate through the ground. Sludge solids from surface water plants are usually land applied. The state is responsible for administering the NPDES program.

IOWA

Department of Natural Resources (DNR)
Environmental Protection Division
Wastewater Permit Section
900 East Grand
Des Moines, IA 50319
(515) 281-8998

DNR is seeking authority from the state legislature to develop a permit by rule process in which permit requirements would be identified by a DNR rule. Discharges to surface waters require an NPDES permit. Land application of WTP residuals may be regulated similarly for industrial and sewage sludges with respect to metals. WTP residuals have not been a primary concern because of limited resources.

KANSAS

Department of Health and Environment
Bureau of Water
Forbes Field
Topeka, KS 66620
(913) 296-5503

The state regulates all types of water wastes but has not actively pursued permitting facilities. At this time, the primary objective is to keep softening sludge solids out of the streams and rivers. Rivers have been identified as nonattainment areas for suspended solids by nonpoint legislation. In the future, this may affect management of presedimentation solids. The state administers the NPDES program for discharges; lagoons are permitted through a state water pollution control permit.

KENTUCKY

Division of Waste Management
Solid Waste Branch
18 Reilly Road
Frankfort, KY 40601
(502) 564-3410

The state regulates WTP residuals under the Division of Solid Waste. While specific regulations are being written, disposal is presently handled on a case-by-case basis. Some liquid wastes, such as backwash decant, are sent to settling basins, then discharged to surface water bodies after settling, while others are recycled to the head of the plant. Roughly half of the plants land-apply sludges, and half send the solids to a landfill (co-disposal). A few monofills exist.

LOUISIANA

Department of Environmental Quality
Water Pollution
P.O. Box 82215
Baton Rouge, LA 70884-2215
(504) 765-0635

WTP residuals are regulated by the state's water pollution control programs. An NPDES permit, administered jointly by the state and EPA, is required for surface water discharges. Only liquid wastes may be discharged. Solid wastes are commonly disposed of in permitted landfills.

MAINE

Department of Environmental Protection (DEP)
(207) 582-8740

The DEP requires that all WTP residuals be dewatered to a minimum of 20 percent total solids and disposed of in a secure landfill. The landfills must be equipped with a system for leachate collection.

MARYLAND

Not contacted.

MASSACHUSETTS

(617) 292-5529

The state is in the process of updating its existing regulations. Sludge disposal is considered on a case-by-case basis. The state is concerned about some gaps in the existing regulations.

MICHIGAN

Department of Public Health (DPH)
3423 North Logan
Lansing, MI 48909
(517) 335-9216

The DPH requires water utilities to develop a management plan for WTP residuals. If the plan is acceptable, no further regulation is necessary. If backwash water is discharged to a surface water, an NPDES permit is required from the Department of Natural Resources. The most common methods of disposal include co-disposal, direct discharge, and land application.

MINNESOTA

Not contacted.

MISSISSIPPI

Office of Pollution Control
P.O. Box 10385
Jackson, MS 39289-0385
(601) 961-5171

Mississippi regulates WTP residuals under the Office of Pollution Control. New regulations take their cue from EPA and use federal effluent guidelines for wastes, particularly with regard to toxicity. Most liquid wastes, such as backwash decant, are sent to settling basins, then discharged to surface water bodies after settling. A few plants discharge to a publicly owned treatment works (POTW) or recycle washwater to the head of the plant. The majority of sludges go to municipal landfills, with some land application of waste solids. No monofills exist in the state.

MISSOURI

Department of Natural Resources
P.O. Box 176
Jefferson City, MO 65102
(314) 751-6624

WTP residuals are regulated as solid waste unless the material is applied to agricultural land or is discharged to a surface water. A solid waste permit may be required for landfill disposal, which could include requirements for TCLP testing, liners, and leachate collection systems. An NPDES permit is required for direct discharge. Solids may be discharged only into large rivers; treated filter backwash may be discharged to small rivers. No restrictions are placed on land application of lime softening sludges. Guidelines for total aluminum loadings to the soil are established for the application of alum sludges. With some limitations, brine wastes may be discharged to a sanitary sewer, abandoned quarries, gravel pits, or evaporation lagoons.

MONTANA

Department of Health and Environmental Services (MDHES)
Water Quality Building
Cogswell Building
Helena, MT 59620
(406) 444-2406

MDHES may regulate all types of WTP residuals on a case-by-case basis. In the future, these residuals may be addressed along with sewage sludges. NPDES regulations apply to any discharges. The majority of the sludges are from lime softening facilities and are land-applied. The primary concern of this practice is checking soil pH to ensure suitability for application.

NEBRASKA

Department of Environmental Control (NDEC)
P.O. Box 98922
Lincoln, NE 68509-8922
(402) 471-2186

NDEC regulates disposal of WTP residuals on a case-by-case basis. In the future, residuals may be addressed along with sewage sludges. NPDES regulations apply to any discharges. The majority of the sludges are from lime softening facilities and are land-applied.

NEVADA

Division of Environmental Protection
123 West Nye Lane
Carson City, NV 89710
(702) 687-5872

In general, WTP residuals are not a major concern because they are nonhazardous. No permits are required for landfilling solid waste; however, discharge of treated washwater to rivers requires a permit. The state plans to update existing regulations in the near future and to develop new regulations within 1 to 5 years.

NEW HAMPSHIRE

(603) 271-3503

The state encourages landfilling and land application of WTP residuals. The state plans to update existing regulations within 1 year. Concerns include heavy metals and nutrients.

NEW JERSEY

(609) 292-5550

The state regulates WTP residuals treatment and disposal processes on a case-by-case basis and is responsible for administering the NPDES program. Concerns include heavy metals and organics.

NEW MEXICO

Environmental Department
P.O. Box 26110
Santa Fe, NM 87502

An NPDES permit is required for any discharges from water treatment plants. The EPA has primacy for permitting discharges. The state receives a 106 grant from the Clean Water Act to assist EPA with compliance monitoring. The most common methods of disposal include co-disposal and direct discharge.

NEW YORK

Department of Environmental Conservation
(518) 457-1067

Permits are issued based on the type of disposal practice proposed and the sludge quality characteristics. A wide range of disposal practices, including monofilling, co-disposal, direct discharge, land application, and recycle/reuse, are commonly used. Concerns include heavy metals, organics, and nutrients.

NORTH CAROLINA

Division of Environmental Management
P.O. Box 29535
Raleigh, NC 27626
(919) 733-5083

North Carolina regulates WTP residuals through the Division of Environmental Management. Wastes are classified by the treatment process, and a certified operator is required to run the waste treatment aspects of the plant. Most liquid wastes are discharged to a POTW via a sanitary sewer. Some direct discharge, recycling to the head of the plant, and subsurface disposal of liquid wastes are practiced. Solids disposal is generally through a municipal landfill, although there are a few monofills and some land application of sludges.

NORTH DAKOTA

Department of Health
Solid Waste Division
P.O. Box 5520
Bismarck, ND 58502-5520
(701) 221-5166

The state has no specific rules for WTP residuals. Technically, all types of water treatment plant residuals are regulated; for the most part, only lime softening wastes are regulated. Solids are disposed of primarily by landfilling, which is regulated on a case-by-case basis. Landfills must have a plan established for managing the waste.

OHIO

Environmental Protection Agency
Division of Public Drinking Water
18 West Water Mark Drive
Columbus, OH 43266
(614) 644-2752

The state is responsible for administering the NPDES program. Discharge of backwash water is regulated by water quality standards and limitations have been established for TSS, pH, iron, manganese, and chlorides. The state prefers that utilities discharge to a sanitary sewer. No unregulated discharges exist, and wastes are not recycled through the plant. The primary disposal method for solids is agricultural land application of lime softening sludge. Other acceptable methods of disposal include landfilling and mixing with compost.

OKLAHOMA

State Department of Health
1000 Northeast 10th Street
Oklahoma City, OK 73152
(405) 271-7370

Oklahoma jointly administers the WTP residuals program with EPA. Permits for direct discharges require analysis for pollutants listed in 40 CFR Part 122, Appendix D, Tables II and III. The most common disposal practices include co-disposal, direct discharge, and land application. WTP residuals are being more closely controlled.

OREGON

Department of Environmental Quality
811 S.W. Sixth Avenue
Portland, OR 97204
(503) 229-5782

Disposal of WTP residuals in Oregon is regulated under solid waste management rules. A solid waste permit is required to construct and operate onsite waste handling facilities. Offsite agricultural land application practices are covered in the permit. For discharge to surface water, an NPDES permit is required. The most common methods of disposal include monofilling and co-disposal with solid waste.

PENNSYLVANIA

Department of Environmental Resources
Room 518, Executive House, South 2nd Street
Harrisburg, PA 17105
(717) 787-9035

The state has primacy for regulating WTP residuals. It regulates backwash solids, alum, ferric chloride, polymer, and activated carbon discharge. The primary concern has been solids, alum, and ferric wastes. (No softening plants exist in the state.) The NPDES classification system is used with organics, inorganics, TSS, and pH monitored. Common sludge disposal includes landfill and land application. Backwash water can be recycled to the head of the plant or lagoon-treated. No new regulations are planned.

RHODE ISLAND

Not contacted.

SOUTH CAROLINA

Department of Health and Environmental Control
2600 Bull Street
Columbia, SC 29201
(803) 734-5241

The state regulates WTP residuals under the Industrial Wastewater Division of the Department of Health and Environmental Control. Reverse osmosis concentrate has recently become a concern. Most liquid wastes, such as backwash decant, are sent to settling basins, then discharged to surface water bodies after settling. Few facilities discharge to POTWs. Future goals are to encourage recycling of all washwaters to the head of the plant. Most sludges are disposed of at municipal landfills or transferred to a WWTP for mixing and disposal with sewage sludge. Land application of sludges is not practiced.

SOUTH DAKOTA

Department of Environment and Natural Resources
Office of Drinking Water
Joe Foss Building, Room 412
Pierre, SD 57501
(605) 773-3754

Landfilling and land application are the primary methods of WTP residuals disposal. Therefore, the solid waste program covers most of the monitoring requirements. Landfills must be permitted. Radium removal is a concern, with allowable limits set at the NRC's limits for low-level radioactive wastes.

TENNESSEE

Water Quality Control Department
Industrial Facilities Section
150 Ninth Avenue North, 4th Floor
Nashville, TN 37247-3001
(615) 741-7883

Tennessee regulates WTP residuals under the Industrial Facilities Section of the Water Quality Control Department. New regulations will be developed within the year for filter backwash wastes. The most common disposal practice for liquid wastes is to send them to a POTW, while discharge to surface water bodies and recycling to the head of the plant are lesser used options. Sludges are most often disposed of in municipal landfills, yet a few plants land-apply or monofill their solid wastes.

TEXAS

Department of Environmental Health
1100 West 49th Street
Austin, TX 78756
(512) 458-7542

Permitting is performed jointly with EPA, but the Texas Water Commission controls the permitting. Treated effluent from filter backwash and clarifier blowdown is typically recycled to the plant. Waste solids are usually not discharged to the watercourse. Sludge disposal is regulated in two ways: 1) onsite disposal must be registered with the Municipal Waste Department, and 2) offsite disposal requires a solid waste permit from the Department.

UTAH

Bureau of Water Pollution Control
P.O. Box 16690
Salt Lake City, UT 84116
(801) 538-6146

The most common disposal practices for WTP residuals are landfilling and land application. The state has no specific regulations for WTP residuals. Best management practices are required for disposal. These practices include containment (no runoff), dewatering, and disposal in a way that does not create a nuisance. No discharge is allowed to natural drainage.

VERMONT

(802) 244-7831

The most common options for disposal of WTP residuals include landfilling and land application. All types of water plant wastes are regulated. The constituents of concern include heavy metals, organics, and nutrients.

VIRGINIA

Department of Health
State Water Control Board
(804) 786-1766

The Department of Health regulates backwash water reuse. Direct return of the backwash to the treatment train is discouraged. The State Water Control Board monitors and permits plants on a case-by-case basis. Constituents of concern for discharges include aluminum, TSS, pH, and chlorine residual.

WASHINGTON

Department of Ecology
Mail Stop PV-11
Olympia, WA 98504-8711
(206) 438-7037

The state has established guidelines for the disposal of WTP residuals. The guidelines establish NPDES permit requirements, including effluent limitations and monitoring requirements, and guidelines for permitting mechanical dewatering and sewage disposal facilities. A state statute allows plants on specific rivers to discharge sediment to the originating river. EPA may prohibit this practice by not concurring with the draft NPDES permit.

WEST VIRGINIA

Department of Natural Resources (DNR)
Water Resources Division
1201 Greenbriar Street
Charleston, WV 25375
(304) 348-2107

DNR is responsible for administering the NPDES program. Treated filter backwash liquid requires a permit for discharge to rivers. Discharge criteria are established on a case-by-case basis but typically include criteria for aluminum and other metals. Solids are typically land applied, but some sewer discharge is practiced. WTP residuals have not been a primary concern for the state.

WISCONSIN

Department of Natural Resources
(608) 266-2290

Wisconsin is in the process of upgrading all regulations. The state requires a complete analysis of the waste stream, and an NPDES permit is required for discharge to surface waters. For backwash water, the state prefers pond percolation into the ground. This practice requires a ground-water discharge permit. The primary method of waste disposal is discharge to a sanitary sewer system.

WYOMING

Department of Environmental Quality
Water Quality Division
Herscher Building, 4 West
Cheyenne, WY 82002
(307) 777-7781

WTP residuals have not been a major concern. No discharges to natural drainage are allowed, and all filter backwash effluent must be recycled to the plant. Landfilling is the most common disposal practice and is regulated by the solid waste criteria, which primarily require dewatering.

Appendix B
1992 Survey of Water Treatment Plants Discharging to Wastewater Treatment Plants

All of the water treatment plants that appear in the following tables (AWWA/AWWARF, 1992) treat surface water.

	Size of Plant					**Chemicals Added at WTP**					
	Design Capacity (MGD)	**Average Daily Demand (MGD)**	**Maximum Daily Demand (MGD)**	**Peak Hourly Demand (MGD)**	**Major Processes at Water Treatment Plant**	**Coagulant (lb/day)**	**Coagulant Aid (lb/day)**	**Filter Aid (lb/day)**	**PAC (lb/day)**	**Oxidant/ Disinfectant (lb/day)**	**Other (lb/day)**
F.E. Weymouth Filtration Plant, LaVerne, CA, Metropolitan Water District	520	330	—	—	Coagulation, filtration	Alum 7,930	Cationic polymer 2,420	—	—	Chlorine 8,120	Ammonia 750
Robert A. Skinner Filtration Plant, Temecula, CA, Metropolitan Water District	520	300	402	402	Coagulation, filtration	Alum 15,290	Cationic polydadmac 3,290	—	—	Chlorine 9,935	Ammonia 1,135
Joseph Jensen Filtration Plant, Granada Hills, CA, Metropolitan Water District	550	333	—	—	Coagulation, filtration	Alum 5,510	Cationic polydadmac 2,770	Polyacrylamide 28	—	Chlorine 8,720	Ammonia 800
Dunkirk WTP, Dunkirk, NY	8	4	—	—	Coagulation, lime softening, filtration, carbon adsorption (summer)	Polyaluminum chloride 100	—	—	500–1,000 (July–Aug)	Chlorine 65	Lime 30
Chesterfield County WTP	12	10.5	—	—	Fe/Mn removal, coagulation, lime softening, filtration						
City of Myrtle Beach WTP, NC	29.5	14	20	1	Coagulation, filtration, ozone	Alum 13,000–2,000	—	20–3	—	Ozone 2,800–400	Chlorine 2,000–500

	Residuals Streams at WTP			Monitored Characteristics of Residuals Streams					Residuals Handling Facilities						
	Filter Backwash	Clarification Basin Sludge	Other	TSS	COD/ BOD	pH	Metals	Other	Equalization Basins	Chemical Recovery	Process Streams Recycled		Dewatering	Thickening	Final % Solids at Treatment Plant
F.E. Weymouth Filtration Plant, LaVerne, CA, Metropolitan Water District	31.7 MGD	1.4 MGD	—	Yes	COD	Yes	—	Sulfides	—	—	Yes	Cationic polyacrylamide	—	Yes	<3%
Robert A. Skinner Filtration Plant, Temecula, CA, Metropolitan Water District	7.5 MGD	0.036 MGD	—	Filter backwash 3%					—	—	Yes	Yes	Yes, belt press	Yes	<1%
Joseph Jensen Filtration Plant, Granada Hills, CA, Metropolitan Water District	21 MGD (max)	0.8 MGD (max)	—	Yes					—	—	Future construction to pump to head of plant	Cationic polyacrylamide	—	Yes	2-3%
Dunkirk WTP, Dunkirk, NY	Yes	Yes	—	—					Yes, holding tank	—	—	—	—	—	0.5–1.0%
Chesterfield County WTP	325,000 GPD	—	—	250					Yes	—	—	—	—	—	16%
City of Myrtle Beach WTP, NC	0.3 MGD	0.043 MGD	—	—					—	—	Backwash decant	—	Constructing mechanical dewatering processes	—	1%

	Conveyance to WWTP	Where Residuals Introduced	Problems Encountered	Formal Agreement With WWTP	Alternate Disposal/Beneficial Reuse Options
F.E. Weymouth Filtration Plant, LaVerne, CA, Metropolitan Water District	Discharge to sewer	Front end	—	Surcharge if exceed levels	Yes
Robert A. Skinner Filtration Plant, Temecula, CA, Metropolitan Water District	Discharge to sewer	Front end	—	—	This WTP has onsite reclamation plant, full liquid recovery system, and onsite disposal approved site
Joseph Jensen Filtration Plant, Granada Hills, CA, Metropolitan Water District	Discharge to sewer	Front end	—	Heavy metals not accepted; surcharge if exceed levels	Yes
Dunkirk WTP, Dunkirk, NY	Pumped through force main	Front end	—	—	—
Chesterfield County WTP	Pumped through force main	Front end	—	—	Land application
City of Myrtle Beach WTP, NC	Pumped through force main	Front end decant from alum lagoons	—	—	Composting

	Size of Plant					Chemicals Added at WTP					
	Design Capacity (MGD)	Average Daily Demand (MGD)	Maximum Daily Demand (MGD)	Peak Hourly Demand (MGD)	Major Processes at Water Treatment Plant	Coagulant (lb/day)	Coagulant Aid (lb/day)	Filter Aid (lb/day)	PAC (lb/day)	Oxidant/ Disinfectant (lb/day)	Other (lb/day)
City of Greensboro WTP, NC	44	31	44	54	Coagulation, filtration	Alum 5,900	—	—	—	Chlorine 500	—
Texarkana WTP, TX	18	11	17		Sedimentation, filtration	—	—	—	—	Chlorine dioxide chloramines	Potassium permanganate
Knoxville WTP, TN	60	30–33	40–42	34	Coagulation, filtration	Alum 600	—	20	—	Chlorine 1,200	—
Nashville WTP, TN	180	120	140	—	Coagulation, softening, filtration	Alum	—	—	—	—	Lime
Queens Lane WTP, Philadelphia, PA, City of Philadelphia	100	100	120	130	Coagulation, filtration	Ferric chloride; alum as needed	—	—	PAC as required	—	Lime
Belmont WTP, Philadelphia, PA, City of Philadelphia	80	60	75	90	Coagulation, filtration	Ferric chloride; alum backup	—	—	PAC as required	—	Lime
Franklin WTP, NC, Charlotte-Mecklenburg Utility District	96	47.6	71.4	—	Coagulation, filtration	Alum 9.0 mg/L	—	—	Carbon 0.9 mg/L	—	—
Vest WTP, NC, Charlotte-Mecklenburg Utility District	24	18.9	24	—	Coagulation, filtration	Alum 9.2 mg/L	—	—	Carbon <1.0 mg/L	—	—
City of Boulder WTP, CO	12	5.3	12	12	Coagulation, filtration	Alum	—	—	Yes	Prechlorination	—
T.W. Moses WTP, Indianapolis, IN, Indianapolis Water Company	16	10	20	1	Coagulation, filtration	Alum	Cationic polymer	—	—	Chlorine	

	Residuals Streams at WTP			Monitored Characteristics of Residuals Streams					Residuals Handling Facilities						
	Filter Backwash	**Clarification Basin Sludge**	**Other**	**TSS**	**COD/ BOD**	**pH**	**Metals**	**Other**	**Equalization Basins**	**Chemical Recovery**	**Process Streams Recycled**	**Chemical**	**Dewatering**	**Thickening**	**Final % Solids at Treatment Plant**
City of Greensboro WTP, NC	0.5 MGD	0.1 MGD	—	Not available									Yes	Yes	—
Texarkana WTP, TX	Yes	—	—	300–700 mg/L	—	—	—	—	Filter backwash and sedimentation basins						
Knoxville WTP, TN	—	Yes	—	30,000 mg/L	—	—	—	—	—	—	—	—	—	Gravity thickener	2% thickened solids
Nashville WTP, TN				Not available					Yes	—	—	—	—	—	—
Queens Lane WTP, Philadelphia, PA City of Philadelphia	Yes	Yes	—	Not available					—	—	—	—	—	—	—
Belmont WTP, City of Philadelphia, PA	Yes	Yes	—	Not available					—	—	—	—	—	—	—
Franklin WTP, NC Charlotte-Mecklenburg Utility District	Yes	Yes	—	—	—	—	Yes	Yes	Yes	—	—	—	—	—	—
Vest WTP, NC Charlotte-Mecklenburg Utility District	Yes	Yes	—	—	—	—	Yes	Yes	—	—	—	—	—	—	—
City of Boulder WTP, CO	Yes	Yes	—	—	—	—	—	—	Yes	—	Backwash water	—	—	—	—
T.W. Moses WTP, Indianapolis, IN Indianapolis Water Company	Yes	Yes	—	—	—	—	—	—	—	—	Yes	—	—	—	—

	Conveyance to WWTP	Where Residuals Introduced	Problems Encountered	Formal Agreement With WWTP	Alternate Disposal/Beneficial Reuse Options
City of Greensboro WTP, NC	Discharge to sewer	Front end	—	WTP and WWTP operated by same agency	—
Texarkana WTP, TX	Discharge to sewer	Front end	—	WTP and WWTP operated by same agency	—
Knoxville WTP, TN	Pumped through force main	Front end	Problems in flocculation basin; a lot of sedimentation; yearly maintenance	WTP and WWTP operated by same agency	—
Nashville WTP, TN	Pumped through force main	Front end; influent chamber	Minor problems on small line	—	Composting
Queens Lane WTP, Philadelphia, PA, City of Philadelphia	Discharge to sewer	Front end	WWTP can handle routine discharges but seasonal basin cleaning causes problems	Discharge restricted by permit	Evaluating onsite dewatering with disposal to landfill/reclamation on site
Belmont WTP, Philadelphia, PA, City of Philadelphia	Discharge to sewer	Front end	WWTP can handle but seasonal basin cleaning at WTP must be cleared first	Discharge restricted by permit	Evaluating onsite dewatering with disposal to landfill/reclamation on site
Franklin WTP, NC, Charlotte-Mecklenburg Utility District	Discharge to sewer	Front end	Pretreatment and toxicity concerns at WWTP; this disposal method will not be feasible in future due to requirements of discharge permits for WWTP	WTP and WWTP operated by same utility	Co-disposal with biosolids; agronomic land application; dedicated land application; turf farming
Vest WTP, NC, Charlotte-Mecklenburg Utility District	Discharge to sewer	Front end	Pretreatment and toxicity concerns at WWTP; this disposal method will not be feasible in future due to requirements of discharge permits for WWTP	WTP and WWTP operated by same utility	Co-disposal with biosolids; agronomic land application; dedicated land application; turf farming
City of Boulder WTP, CO	Discharge to sewer	Front end	—	—	Co-disposal with wastewater sludge
T.W. Moses WTP, Indianapolis, IN, Indianapolis Water Company	Gravity main	Front end	Main problem encountered is too thick a sludge for gravity flow	Allowed a no-charge level for ammonia-nitrogen, TSS, and BOD. The WTP always exceeds the levels for TSS and BOD and never exceeds nitrogen allowance. There are no restrictions	—

	Size of Plant					Chemicals Added at WTP					
	Design Capacity (MGD)	Average Daily Demand (MGD)	Maximum Daily Demand (MGD)	Peak Hourly Demand (MGD)	Major Processes at Water Treatment Plant	Coagulant (lb/day)	Coagulant Aid (lb/day)	Filter Aid (lb/day)	PAC (lb/day)	Oxidant/ Disinfectant (lb/day)	Other (lb/day)
Nottingham WTP, Cleveland, OH City of Cleveland	100	87.1	131	152	Disinfection, adsorption, oxidation, coagulation, flocculation, sedimentation, filtration, fluoridation	Alum	—	—	PAC	Chlorine	Sodium silicofluoride, potassium permanganate
Morgan WTP, Cleveland, OH City of Cleveland	150	80	140	150	Disinfection, adsorption, oxidation, coagulation, flocculation, sedimentation, filtration, fluoridation	Alum	—	—	PAC	Chlorine	Hydrofluosilicic acid
Baldwin WTP, Cleveland, OH City of Cleveland	165	85	155	162	Disinfection, adsorption, oxidation, coagulation, flocculation, sedimentation, filtration, fluoridation	Alum	—	—	PAC	Chlorine	Sodium hypochlorite, sodium silicofluoride, potassium permanganate
Erie City Water Authority, Erie, PA	60–80	35–40	60		Coagulation, filtration	Polyalum chloride	—	—	PAC	Potassium permanganate	
Conestoga WTP, Lancaster, PA City of Lancaster	16	7	10.2	0.425 MGH	Coagulation, filtration	Alum	—	—	PAC	Chlorine	Lime (hydrated), liquid hydrofluosilicic acid, potassium permanganate

	Residuals Streams at WTP			Monitored Characteristics of Residuals Streams					Residuals Handling Facilities						
	Filter Backwash	Clarification Basin Sludge	Other	TSS	COD/ BOD	pH	Metals	Other	Equalization Basins	Chemical Recovery	Process Streams Recycled	Chemical	Dewatering	Thickening	Final % Solids at Treatment Plant
Nottingham WTP, Cleveland, OH, City of Cleveland	Yes	Yes	—	2,500–4,000 mg/L	BOD 210 mg/L	7.0–7.4	—	—	Attenuation tank	—	Yes	—	—	—	—
Morgan WTP, Cleveland, OH, City of Cleveland	Yes	Yes	—	2,500–4,000 mg/L	BOD 200 mg/L	7.0–7.4	—	—	—	—	Yes	—	—	—	—
Baldwin WTP, Cleveland, OH, City of Cleveland	Yes	Yes	—	2,500–4,000 mg/L	BOD 210 mg/L	7.0–7.4	—	—	—	—	Yes	—	—	—	—
Erie City Water Authority, Erie, PA	Yes	Yes	—	30–60	2–14	7.6–8.0	—	Alum, 75 mg/L	Yes	—	Yes	—	—	Yes	—
Conestoga WTP, Lancaster, PA, City of Lancaster	Yes	Yes	—	—	—	—	—	—	—	—	—	—	—	Sludge, partially thickened	—

	Conveyance to WWTP	Where Residuals Introduced	Problems Encountered	Formal Agreement With WWTP	Alternate Disposal/Beneficial Reuse Options
Nottingham WTP, Cleveland, OH, City of Cleveland	Solids diluted if necessary and pumped through force main	Front end	—	Yes	—
Morgan WTP, Cleveland, OH, City of Cleveland	Force main	Front end	Maintenance of optimal discharge parameters for cost-effectiveness requires ongoing effort	Yes	—
Baldwin WTP, Cleveland, OH, City of Cleveland	Diluted then discharged to sewer	Front end	—	Yes	—
Erie City Water Authority, Erie, PA	Discharge to sewer	Front end	—	—	—
Conestoga WTP, Lancaster, PA, City of Lancaster	Discharge to sewer	Front end	—	—	Considering beneficial use via private hauling to Susquehanna WTP, Lancaster, PA

	Size of Plant					Chemicals Added at WTP					
	Design Capacity (MGD)	Average Daily Demand (MGD)	Maximum Daily Demand (MGD)	Peak Hourly Demand (MGD)	Major Processes at Water Treatment Plant	Coagulant (lb/day)	Coagulant Aid (lb/day)	Filter Aid (lb/day)	PAC (lb/day)	Oxidant/ Disinfectant (lb/day)	Other (lb/day)
City of Milwaukee Waterworks, Milwaukee, WI	250	Winter–60 Summer–120	171	240	Coagulation, filtration	Poly-aluminum hydroxychloride	—	—	PAC when needed	Chlorine	Ammonia, fluosilicic acid
Chattanooga WTP, TN	72	38	—	48	Coagulation, filtration, GAC	Polymers	—	—	—	Chlorine	Fluorine

	Residuals Streams at WTP			Monitored Characteristics of Residuals Streams					Residuals Handling Facilities						
	Filter Back-wash	Clarification Basin Sludge	Other	TSS	COD/ BOD	pH	Metals	Other	Equalization Basins	Chemical Recovery	Process Streams Recycled	Chemical	De-watering	Thickening	Final % Solids at Treatment Plant
City of Milwaukee Waterworks, Milwaukee, WI	Yes	Yes	Spent PAC	4,257 mg/L	310 mg/L	—	—	—	—	—	Yes	—	—	—	—
Chattanooga WTP, TN	Yes	Yes	—	—				—							

	Conveyance to WWTP	Where Residuals Introduced	Problems Encountered	Formal Agreement With WWTP	Alternate Disposal/Beneficial Reuse Options
City of Milwaukee Waterworks, Milwaukee, WI	Discharge to sewer	Front end	Sometimes exceed solids limit; sometimes need to flush sewer	Industrial surcharge; discharge restrictions; cannot exceed 20,000 lbs solids per day: Copper – 6.0 mg/L, Lead – 2.0 mg/L, Nickel – 4.0 mg/L, Zinc – 8.0 mg/L	
Chattanooga WTP, TN					

Appendix C
Charges From Publicly Owned Treatment Works

The publicly owned treatment works (POTW) charges presented in Chapter 11 were based on a limited sampling of POTW rates conducted for this analysis and on a survey published by the League of Minnesota Cities. Table C-1 presents the results of the telephone sample, and Table C-2 presents selected results from the League of Minnesota Cities Survey. Ten of the cities contacted during the telephone sample do not charge to accept wastes from water treatment plants; three were not certain. In addition, some of the POTWs contacted during the telephone sample and by the League of Minnesota Cities charge a flat rate per month with no additional charges for higher flow rates.

Table C-1. Sewer Rates—Large Cities (DPRA, Inc., 1992)

City	State	Population	Sewage Fee Charged	BOD	TSS	Misc.
Little Rock	Arkansas	175,818	$1.42/1,000 gallons + monthly service charge	>300 ppm: $0.092/pound	>300 ppm: $0.0883/pound	Oil and grease >100 ppm: $0.047/pound
Tucson	Arizona	405,390	Varies per industry		[a]	COD[a]
Chula Vista	California	138,000	$16.70/month		0–200 ppm: $1.97/1,000 gallons 200–499 ppm: $2.42/1,000 gallons >500 ppm: $3.26/1,000 gallons	
Denver	Colorado	467,610	$1.65/1,000 gallons	[b]	[b]	Pretreatment: $450/million gallons for metal treating facilities $250/million gallons for other significant industries
Derby	Connecticut	12,346	$0.82/gallon (commercial) $0.74/gallon (industrial)	No surcharge	No surcharge	
Boca Raton	Florida	49,505	$1.70/1,000 gallons	No surcharge	No surcharge	$0.25/gallon surcharge outside city limits
Lewiston	Idaho	27,986	$1.47/1,000 gallons	$0.12/pound	$0.11/pound	Industry surcharge: $0.09/1,000 gallons
Anderson	Indiana	64,695	$2.57/1,000 gallons	>300 ppm: $0.14/pound	>300 ppm: $0.12/pound	Billing charge: $0.77/month
Ft. Scott	Kansas	8,893	2,244 gallons: $6.16/month 2,244+ gallons: $1.18/1,000 gallons	No surcharge	No surcharge	
Bloomington	Minnesota	81,831	$1.53/1,000 gallons	No surcharge	No surcharge	

Table C-1. Sewer Rates—Large Cities (Continued)

City	State	Population	Sewage Fee Charged	BOD	TSS	Misc.
Duluth	Minnesota	85,000	\$4.25 + \$2.32/1,000 gallons	[c]	[c]	[c]
Minneapolis	Minnesota	370,951	\$2.21/1,000 gallons		>250 ppm: \$0.106/pound	MPCA charge for non-residential: \$25/month COD >250 ppm: \$0.053/pound
Bozeman	Montana	24,500	\$3.95 + \$0.87/1,000 gallons	No surcharge	No surcharge	
Fargo	North Dakota	76,000	\$0.70/1,000 gallons (Minimum: \$7.70/month)	No surcharge	No surcharge	
Salem	Oregon	100,000	\$0.41/1,000 gallons	\$0.15/pound	\$0.117/pound	\$1.55 billing charge; \$108/diameter inch mile
Richmond	Virginia	219,214	\$1.79/1,000 gallons	>250 ppm: \$12.82/ hundred-weight	>275 ppm: \$14.84/ hundred-weight	
Seattle	Washington	493,846	\$3.42/1,000 gallons	>300 ppm: \$0.093/pound	>400 ppm: \$0.14/pound	Metals: \$0.000806/ gallon Oils, grease, fats: \$0.000124/gallon

[a] If average TSS and COD are exceeded, a high strength factor ranging from \$1.00 to \$3.78 is assessed.
[b] (V_C) 8.34 [(0.142)(BOD–220)] + [(0.10)(TSS–250)].
[c] Surcharges unknown.

Key
BOD = Biological oxygen demand.
TSS = Total suspended solids.
COD = Chemical oxygen demand.

Table C-2. Sewer Rates—Minnesota Cities (League, 1991)

City	State	Population	Sewage Fee Charged	Misc.
Benson	Minnesota	3,656	\$2.47/1,000 gallons	
Wadena	Minnesota	4,699	\$1.55/1,000 gallons	Minimum: \$6.23
Ely	Minnesota	4,820	\$2.62/1,000 gallons	Minimum: \$7.84 Connection charge: \$75 digging fee
Chisholm	Minnesota	5,930	\$0.90/1,000 gallons	
Arden Hills	Minnesota	8,012	\$1.64/1,000 gallons	
Andover	Minnesota	9,387	Varies for “area”; ranges between \$4.50 and \$8.50/month	
Bemidji	Minnesota	10,949	Flat rate ranges between \$4.45 and \$44.00	
Owatonna	Minnesota	18,632	\$2.50/month + \$1.78/1,000 gallons	Minimum: \$2.50
Albert Lea	Minnesota	19,200	\$3.65/month	\$14 connection charge if turned off
Mankato	Minnesota	28,651	\$0.86/unit	
Burnsville	Minnesota	35,674	Commercial: 90% of winter water usage or metered water	Connection charge: \$168/unit
Duluth	Minnesota	92,811	\$3.75/month + \$1.72 to \$2.99/1,000 gallons	

Appendix D
Chemical Monthly Average Doses, Pine Valley Water Treatment Plant, Colorado Springs, CO, 1987–1992 (Pine Valley, 1994)

Table D-1. Raw Water, Pine Valley, 1987

		Water Analysis					Chemical Application						
		Alka.	**$CaC0_3$**	**pH**	**Temp.**	**Turb.**	**Alum**	**Soda Alum**	**Soda Ash**	**Chlorine**	**Polymer 8170**	**Polymer 8101**	**Polymer NP10**
Jan.	Avg.	29.60	38.50	7.65	5.96	1.24	4.05	1.04	11.73	1.64		1.03	12.22
	Max.	38.00	50.00	8.04	6.50	5.60							
	Min.	23.00	30.00	6.25	5.50	0.78							
Feb.	Avg.	28.33	38.21	7.58	6.01	1.47	3.74	1.15	14.18	1.65		1.01	12.39
	Max.	33.00	60.00	7.97	6.40	9.60							
	Min.	24.00	32.00	7.21	5.60	0.81							
Mar.	Avg.	28.70	37.00	7.60	6.30	1.41	4.47	1.03	14.2	1.51		1.00	12.06
	Max.	38.00	44.00	8.16	7.00	4.50							
	Min.	14.00	23.00	7.13	5.40	0.90							
Apr.	Avg.	29.72	36.48	7.53	6.88	1.79	4.59	1.01	14.79	1.32		1.03	12.33
	Max.	34.00	44.00	8.16	8.70	8.90							
	Min.	16.00	32.00	6.99	6.00	0.88							
May	Avg.	26.02	33.31	7.59	7.50	2.66	6.60	1.02	13.18	1.36		1.02	14.75
	Max.	30.00	48.00	8.49	8.30	10.40							
	Min.	22.00	28.00	7.10	6.30	0.89							
June	Avg.	25.78	30.86	7.44	6.93	2.26	5.86	0.97	13.56	1.30		1.00	15.97
	Max.	30.00	40.00	8.40	12.00	21.00							
	Min.	20.00	26.00	6.65	5.20	0.83							
July	Avg.	26.00	26.00	7.36	7.48	1.72	5.81	0.83	14.56	1.33		1.03	18.19
	Max.	34.00	36.00	8.80	8.50	9.20							
	Min.	24.00	24.00	6.20	6.40	0.95							
Aug.	Avg.	25.69	28.53	6.93	7.78	2.18	5.53	1.05	16.55	1.33		1.03	20.29
	Max.	34.00	34.00	7.80	9.10	3.00							
	Min.	24.00	23.00	6.70	7.00	1.06							
Sept.	Avg.	26.30	28.40	6.97	8.63	1.66	5.55	1.02	18.01	1.33		1.00	20.41
	Max.	30.00	32.00	7.70	10.00	4.40							
	Min.	24.00	25.00	6.70	7.50	1.15							
Oct.	Avg.	25.71	28.07	7.23	10.93	1.61	5.75	1.13	13.71	1.41		0.98	21.95
	Max.	30.00	34.00	7.70	12.90	5.30							
	Min.	22.00	20.00	6.60	9.50	0.73							
Nov.	Avg.	26.23	28.57	7.51	8.80	1.09	4.55	0.95	8.58	1.42		1.01	20.85
	Max.	32.00	32.00	7.99	10.50	3.00							
	Min.	21.00	26.00	6.89	6.20	0.73							
Dec.	Avg.	25.46	28.09	7.62	6.93	1.07	3.83	0.99	6.58	1.29		0.99	16.22
	Max.	42.00	40.00	7.89	8.00	5.30							
	Min.	22.00	24.00	7.19	6.00	0.75							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.

Table D-2. Raw Water, Pine Valley, 1988

		Water Analysis					Chemical Application						
		Alka.	$CaC0_3$	pH	Temp.	Turb.	Alum	Soda Alum	Soda Ash	Chlorine	Polymer 8170	Polymer 8101	Polymer NP10
Jan.	Avg.	24.48	26.03	7.61	6.29	1.15	3.92	8.96	0.89	1.36		1.02	15.65
	Max.	28.00	31.00	8.16	6.80	4.90							
	Min.	20.00	20.00	7.16	5.60	0.74							
Feb.	Avg.	24.40	27.35	7.44	6.58	1.30	3.62	9.80	1.00	1.24		1.02	18.29
	Max.	28.00	32.00	7.97	7.60	5.60							
	Min.	20.00	20.00	7.09	6.00	0.92							
Mar.	Avg.	24.89	29.55	7.25	6.39	1.27	3.49	10.23	1.03	1.22		1.00	16.91
	Max.	31.00	36.00	7.54	6.90	4.70							
	Min.	22.00	26.00	6.92	5.50	0.79							
Apr.	Avg.	24.77	29.71	7.34	6.53	1.43	5.42	11.25	0.63	1.20		0.80	15.47
	Max.	32.00	34.00	7.65	7.60	12.30							
	Min.	19.00	28.00	6.94	5.70	0.62							
May	Avg.	23.47	28.75	7.52	7.51	1.40	4.19	9.58	0.96	1.26		0.99	20.23
	Max.	30.00	34.00	8.29	8.60	3.70							
	Min.	20.00	24.00	7.09	6.30	0.55							
June	Avg.	24.53	28.38	7.43	8.88	1.29	3.59	10.33	0.96	1.26		1.01	0.57
	Max.	30.00	40.00	7.83	10.60	8.00							
	Min.	22.00	24.00	7.06	7.50	0.66							
July	Avg.	24.11	28.42	7.41	9.40	1.40	3.54	10.36	0.96	1.25		1.01	20.54
	Max.	32.00	34.00	7.70	12.90	4.40							
	Min.	22.00	23.00	7.14	7.60	0.84							
Aug.	Avg.	24.42	28.23	7.36	9.64	1.16	3.55	12.17	0.91	1.28		1.01	19.95
	Max.	28.00	32.00	7.62	10.50	5.20							
	Min.	22.00	24.00	7.11	9.00	0.74							
Sept.	Avg.	23.80	28.32	7.34	10.19	1.51	3.44	12.72	1.00	1.16		0.99	20.09
	Max.	28.00	32.00	7.65	11.00	5.70							
	Min.	18.00	26.00	7.06	8.90	0.80							
Oct.	Avg.	23.85	28.69	7.51	11.70	1.74	3.52	11.23	0.99	1.12		0.99	19.64
	Max.	26.00	32.00	7.92	13.00	4.60							
	Min.	21.00	26.00	7.06	10.40	0.63							
Nov.	Avg.	23.19	27.79	7.85	9.27	1.09	3.96	7.32	1.01	1.30		0.96	18.68
	Max.	28.00	31.00	8.33	11.70	3.37							
	Min.	20.00	26.00	7.32	6.50	0.59							
Dec.	Avg.	23.19	26.85	7.90	5.68	0.93	3.91	6.75	1.04	1.17	14.23	0.99	
	Max.	26.00	32.00	8.25	6.80	2.20							
	Min.	20.00	20.00	7.51	4.30	0.59							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.

Table D-3. Raw Water, Pine Valley, 1989

		Water Analysis					Chemical Application						
		Alka.	CaC0$_3$	pH	Temp.	Turb.	Alum	Soda Alum	Soda Ash	Chlorine	Polymer 8170	Polymer 8101	Polymer NP101P
Jan.	Avg.	22.35	27.31	7.88	4.96	0.62	3.93	7.53	1.02	1.15	13.78	0.97	13.67
	Max.	26.00	32.00	8.63	5.30	1.88							
	Min.	17.00	24.00	7.49	4.40	0.47							
Feb.	Avg.	22.34	26.96	7.89	4.95	0.70	4.00	7.92	1.02	1.12	12.73	1.00	10.79
	Max.	28.00	30.00	8.13	5.40	2.50							
	Min.	16.00	26.00	7.46	44.00	0.54							
Mar.	Avg.	21.05	26.77	7.97	5.65	0.73	4.33	8.18	0.92	1.16	0.28	0.97	0.24
	Max.	24.00	30.00	8.41	6.40	6.00							
	Min.	16.00	24.00	7.32	5.00	0.43							
Apr.	Avg.	21.90	27.00	7.89	6.52	0.65	4.69	8.45	0.96	1.15		1.09	16.33
	Max.	25.00	30.00	8.34	7.80	4.30							
	Min.	17.00	25.00	7.45	5.80	0.44							
May	Avg.	21.48	26.13	7.87	7.84	0.85	3.88	8.65	0.76	1.18	18.34	1.18	13.16
	Max.	26.00	29.00	8.44	8.60	3.90							
	Min.	17.00	22.00	7.40	6.80	0.49							
June	Avg.	21.00	26.00	7.84	8.60	1.11	3.81	9.68	0.61	1.15	17.32	1.03	17.53
	Max.	73.00	28.00	8.45	9.30	4.50							
	Min.	7.00	24.00	7.00	7.70	0.04							
July	Avg.	20.00	26.00	7.80	9.40	1.20	3.66	11.41	0.98	1.11	19.78	1.06	17.71 (NP10P)
	Max.	24.00	32.00	8.53	10.40	4.90							
	Min.	12.00	18.00	7.31	8.70	0.86							
Aug.	Avg.	21.00	26.00	7.77	10.00	1.17	3.69	12.66	0.93	1.13	18.28	1.01	0.77 (Polymer 8100)
	Max.	28.00	32.00	8.38	10.70	4.70							
	Min.	15.00	21.00	7.28	8.70	0.71							
Sept.	Avg.	23.00	27.00	7.71	11.00	1.22	3.90	13.72	1.00	1.19	18.42	1.01	
	Max.	27.00	24.00	8.21	12.20	5.70							
	Min.	20.00	30.00	7.25	10.10	0.81							
Oct.	Avg.	23.00	27.00	7.76	12.50	1.29	5.03	12.50	1.01	1.20	16.42		0.52 (Polymer 8100)
	Max.	27.00	32.00	8.35	14.20	12.40							
	Min.	19.00	24.00	7.24	11.00	0.62							
Nov.	Avg.	23.00	26.00	7.84	8.80	1.49	6.03	8.53	0.95	1.16	15.97		0.51 (Polymer 8100)
	Max.	28.00	28.00	8.25	11.20	15.00							
	Min.	18.00	24.00	7.48	6.70	0.66							
Dec.	Avg.	23.00	27.00	7.94	5.80	1.08	6.38	8.64	1.05	1.11	16.35		0.51 (Polymer 8100)
	Max.	28.00	30.00	8.23	6.80	2.90							
	Min.	19.00	24.00	7.62	4.90	0.45							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.

Table D-4. Raw Water, Pine Valley, 1990

		Water Analysis					Chemical Application						
		Alka.	CaC0$_3$	pH	Temp.	Turb.	Alum	Soda Alum	Soda Ash	Chlorine	Polymer 8100	Polymer 8170	Polymer NP10
Jan.	Avg.	23.00	27.00	7.96	5.50	1.02	5.56	9.30	1.00	1.13	0.52	16.02	
	Max.	28.00	31.00	8.30	5.80	1.98							
	Min.	14.00	24.00	7.36	5.10	0.74							
Feb.	Avg.	21.00	27.00	7.71	5.70	0.91	5.77	8.36	1.00	1.14	0.52	16.19	
	Max.	26.00	36.00	8.04	6.10	3.40							
	Min.	14.00	22.00	7.41	5.40	0.54							
Mar.	Avg.	22.00	27.00	7.47	5.90	0.88	5.90	10.40	1.04	1.23	0.51	16.16	
	Max.	28.00	32.00	9.99	6.20	2.30							
	Min.	16.00	24.00	7.18	5.50	0.62							
Apr.	Avg.	22.00	26.00	7.50	6.20	0.75	6.29	11.75	0.56	1.11	0.51	16.22	
	Max.	28.00	30.00	7.90	6.80	3.00							
	Min.	19.00	21.00	7.30	6.00	0.48							
May	Avg.	21.00	25.00	7.72	7.20	0.97	6.65	9.24	0.74	1.13	0.51	16.31	
	Max.	26.00	28.00	7.91	8.20	3.80							
	Min.	17.00	22.00	7.11	6.10	0.66							
June	Avg.	20.00	25.00	7.55	8.70	1.16	6.77	10.49	0.92	1.08	0.57	15.65	
	Max.	26.00	28.00	8.54	9.40	4.40							
	Min.	18.00	22.00	7.21	7.80	0.71							
July	Avg.	20.00	25.00	7.43	9.60	1.58	6.78	11.53	1.04	1.14	0.56	16.42	
	Max.	26.00	27.00	7.61	11.00	10.20							
	Min.	16.00	20.00	7.18	8.90	0.96							
Aug.	Avg.	21.00	25.00	7.35	9.60	1.31	6.31	11.95	1.06	1.13	0.50	16.11	
	Max.	25.00	28.00	7.51	10.20	4.90							
	Min.	18.00	20.00	7.21	9.10	0.92							
Sept.	Avg.	21.00	25.00	7.29	9.90	1.03	6.08	12.45	1.02	1.13	0.50	16.21	
	Max.	25.00	28.00	7.90	10.50	4.30							
	Min.	18.00	20.00	7.15	9.60	0.68							
Oct.	Avg.	21.00	26.00	7.26	9.90	1.07	5.81	14.40	0.96	1.18	0.50	16.11	
	Max.	26.00	28.00	7.45	10.60	9.60							
	Min.	18.00	20.00	7.04	9.60	0.77							
Nov.	Avg.	21.00	25.00	7.95	9.40	0.85	6.67	9.30	0.75	1.20	0.53	16.04	
	Max.	26.00	34.00	8.55	10.70	1.81							
	Min.	18.00	14.00	7.14	7.60	0.58							
Dec.	Avg.	21.00	25.00	8.06	5.90	0.76	7.00	8.69	0.88	1.15	0.51	16.05	
	Max.	26.00	28.00	8.75	7.80	1.06							
	Min.	19.00	22.00	7.54	4.20	10.50							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.

Table D-5. Raw Water, Pine Valley, 1991

		Water Analysis					Chemical Application						
		Alka.	CaC0$_3$	pH	Temp.	Turb.	Alum	Soda Alum	Soda Ash	Chlorine	Polymer 8100	Polymer 8170	Polymer NP10
Jan.	Avg.	22.00	25.00	7.76	5.20	1.34	7.29	10.13	0.97	1.27	0.50	15.74	
	Max.	28.00	28.00	8.02	5.50	54.00							
	Min.	18.00	22.00	7.22	4.80	0.41							
Feb.	Avg.	21.00	26.00	7.73	5.50	0.73	7.64	10.40	1.01	1.37	0.49	16.39	
	Max.	28.00	30.00	7.97	6.20	3.50							
	Min.	19.00	24.00	7.51	5.10	0.33							
Mar.	Avg.	21.00	26.00	7.57	5.90	0.62	7.56	11.11	1.02	1.38	0.49	16.28	
	Max.	26.00	30.00	7.81	6.30	2.50							
	Min.	15.00	22.00	7.29	5.40	0.37							
Apr.	Avg.	21.00	25.00	7.59	6.20	0.84	7.57	10.34	1.03	1.21	0.50	16.32	
	Max.	24.00	28.00	7.90	6.70	7.20							
	Min.	17.00	20.00	7.14	6.00	0.45							
May	Avg.	20.00	24.00	7.68	7.70	0.95	7.53	9.67	0.98	1.19	0.50	16.00	
	Max.	22.00	26.00	7.94	8.90	3.70							
	Min.	18.00	22.00	7.47	6.00	0.67							
June	Avg.	20.00	24.00	7.42	8.80	1.00	7.49	11.35	0.90	1.15	0.51	18.20	
	Max.	24.00	26.00	7.66	9.60	3.10							
	Min.	18.00	22.00	7.09	8.00	0.72							
July	Avg.	21.00	24.00	9.50	7.34	0.92	7.50	12.62	0.96	1.10	0.51	18.23	
	Max.	26.00	28.00	10.60	7.68	3.00							
	Min.	18.00	22.00	8.80	7.04	0.69							
Aug.	Avg.	20.00	24.00	9.40	7.14	1.06	7.34	14.68	0.99	1.21	0.51	18.27	
	Max.	23.00	30.00	10.30	7.30	2.70							
	Min.	17.00	20.00	8.90	7.04	0.70							
Sept.	Avg.	21.00	24.00	9.40	7.10	1.13	7.32	16.14	1.00	1.21	0.49	18.00	
	Max.	24.00	30.00	10.40	7.25	3.70							
	Min.	18.00	20.00	9.00	6.99	0.82							
Oct.	Avg.	21.00	24.00	9.80	7.00	1.11	6.76	16.89	1.09	1.20	0.51	18.81	
	Max.	24.00	28.00	10.60	7.63	2.90							
	Min.	18.00	20.00	9.20	6.88	0.68							
Nov.	Avg.	20.00	24.00	8.50	7.71	0.90	7.42	9.63	0.98	1.43	0.51	17.89	
	Max.	22.00	28.00	10.60	7.99	46.00							
	Min.	17.00	20.00	6.40	7.29	0.48							
Dec.	Avg.	20.00	24.00	5.40	7.78	0.58	7.31	9.67	0.58	1.33	0.49	18.09	
	Max.	22.00	28.00	6.60	7.95	1.61							
	Min.	18.00	20.00	4.90	7.56	0.43							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.

Table D-6. Raw Water, Pine Valley, 1992

		Water Analysis					Chemical Application						
		Alka.	$CaC0_3$	pH	Temp.	Turb.	Alum	Soda Alum	Soda Ash	Chlorine	Polymer 8100	Polymer 8170	Polymer NP10
Jan.	Avg.	20.00	24.00	5.20	7.60	0.47	7.11	10.27	1.08	1.32	0.50	18.33	
	Max.	22.00	26.00	5.70	7.79	1.43							
	Min.	18.00	20.00	4.80	7.37	0.24							
Feb.	Avg.	20.00	24.00	4.90	7.39	0.72	6.95	11.97	0.89	1.44	0.50	17.98	
	Max.	22.00	32.00	5.40	7.56	1.94							
	Min.	16.00	20.00	4.70	7.21	0.43							
Mar.	Avg.	19.00	24.00	5.20	7.37	0.88	7.02	12.22	1.03	1.45	0.50	18.20	
	Max.	22.00	30.00	5.50	7.82	10.80							
	Min.	15.00	22.00	4.90	7.13	0.53							
Apr.	Avg.	20.00	24.00	5.80	7.41	0.92	7.58	12.04	1.07	1.41	0.52	17.95	
	Max.	24.00	30.00	6.80	8.31	2.80							
	Min.	18.00	16.00	5.20	7.04	0.57							
May	Avg.	20.00	23.00	7.40	7.46	0.85	7.54	11.41	0.85	1.41	0.49	18.83	
	Max.	24.00	26.00	8.10	7.72	3.10							
	Min.	17.00	19.00	6.60	7.29	0.48							
June	Avg.	20.00	23.00	8.20	7.30	0.57	7.83	12.60	1.04	1.41	0.51	22.29	
	Max.	27.00	24.00	8.80	7.51	2.30							
	Min.	17.00	20.00	7.40	7.15	0.32							
July	Avg.	19.00	23.00	8.60	7.18	0.68	7.92	14.20	0.99	1.39	0.51	23.80	
	Max.	24.00	28.00	9.20	7.80	2.10							
	Min.	16.00	19.00	8.20	7.02	0.43							
Aug.	Avg.	19.00	24.00	9.00	7.08	0.70	7.98	15.80	0.91	1.37	0.51	24.08	
	Max.	22.00	30.00	10.30	7.51	3.70							
	Min.	16.00	20.00	7.80	6.90	0.47							
Sept.	Avg.	20.00	24.00	8.80	7.07	0.89	7.99	17.07	0.91	1.38	0.49	23.76	
	Max.	23.00	30.00	9.20	7.23	2.20							
	Min.	17.00	20.00	8.20	6.87	0.53							
Oct.	Avg.	20.00	24.00	9.40	7.03	1.12	7.50	17.24	0.95	1.41	0.50	23.99	
	Max.	22.00	26.00	9.90	7.25	2.60							
	Min.	17.00	18.00	8.80	6.90	0.79							
Nov.	Avg.	20.00	23.00	8.70	7.74	0.75	8.17	11.52	0.30	1.43	0.50	23.82	
	Max.	22.00	26.00	10.30	8.06	3.40							
	Min.	17.00	18.00	6.60	6.97	0.47							
Dec.	Avg.	20.00	24.00	5.50	7.97	0.64	8.52	10.94		1.44	0.50	24.16	
	Max.	22.00	26.00	6.60	8.37	20.00							
	Min.	17.00	20.00	5.00	7.51	0.39							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.

Appendix E
Chemical Monthly Average Doses, Mesa Water Treatment Plant, Colorado Springs, CO, 1987–1992 (Mesa, 1994)

Table E-1. Raw Water, Mesa, 1992

		Water Analysis					Chemical Application						
		Alka.	**$CaC0_3$**	**pH**	**Temp.**	**Turb.**	**Alum**	**Soda Ash**	**Soda Alum**	**Chlorine**	**Polymer 8100**	**Polymer 8170**	**Polymer NP10**
Jan.	Avg.	40.14	39.88	7.65	40.55	2.26	5.52	6.65	3.91	1.49	0.71	8.00	
	Max.	52.00	52.00	7.86	43.00	6.10							
	Min.	22.00	20.00	7.26	39.00	1.40							
Feb.	Avg.	22.71	23.17	7.35	40.65	2.85	3.99	11.57	6.36	1.62	0.80	1.20	
	Max.	28.00	28.00	7.50	43.00	79.50							
	Min.	19.00	18.00	7.20	49.00	1.60							
Mar.	Avg.	21.75	23.11	7.35	41.67	2.20	4.08	10.51	6.21	1.62	0.79		
	Max.	26.00	28.00	8.09	43.00	7.00							
	Min.	18.00	18.00	7.11	41.00	1.40							
Apr.	Avg.	20.14	21.21	7.42	42.66	2.88	4.25	11.36	6.09	1.70	0.80	7.20	
	Max.	24.00	30.00	7.69	46.00	32.00							
	Min.	16.00	16.00	7.04	41.00	1.40							
May	Avg.	18.89	19.37	7.49	46.53	1.63	4.49	11.37	3.84	1.75	0.60	8.60	
	Max.	24.00	25.00	7.74	50.00	15.50							
	Min.	16.00	16.00	7.17	44.00	0.60							
June	Avg.	18.05	17.52	7.15	50.25	2.22	6.13	14.87	3.82	2.03	0.67	8.10	
	Max.	22.00	22.00	7.69	55.00	58.10							
	Min.	16.00	12.00	6.43	45.00	0.60							
July	Avg.	17.82	16.63	7.29	53.33	1.74	4.93	11.45	6.30	2.04	0.89	0.80	
	Max.	24.00	20.00	7.73	56.00	20.70							
	Min.	14.00	12.00	7.05	51.00	0.80							
Aug.	Avg.	18.75	17.89	7.15	53.25	3.42	5.11	13.12	6.20	2.11	0.97	0.30	
	Max.	24.00	28.00	7.52	58.00	75.50							
	Min.	14.00	12.00	6.83	49.00	1.00							
Sept.	Avg.	18.47	17.78	7.02	51.91	1.65	5.33	13.90	6.24	2.06	0.94	1.30	
	Max.	24.00	28.00	7.35	55.00	19.50							
	Min.	14.00	14.00	6.73	49.00	0.90							
Oct.	Avg.	23.42	22.07	7.35	49.53	3.45	5.43	11.41	5.76	1.79	0.90	5.30	
	Max.	60.00	54.00	7.87	52.00	100.00							
	Min.	16.00	16.00	7.07	47.00	0.70							
Nov.	Avg.	43.20	40.95	7.90	41.00	3.99	5.54	8.19	2.56	1.47	0.66	8.00	
	Max.	60.00	56.00	8.73	48.00	76.00							
	Min.	22.00	20.00	7.56	36.00	0.70							
Dec.	Avg.	49.37	48.56	7.92	38.26	2.27	5.27	8.12	2.41	1.43	0.61	6.80	
	Max.	62.00	60.00	8.82	41.00	15.50							
	Min.	34.00	30.00	7.40	36.00	0.80							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.

Table E-2. Raw Water, Mesa, 1991

		Water Analysis					Chemical Application						
		Alka.	$CaC0_3$	pH	Temp.	Turb.	Alum	Soda Ash	Soda Alum	Chlorine	Polymer 8100	Polymer 8170	Polymer NP10
Jan.	Avg.	28.89	28.89	7.87	38.39	1.86	6.17	8.72		1.36	0.39	15.80	
	Max.	68.00	66.00	8.22	39.00	27.00							
	Min.	14.00	18.00	7.49	37.00	0.50							
Feb.	Avg.	51.70	53.34	7.92	39.19	2.66	7.51	10.03		2.01	0.55	18.40	
	Max.	62.00	64.00	8.16	41.00	22.00							
	Min.	20.00	24.00	7.55	37.00	1.40							
Mar.	Avg.	51.76	52.68	8.00	41.14	4.20	7.76	6.51		1.66	0.59	18.90	
	Max.	82.00	78.00	8.37	44.00	20.00							
	Min.	26.00	28.00	7.60	39.00	1.20							
Apr.	Avg.	44.33	45.68	8.10	43.17	7.74	7.96	5.63		1.79	0.62	16.80	
	Max.	56.00	58.00	8.64	46.00	196.00							
	Min.	24.00	24.00	7.49	40.00	0.90							
May	Avg.	38.60	39.40	7.98	47.71	6.69	8.56	6.35		2.05	0.69	17.20	
	Max.	54.00	54.00	8.62	55.00	122.30							
	Min.	18.00	18.00	7.52	41.00	0.70							
June	Avg.	27.34	28.15	7.74	52.41	13.33	10.57	10.43		2.39	1.13	14.60	
	Max.	54.00	54.00	8.24	58.00	230.00							
	Min.	14.00	14.00	7.39	47.00	0.70							
July	Avg.	24.52	24.19	7.45	56.43	10.14	10.28	14.36		2.37	0.86	17.30	
	Max.	44.00	40.00	7.94	60.00	900.00							
	Min.	12.00	12.00	6.93	53.00	0.70							
Aug.	Avg.	26.46	25.92	7.60	55.96	13.82	11.99	14.37	0.14	2.22	1.11	15.90	
	Max.	40.00	40.00	8.04	61.00	180.00							
	Min.	16.00	16.00	7.15	50.00	1.10							
Sept.	Avg.	27.68	26.98	7.56	54.28	5.28	6.48	8.24	1.14	2.11	0.80	14.30	
	Max.	48.00	46.00	7.91	59.00	370.00							
	Min.	14.00	16.00	7.08	50.00	0.70							
Oct.	Avg.	27.48	26.03	7.64	50.07	1.67	5.90	7.78	0.97	1.88	0.67	8.80	
	Max.	44.00	40.00	7.98	54.00	7.50							
	Min.	16.00	14.00	7.18	42.00	0.60							
Nov.	Avg.	38.96	38.60	7.88	41.78	3.01	6.01	5.81	1.12	1.65	0.71	11.80	
	Max.	58.00	56.00	8.08	45.00	31.00							
	Min.	20.00	22.00	7.58	39.00	0.70							
Dec.	Avg.	42.16	41.96	7.83	40.23	2.50	5.75	5.54	1.03	1.56	0.66	9.20	
	Max.	54.00	58.00	8.19	42.00	35.00							
	Min.	26.00	30.00	7.47	38.00	0.60							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.

Table E-3. Raw Water, Mesa, 1990

		Water Analysis					Chemical Application						
		Alka.	CaC0$_3$	pH	Temp.	Turb.	Alum	Soda Ash	Soda Alum	Chlorine	Polymer 8100	Polymer 8170	Polymer NP10
Jan.	Avg.	43.70	46.50	7.77	39.90	1.73	5.75	10.32		1.42	0.38	14.24	
	Max.	58.00	58.00	8.10	41.00	6.20							
	Min.	40.00	34.00	7.46	39.00	1.10							
Feb.	Avg.	47.60	48.40	7.81	40.30	2.19	5.27	8.81		1.30	0.33	12.60	
	Max.	70.00	66.00	8.16	43.00	34.30							
	Min.	24.00	28.00	7.49	39.00	0.50							
Mar.	Avg.	40.90	43.60	7.66	41.30	5.47	6.61	12.00		1.56	0.47	16.70	
	Max.	60.00	62.00	8.01	42.00	107.80							
	Min.	24.00	26.00	7.32	39.00	1.20							
Apr.	Avg.	42.50	44.10	7.78	43.70	15.49	9.40	13.92		1.72	0.76	14.92	
	Max.	62.00	66.00	8.09	47.00	180.00							
	Min.	20.00	22.00	7.29	41.00	1.70							
May	Avg.	31.30	33.40	7.77	46.50	7.11	9.72	14.66		1.90	0.79	14.61	
	Max.	46.00	46.00	8.04	53.00	91.70							
	Min.	15.00	16.00	7.34	41.00	0.60							
June	Avg.	30.10	29.90	7.39	55.50	7.91	5.61	14.55		2.38	1.57	16.88	
	Max.	44.00	44.00	7.92	59.00	615.00							
	Min.	14.00	16.00	6.91	48.00	0.80							
July	Avg.	26.00	25.30	7.31	56.20	47.50	7.79	19.61		2.71	2.00	14.59	
	Max.	46.00	42.00	8.87	62.00	2,700.00							
	Min.	12.00	10.00	6.82	52.00	1.00							
Aug.	Avg.	24.50	23.80	7.56	54.90	11.60	6.84	14.48		2.39	1.55	18.31	
	Max.	42.00	40.00	8.05	60.00	1,200.00							
	Min.	14.00	10.00	6.80	51.00	0.70							
Sept.	Avg.	33.10	31.80	7.52	54.80	3.62	5.19	12.74		2.09	1.41	18.59	
	Max.	54.00	52.00	7.94	57.00	48.00							
	Min.	12.00	15.00	7.18	51.00	0.70							
Oct.	Avg.	30.90	30.00	7.73	46.40	3.04	5.16	11.92		1.95	1.42	16.54	
	Max.	61.00	64.00	8.13	54.00	63.60							
	Min.	14.00	16.00	7.34	44.00	0.60							
Nov.	Avg.	41.10	40.00	8.05	42.20	1.59	3.21	10.97		1.61	1.22	15.95	
	Max.	96.00	84.00	8.25	46.00	51.00							
	Min.	16.00	16.00	7.83	39.00	0.50							
Dec.	Avg.	54.50	58.80	8.01	38.60	1.16	6.65	9.38		1.27	0.62	16.58	
	Max.	72.00	70.00	8.25	41.00	6.40							
	Min.	20.00	24.00	7.54	39.00	0.60							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.

Table E-4. Raw Water, Mesa, 1989

		Water Analysis					Chemical Application						
		Alka.	CaC0$_3$	pH	Temp.	Turb.	Alum	Soda Ash	Soda Alum	Chlorine	Polymer 8100	Polymer 8170	Polymer NP10
Jan.	Avg.	46.50	42.30	7.86	48.40	3.51	6.07	8.06	0.97	1.26		0.90	14.19
	Max.	60.00	62.00	8.12	50.00	135.10							
	Min.	24.00	26.00	7.58	47.00	1.30							
Feb.	Avg.	38.20	39.80	7.62	48.80	8.44	7.99	16.92	0.97	1.84	5.90	1.33	8.56
	Max.	50.00	65.00	8.02	50.00	133.00							
	Min.	20.00	18.00	7.11	47.00	1.20							
Mar.	Avg.	38.70	41.50	7.79	49.30	12.66	9.13	15.95	9.13	1.75	7.45	1.17	8.89
	Max.	48.00	52.00	8.13	55.00	288.00							
	Min.	38.00	24.00	7.53	43.00	1.80							
Apr.	Avg.	36.38	38.90	7.90	46.70	4.93	7.78	10.52		1.63	12.82	0.79	0.82 (N101P)
	Max.	56.00	52.00	8.15	51.00	79.50							
	Min.	16.00	16.00	7.70	43.00	0.80							
May	Avg.	27.60	30.50	7.88	51.10	4.31	7.37	11.35		1.53		0.73	11.30 (N101P)
	Max.	42.00	44.00	8.14	56.00	73.00							
	Min.	16.00	18.00	7.61	47.00	1.00							
June	Avg.	23.70	25.90	7.64	55.10	3.09	8.50	14.83		1.81		0.90	14.28 (N101P)
	Max.	46.00	50.00	7.85	58.00	43.00							
	Min.	12.00	14.00	7.26	52.00	1.40							
July	Avg.	27.90	29.70	7.31	58.00	7.71	9.57	18.69		2.05	11.24	1.10	12.80 (N101P)
	Max.	44.00	46.00	7.57	60.00	56.00							
	Min.	16.00	18.00	6.90	56.00	1.80							
Aug.	Avg.	30.70	30.10	7.44	56.70	11.95	9.62	17.63		1.93	12.54	1.09	
	Max.	60.00	50.00	7.85	62.00	430.00							
	Min.	14.00	14.00	6.96	53.00	1.00							
Sept.	Avg.	30.00	30.80	7.34	54.00	5.06	8.47	18.46		1.73	11.64	0.90	
	Max.	50.00	46.00	7.72	58.00	475.00							
	Min.	18.00	20.00	7.05	50.00	0.80							
Oct.	Avg.	34.70	36.60	7.89	51.00	1.64	7.25	11.30		1.71	12.39	0.75	0.48 (Polymer 8100)
	Max.	48.00	54.00	8.59	55.00	15.20							
	Min.	18.00	22.00	7.28	44.00	0.50							
Nov.	Avg.	34.60	35.60	8.06	42.60	0.69	6.07	9.11		1.59	16.53		0.39 (Polymer 8100)
	Max.	58.00	67.00	8.32	45.00	4.50							
	Min.	14.00	20.00	7.71	40.00	0.30							
Dec.	Avg.	50.10	52.40	7.99	39.60	1.18	6.09	7.89		1.50	16.78		0.42 (Polymer 8100)
	Max.	62.00	64.00	8.26	42.00	7.10							
	Min.	18.00	20.00	7.55	39.00	0.40							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.

Table E-5. Raw Water, Mesa, 1988

		Water Analysis					Chemical Application						
		Alka.	**$CaC0_3$**	**pH**	**Temp.**	**Turb.**	**Alum**	**Soda Ash**	**Soda Alum**	**Chlorine**	**Polymer 8100**	**Polymer 8170**	**Polymer NP10**
Jan.	Avg.	50.90	47.40	7.94	41.20	4.71	6.38	7.14	0.19	1.22		0.70	12.05
	Max.	66.00	62.00	8.10	43.00	200.00							
	Min.	40.00	38.00	7.69	39.00	1.00							
Feb.	Avg.	49.10	46.60	7.91	40.40	6.19	6.13	6.97	1.02	1.43		0.72	12.17
	Max.	70.00	68.00	8.13	42.00	77.00							
	Min.	20.00	22.00	7.45	30.00	1.10							
Mar.	Avg.	48.50	44.60	7.77	38.30	16.21	6.33	8.60	1.00	1.70		0.96	14.53
	Max.	70.00	60.00	808.00	41.00	340.00							
	Min.	22.00	20.00	731.00	35.00	1.00							
Apr.	Avg.	32.30	30.00	7.77	39.80	7.26	8.48	10.96	1.01	1.59		1.01	15.03
	Max.	70.00	68.00	8.33	45.00	1,480.00							
	Min.	10.00	12.00	6.67	34.00	1.00							
May	Avg.	31.50	28.20	7.87	45.80	2.99	7.84	11.29	0.37	1.69		1.06	18.19
	Max.	50.00	44.00	8.34	52.00	25.00							
	Min.	14.00	8.00	7.59	41.00	1.00							
June	Avg.	23.30	21.00	7.71	55.50	10.64	9.89	14.58		1.95		1.27	15.80
	Max.	38.00	35.00	8.15	59.00	492.00							
	Min.	8.00	8.00	6.80	48.00	1.10							
July	Avg.	26.20	23.70	7.53	57.50	6.51	9.49	15.40		1.87		1.15	16.83
	Max.	158.00	126.00	8.35	63.00	567.00							
	Min.	12.00	10.00	7.19	55.00	0.80							
Aug.	Avg.	23.00	20.90	7.54	61.00	9.92	9.70	15.96		2.02		1.32	14.55
	Max.	46.00	40.00	8.70	63.00	1,330.00							
	Min.	12.00	12.00	7.19	58.00	0.80							
Sept.	Avg.	28.50	23.30	7.58	58.80	3.70	8.09	13.66		1.72		0.88	14.88
	Max.	42.00	34.00	8.07	62.00	100.00							
	Min.	14.00	12.00	7.21	56.00	0.80							
Oct.	Avg.	34.30	31.90	7.67	55.70	1.57	7.20	12.84		1.53		0.85	13.91
	Max.	50.00	48.00	8.17	60.00	16.10							
	Min.	20.00	17.00	7.15	63.00	0.70							
Nov.	Avg.	34.50	32.00	7.97	51.20	1.30	7.34	10.52		1.42		0.73	17.42
	Max.	42.00	41.00	8.25	55.00	525.00							
	Min.	22.00	22.00	7.37	48.00	0.70							
Dec.	Avg.	42.90	38.70	7.86	48.40	6.14	7.30	10.30	0.34	1.35		0.82	18.64
	Max.	60.00	52.00	8.13	49.00	168.00							
	Min.	22.00	18.00	7.44	47.00	0.60							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.

Table E-6. Raw Water, Mesa, 1987

		Water Analysis					Chemical Application						
		Alka.	$CaC0_3$	pH	Temp.	Turb.	Alum	Soda Ash	Soda Alum	Chlorine	Polymer 8100	Polymer 8170	Polymer NP10
Jan.	Avg.	35.00	35.50	7.69	36.90	3.18	8.23	13.58		1.51		0.95	21.31
	Max.	66.00	68.00	8.19	39.00	47.00							
	Min.	18.00	18.00	7.32	34.00	0.60							
Feb.	Avg.	34.70	36.50	7.64	38.80	7.04	9.18	12.41		1.72		1.08	20.12
	Max.	56.00	64.00	8.17	41.00	86.00							
	Min.	18.00	18.00	7.13	37.00	1.60							
Mar.	Avg.	49.80	53.90	7.74	38.70	63.65	12.91	10.73		2.47		1.68	18.89
	Max.	66.00	72.00	8.13	46.00	925.00							
	Min.	18.00	20.00	7.17	35.00	2.10							
Apr.	Avg.	38.20	43.50	7.68	43.40	17.45	11.88	14.23		1.77		1.57	13.91
	Max.	58.00	64.00	8.11	54.00	208.00							
	Min.	20.00	24.00	7.14	37.00	2.30							
May	Avg.	25.20	29.40	7.58	50.10	27.63	11.31	16.32		1.70		1.34	17.37
	Max.	44.00	44.00	8.57	57.00	1,554.00							
	Min.	16.00	20.00	6.59	42.00	1.40							
June	Avg.	21.70	24.30	7.47	59.40	22.03	12.30	18.89		2.12		1.63	16.89
	Max.	44.00	46.00	8.17	66.00	1,073.00							
	Min.	10.00	16.00	6.63	51.00	2.50							
July	Avg.	24.80	26.90	7.54	63.50	6.28	10.92	18.41		1.94		1.28	17.78
	Max.	40.00	46.00	8.58	67.00	27.00							
	Min.	12.00	16.00	6.81	59.00	2.50							
Aug.	Avg.	26.00	28.20	7.47	64.30	12.76	10.58	17.25		2.05		1.41	16.75
	Max.	50.00	46.00	8.88	68.00	533.00							
	Min.	12.00	14.00	6.73	61.00	0.80							
Sept.	Avg.	38.30	40.40	7.78	61.00	24.25	10.15	10.06		1.73		1.17	15.95
	Max.	58.00	58.00	8.24	65.00	861.00							
	Min.	18.00	20.00	7.29	57.00	2.90							
Oct.	Avg.	29.40	30.20	7.85	53.90	1.76	7.58	10.96		1.66		0.81	17.92
	Max.	54.00	50.00	8.29	58.00	16.60							
	Min.	14.00	15.00	7.31	50.00	0.40							
Nov.	Avg.	32.90	32.10	8.04	50.60	1.11	6.14	7.99		1.43		0.67	13.73
	Max.	68.00	54.00	8.73	56.00	3.10							
	Min.	16.00	15.00	7.59	46.00	0.30							
Dec.	Avg.	36.10	35.20	7.97	44.70	1.62	5.75	7.79		1.24		0.58	12.56
	Max.	58.00	56.00	8.24	48.00	15.40							
	Min.	18.00	20.00	7.31	42.00	0.30							

Note: Polymer 8170 dosage figures are in ppb. All other chemicals are expressed in mg/L.